HASAN KALYONCU UNIVERSITY
GRADUATE SCHOOL OF
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COMPARATIVE ANALYSIS OF HIGH-RISE RC STRUCTURES ACCORDING TO INTERNATIONAL SEISMIC DESIGN CODES

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Ali Khalid HUSSEIN

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Comparative Analysis of High-Rise RC Structures According to International Seismic Design Codes

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Supervisor

Assist .Prof. Dr. Dia Eddin NASSANI

By

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Ali Khalid Hussein

ABSTRACT

COMPARATIVE ANALYSIS OF HIGH-RISE RC STRUCTURES ACCORDING TO INTERNATIONAL SEISMIC DESIGN CODES

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The field of earthquake engineering and seismology is of great importance to structural engineers around the world. The location, size and consequences of an earthquake are variable depending on several conditions. Surface conditions, boundary/fault type and distance from the boundary and hypocenter are all elements that dictate the outcome of a seismic event. Describing the effects of an earthquake can be difficult. Early records of earthquakes date back to ancient civilizations.

In present study, a high-rise reinforced concrete building is analyzed namely, International building code and European code. This building composed of 20 floors, the area of floors (1 to 12) is 1750 m^2. and the area floors of (13 to 20) is 1225 m^2. Applied live load is equal o.4 ton /m^2 , finish floor 0.3 ton /m^2 .

The well known structure program ETABS has been used to analyze the structure .The Equivalent static method has been used to calculate the seismic loads . Table of displacement , moment story ,shear story ,torsion story ,column moment and column shear are obtained.

Keywords: Comparative Analysis of High-Rise RC Structures According to International Seismic Design Codes

ÖZET

YÜKSEK BETONARME YAPILARIN ULUSLARARASI SISMIK TASARIM KODLARINA GÖRE KARŞILAŞTIRMALI ANALIZI

HUSSEIN, Ali Khalid
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Deprem mühendisliği ve sismoloji bütün dünyada yapı mühendislerini büyük ölçüde ilgilendiren alanlardır. Depremin yeri, büyüklüğü ve sonuçları çeşitli durumlara bağlı olarak değişkenlik gösterir. Yüzey koşulları, sınır/fay türü, faydan uzaklık ve etki alan merkezinden olan uzaklıkların hepsi sismik bir olayın sonuçlarını etkileyen faktörlerdir. Bir depremin etkilerini tarif etmek zor olabilir. Depremlerle ilgili ilk kayıtlar çok eski medeniyetlere kadar dayanmaktadır.

Bu çalışmada, yüksek betonarme bir yapı uluslararası koda ve Avrupa koduna göre analiz edilmiştir. Bina 20 kattan oluşmaktadır. Kat alanları 1750 m2 (1-12. katlar arası) ve 1225 m2'dir (13-20. katlar arası). Uygulanan döşeme yükü o.4 ton /m^2, döşeme ölüm yükü ise 0.3 ton /m^2'dir.

Yapının analizi için yaygın olarak bilinen yapı analiz programı ETABS kullanılmıştır. Sismik yüklerin hesabı için eşdeğer statik yöntem kullanılmıştır. Analizler sonucunda deplasman, kat momenti, kat kesme kuvveti, kat burulma, kolon momenti ve kolon kesme kuvveti değerleri tablosu elde edilmiştir.

Anahtar Kelimeler: Yüksek Betonarme Yapıların Uluslararası Sismik Tasarım Kodlarına Göre Karşılaştırmalı Analizi

TO MY DEAR PARENTS,MY DEAR FATHER ,MY DEAR MOTHER ,MY DEAR BROTHER AND MY LOVELY SISTERS.

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I would like to express my deep gratitude my supervisor : Assist .Prof. Dr. Dia Eddin NASSANI, for suggesting the research project , and for his continuous guidance and encouragement during my work , without them it would have been impossible for this study to be completed and thanks to my friend Engineer Abbas Harage to help me .

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURE

LIST OF SYMBOLS

A_{Ed} Design value of seismic action

A_{Ek} Characteristic value of the seismic action for the reference return period

Ed Design value of action effects

$S_{e(T)}$ Elastic horizontal ground acceleration response spectrum

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

$S_{d(T)}$ Design spectrum for elastic analysis

T Vibration period of a linear single degree of freedom system

a_g Design ground acceleration on type A ground

d_g Design ground displacement

g Acceleration of gravity

q Behaviour factor

E_E Effect of the seismic action

E_{Edx}, E_{Edy} Design values of the action effects due to the horizontal components (x and y) of the seismic action

E_{Edz} Design value of the action effects due to the vertical component of the seismic action

F_i Horizontal seismic force at storey i

F_a Horizontal seismic force acting on a non-structural element (appendage)

F_b Base shear force

H Building height from the foundation or from the top of a rigid basement

L_{max}, L_{min} Larger and smaller in plan dimension of the building measured in orthogonal directions

R_d Design value of resistance

T_1 Fundamental period of vibration of a building

d Displacement

dr Design interstorey drift

e_a Accidental eccentricity of the mass of one storey from its nominal location

h Interstorey height

mi Mass of storey i

n Number of storeys above the foundation or the top of a rigid basement

zi Height of mass mi above the level of application of the seismic action

M_{Ed} Design bending moment from the analysis for the seismic design situation

CHAPTER 1

INTRODUCTION

1.1 Introduction

The primary purpose of all kinds of structural systems used in the building type of structures is to support gravity loads. The most common loads resulting from the effect of gravity are dead load, live load and snow load. Besides these vertical loads, buildings are also subjected to lateral loads caused by wind, blasting or earthquake. Lateral loads can develop high stresses, produce sway movement or cause vibration.

Therefore, it is very important for the structure to have sufficient strength against vertical loads together with adequate stiffness to resist lateral forces. In Turkey, a considerable number of buildings have reinforced concrete structural systems. This is due to economic reasons. Reinforced concrete building structures can be classified as:

1. Structural Frame Systems: The structural system consist of frames, Floor slabs, beams and columns are the basic elements of the structural system. Such frames can carry gravity loads while providing adequate stiffness.

2. Structural Wall Systems: In this type of structures, all the vertical members are made of structural walls, generally called shear walls.

3. Shear Wall–Frame Systems (Dual Systems): The system consists of reinforced concrete frames interacting with reinforced concrete shear walls. Most of the residential reinforced concrete building structures in Turkey have shear wall-frame systems.

In the last years earthquake design of structures becomes an important phenomena due to the disastrous earthquakes which cause a big human tragedy all around the world. These earthquakes show that the buildings have low seismic performance due to the usage of low quality material, low quality of workmanship and inadequacy of the design codes Since then, many new codes detailing requirements have been introduced to ensure seismic resistance.

In the design of reinforced concrete structures, both of the earthquake codes and Reinforced Concrete design codes should be followed simultaneously. These structural codes are frequently used between countries member of European Union. The main objective of this thesis is to present the difference and similarities between the international building code (IBC 2006) and European EC8 committee for standardization which are likely to be used in Turkey in the European Union entry process in the future.

1.2 Object and Scope

In present study, a high-rise reinforced concrete building is analyzed namely, International building code and European code. This building composed of 20 floors, the area of floors (1 to 12) is 1750 m^2 and the area floors of (13 to 20) is 1225 m^2. Applied live load is equal o.4 ton /m^2 , finish floor 0.3 ton /m^2 .

The well known structure program ETABS has been used to analyze the structure .The Equivalent static method has been used to calculate the seismic loads . Table of displacement , moment story ,shear story ,torsion story ,column moment and column shear are obtained.

1.3 Earthquakes loads

Earthquakes are catastrophic events that occur mostly at the boundaries of portions of the earth’s crust called tectonic plates. When movement occurs in these regions, along faults, waves are generated at the earth’s surface that can produce very destructive effects. Aftershocks are smaller quakes that occur after all large earthquakes. They are usually most intense in size and number within the first week of the original quake. They can cause very significant re-shaking of damaged structures, which makes earthquake-induced disasters more hazardous. A number of moderate quakes (6+ magnitudes on the Richter scale) have had aftershocks that were very similar in size to the original quake. Aftershocks diminish in intensity and number with time.

1.4 Previous Study

Jawad (2006) concluded from the study can be summarized as follow: Although the principles contained in the considered building regulations are generally the same, they differ in details.In general EC2 and BS8110 are not very different from ACI Code in terms of the design approach. They give similar answers and offer scope for more economical concrete structures.A true factor of safety can only be determined by comparing design loading with that at collapse. While partial safety factors for materials and loadings are not safety factors; they only reflect degrees of confidence in material properties and accuracy of load predictionEC2 and ACI Code are more extensive for design requirements point of view than BS8110. For example in permitting using higher concrete strength.After study some numerical examples; EC2 and BS8110 show close agreement in flexure plus axial compression results, while ACI Code results diverge in a less economical side.BS8110 exhibits larger allowable design shear strength of concrete.ACI Code, EC2, and BS8110 give a very close design moment capacity for steel ratios within or less than balanced steel ratio. But EC2 is more generous in doubly reinforced sections.

Basaran (2009) concluded , the EC 3 is more comprehensive and complicated than Turk Standartları Enstitiusu (TSE) 648 in terms of designprinciple. However, TSE 648 should be reviewed and republished by taking account the today's innovations. The design methods introduced in EC 3 are more detailed, descriptive and more systematic which do not give initiation to designer. Under certain loading conditions, the more economical steel sections can be chosen according to the design results in EC 3 since the design of sections is performed by limit state theory.

Tonning (2009) concluded the Eurocode static story shears for both buildings are slightly higher than that of the IBC. This means that if the buildings were to be designed based on the static story shears, the Eurocode would have provided a more seismically resistant structure. The same result show that the dynamic story shears are much higher for the IBC analysis approach. Since buildings are designed for the dynamic story shears, the IBC would therefore provide a higher level of safety than the Eurocode. In other words, if the two buildings considered in these case studies would have been constructed based on Eurocode design, they would not have nearly the same lateral strength as with the IBC

Kwon and Kareem(2013) concluded contribute to some differences in the overall loads, the standardization of wind loading codes/standards may be achievable by eliminating differences in the velocity profiles. This is especially true for the alongwind load effects as all are based on a gust loading/effect factor approach. Regarding the acrosswind and torsional directions where wake-induced effects are more dominant than the buffeting effects in the alongwind direction, a database-enabled design framework such as NALD is a more promising design procedure for better assessment of such loading effects on tall buildings. Like ASCE 7 adding such provisions in the supplementary documents of a standard, e.g., a commentary may help to further supplement empirical expressions based on reductive format and simplifications with more realistic information.

Anechitei and all (2010) concluded the Eurocode 8 uses a method of additional eccentricity in defining the computed primary eccentricity that controls the computation of the strength of the elements on the flexible side. All other norms realize a similar effect amplifying the structural eccentricity with a coefficient effect in order to take into account the dynamic effects of the torsion. Unlike other norms, the value of the additional eccentricity presented in

EC8 depends on the configuration of the building in plan, and on the ratio of the total torsion rigidity over the CR compared to its total lateral rigidity. In addition, all this norms base the strength amplification from the flexible side of the element on the structural eccentricity. Unfavourable global behavior of buildings to strong earthquakes can be substantially influenced by improper arrangement of the masses on the height of the building and in plan. Thus, the poorly functions of the establishment in a building plan may lead to pronounced lop-sided location of the masses in the construction plan and therefore to substantial increase of the distance between the CM and the CR.

Hong and all (2009) concluded The constitutive relationships for rectangular CFT columns were established based on the unified theory, and then were verified through comparison between the shaking table test data and numerical analysis results. A 3-D finite element model of Taipei 101 structure was established based on the verified constitutive relationships for the rectangular CFT columns and selected finite element types for the structural members.

Jun and all (2007) concluded an analytical framework was presented for making fragility assessments of RC high-rise buildings and its use was demonstrated in a reference application. The principal contribution presented in this paper is the development of simplified lumped-parameter analytical models that were created to run in the ZEUS–NL environment and that consisted of beam elements, rigid bars and non-linear springs.

Alberto and all (2013) concluded the global displacements and the lateral load distribution of external actions in high-rise buildings. To this purpose, the analytical formulation by Carpinteri et al. Kai and all (2012) conclude All the results of response spectrum analysis calculated by different programs are basically similar, while ETABS may miss the statistic of oblique columns, which need to be paid attention to in future designs. The results of time history analysis by SAP2000 and ETABS are roughly similar. However, SAP2000 does not have the concept of storey which made the post-processing much more complicated. Therefore, to the regular structure, ETABS is recommended; and to those gymnasium or space truss structures, SAP2000 has its irreplaceable advantages. As for the slab stress analysis, ETABS and MIDAS/Gen have their respective advantages: ETABS good at preprocessing with automatically line constraint and area division; and MIDAS/Gen does well in the post-processing such as the stresses combinations.

Munir and all (2011) concluded the factors responsible for high inelastic seismic demands and the large difference between demands obtained from the

RS and NLRHA procedures in the high rise RC core wall buildings were identified.

Tatsuya and all (2000) concluded in our development we utilized our present construction equipment and reduced the development cost by using general-purpose equipment available on the market where possible. We chose the optimal automation level of the system so that workers could still use their skills, and examined the balance of cost and effect.

Wen and all (2002) concluded the site and epicentral distance effects on the vulnerability of high-rise buildings were incorporated into the DPM. This is the first time that these effects are incorporated into the DPM following a systematic and analytical approach, instead of using a more subjective 'expert adjustment approach'.

Lu and Jiang (2011) concluded the table tests on the scaled model have been extensively employed to evaluate the overall seismic performance of complex tall buildings and accordingly revise the structural design to meet the performance objectives.

Wilkinson and Hiley (2006) concluded the construction, which requires the inversion of the rotational, rank mv, stiffness matrix, is required only at time-steps where the pattern of yielding has altered from the previous time-step. This makes it particularly attractive for non-linear response history analysis. The model presented incorporates perfectly elastic perfectly plastic moment rotation relationship for the beam to column connections, but any suitable relationship could be incorporated.

Campbell et al. (2005) conclude The dynamic characteristics of two high-rise residential buildings subjected to typhoons Imbudo and Dujuan have been presented. According to wind records from the Stonecutters Island anemometer, typhoon Imbudo was recorded as having a maximum 3-s gust wind speed of 27.3 m/s and a maximum 10-min mean wind speed of 17.0 m/s.

CHAPTER 2
GENERAL RULES

2.1 General Rules

In the following sections, the definition of seismic loads and analysis requirements to be applied to earthquake resistant design according to EC8 and IBC 2006 are explained in details.

2.1.1 General Rules of EC8

Structures in seismic regions shall be design and constructed in such a way that the following requirements are met.

• No-Collapse Requirement: The structure shall be designed and constructed to withstand the design seismic action without local or global collapse, thus retaining its structural integrity and residual load bearing capacity after seismic events.

• Damage Limitation Requirement: The structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself.

In order to satisfy the fundamental requirements of seismic design stated in EC8, the ultimate limit stated and damage limitation state should be checked. It shall be verified that the structural system has the resistance and energy dissipation capacity. The balance between resistance and energy dissipation capacity is characterized by the

values of the behavior factor “q” and the associated ductility classification. As a limiting case, for the design of structures classified as non-dissipative, no account is taken of any hysteretic energy dissipation and the behavior factor may not be taken.

An adequate degree of reliability against unacceptable damage shall be ensured by satisfying the deformation limits or other relevant limits. In structures important for civil protection the structural system shall be verified to ensure that it has sufficient resistance and stiffness to maintain the function of the vital services in the facilities for a seismic event associated with an appropriate return period.

2.1.2 General Rules of IBC 2006

Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with IBC 2006 ASCE5-7. The seismic design category for a structure is permitted to be determined in accordance with Section 1613 IBC 2006 or ASCE 7.

Exceptions:

• Detached one- and two-family dwellings, assigned to Seismic Design Category A, B or C, or located where the mapped short-period spectral response acceleration, *SS*, is less than 0.4 g.

• The seismic-force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.

• Agricultural storage structures intended only for incidental human occupancy.

• Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.

2.2 International Building Code IBC2006 and ASCE5-7

2.2.1 Scope

According to the IBC 2006 , all structures, including permanent nonstructural elements, must be designed and constructed to resist the effects of earthquake ground motions. This is to be done in accordance with ASCE 7-05(American Society of Civil Engineers, ASCE 7-05 Minimum Design Loads for Buildings and Structures*)*, which specifies all minimum design loads. The seismic design criteria of a building are to be determined according to section 1613 of the 2006 IBC or section 11 of ASCE 7-05. The ASCE 7-05 is the primary reference document for the 2006 IBC, and any seismic design in the United States therefore mainly relies on this code to supply the appropriate provisions. (International Code Council, 2006 International Building Code, Section 1613.1)

2.2.2 Seismic Design Criteria

2.2.2.1 Seismic Ground Motion Values

2.2.2.1.1 Mapped Acceleration Parameters

Ss and S_1 are mapped parameters that indicate the 5% damped spectral acceleration of the Maximum Considered Earthquake (MCE) in short and long periods (0.2s and 1.0s), respectively.

The parameters are determined from 0.2 and1.0-second spectral response accelerations shown on maps provided from the US Geological Survey .(2006 IBC, Section 1613.5.1 / ASCE 7-05, Section 11.4.1)

2.2.2.1.2 Site Classification

Any site is to be classified on a scale from A to F in accordance with the site soil properties ranging from hard rock to soft soil. The properties for each category are defined in a table giving limits for the soil shear wave velocity, standard penetration resistance and soil undrained shear strength .(2006 IBC, Section 1613.5.2 / ASCE 7-05, Section 11.4.2)

2.2.2.1.3 Site Coefficients

The site coefficients, Fa and Fv, are based on short and long period ground motions and are defined in tables in the code (see Tables 2.1 and 2.2).

They are obtained by interpolating values of the mapped spectral response acceleration and the previously determined site class. *(*2006 IBC, Section 1613.5.3 / ASCE 7-05, Section 11.4.3*)*

The maximum considered earthquake spectral response accelerations for short periods, SMS, and at 1 second period, SM1, adjusted for site class effect is determined from the following equations:

$S_{MS}=FaS_S$ (2.1)

$S_{M1}=FvS_1$ (2.2)

Table 2.1 Site coefficient, Fa. (ASCE 7-05)

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIOD				
	$S_s \le 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_s.
b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

Table 2.2 Site coefficient, Fv. (ASCE 7-05)

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT 1-SECOND PERIOD				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, S_1.
b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

2.2.2.1.4 Design Spectral Acceleration Parameters

The design spectral response acceleration is defined to be two-thirds of the MCE spectral response acceleration(2006 IBC, Section 1613.5.4 / ASCE 7-05, Section 11.4.4). Therefore the design values for the previously determined parameters can be found using the following equations:

$$S_{DS}=2/3S_{MS} \tag{2.3}$$

$$S_{D1}=2/3S_{M1} \tag{2.4}$$

2.2.2.2 Importance Factor and Occupancy Category

An importance factor, I, is to be assigned to each structure with values ranging from 1.0 to 1.5 (see Table 2.3) based on the occupancy category found in a table (see Table 2.4). This table describes the different types of buildings in each category and is sorted by the severity of consequences if a building is to collapse in a seismic event.

Category I buildings include agricultural facilities and certain temporary or minor storage structures. Category III buildings include schools and assembly facilities that contain a large number of people, and other structures considered important, such as power plants and water treatment facilities. Category IV buildings are considered essential, and include hospitals and emergency facilities. Buildings that contain large amounts of hazardous or toxic materials are also included in this category. Category II includes all structures that are not included in any other category.

Table 2.3 Importance factors. (ASCE 7-05)

Occupancy Category	*I*
I or II	1.0
III	1.25
IV	1.5

Table 2.4 Occupancy category of buildings and other structures. (ASCE 7-05)

OCCUPANCY CATEGORY	NATURE OF OCCUPANCY
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	Buildings and other structures except those listed in Occupancy Categories I, III and IV
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: • Covered structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structures with elementary school, secondary school or day care facilities with an occupant load greater than 250. • Buildings and other structures with an occupant load greater than 500 for colleges or adult education facilities. • Health care facilities with an occupant load of 50 or more resident patients, but not having surgery or emergency treatment facilities. • Jails and detention facilities. • Any other occupancy with an occupant load greater than 5,000. • Power-generating stations, water treatment for potable water, waste water treatment facilities and other public utility facilities not included in Occupancy Category IV. • Buildings and other structures not included in Occupancy Category IV containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.
IV	Buildings and other structures designated as essential facilities, including but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities. • Fire, rescue and police stations and emergency vehicle garages. • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Occupancy Category IV structures. • Structures containing highly toxic materials as defined by Section 307 where the quantity of the material exceeds the maximum allowable quantities of Table 307.1.(2). • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water treatment facilities required to maintain water pressure for fire suppression.

2.2.2.3 Seismic Design Category

The seismic design category (SDC) is a classification assigned to a structure based on its occupancy category and the severity of the design earthquake ground motion at a particular site. Structures in occupancy category I, II or III located where the S1 parameter is greater than or equal to 0.75 is assigned to SDC E. Structures in occupancy category IV located where S1 is greater or equal to 0.75 is assigned to SDC F. All other structures are assigned to a seismic category based on their occupancy category and the design spectral response acceleration coefficients, SDS and SD1 (see Table 2.5 and 2.6) *(*2006 IBC, Section 1613.5.6 / ASCE 7-05, Section 11.6)

Table 2.5 Seismic Design Category based on short period response acceleration parameter. (ASCE 7-05)

Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0.167$	A	A	A
$0.167 \leq S_{DS} < 0.33$	B	B	C
$0.33 \leq S_{DS} < 0.50$	C	C	D
$0.50 \leq S_{DS}$	D	D	D

Table 2.6 Seismic Design Category based on 1 s period response acceleration parameter.(ASCE 7-05)

Value of S_{D1}	OCCUPANCY CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067$	A	A	A
$0.067 \leq S_{D1} < 0.133$	B	B	C
$0.133 \leq S_{D1} < 0.20$	C	C	D
$0.20 \leq S_{D1}$	D	D	D

2.2.3 Seismic Design Requirements for Buildings

2.2.3.1 Structural Design Basis

The basic requirement of the code states that a building must include complete lateral and vertical force resisting systems capable of providing adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions. The design seismic forces, and their distribution over the height of the building, are established in accordance with the relevant procedures. The corresponding internal forces and deformations in the members of the structure can then be determined. The individual members, including those that are not part of the seismic force resisting system, must have adequate strength to resist the shear, axial forces, and moments determined in accordance with the standard. A continuous load path with adequate strength and stiffness is necessary to transfer all forces from the point of application to the final point of resistance. All parts of the structure between separation joints need to be interconnected to form a continuous path to the seismic force resisting system. The connections must also be capable of transmitting the seismic force induced by the parts that are being connected. (ASCE 7-05, Section 12.1)

2.2.3.2 Structural System Selection

The basic lateral and vertical seismic force resisting system is to conform to one of the types indicated in the code. These types include bearing wall, building frame, moment resisting frame systems and various dual systems,

and are divided into subcategories that specifically describe the structural systems to be used. The appropriate response modification coefficient, R, system overstrength factor, $\Omega 0$, and the deflection amplification factor, Cd, indicated for each system is to be used in determining the base shear, element design forces, and design story drift (see Table 2.7). (ASCE 7-05, Section 12.2.1).

Table 2.7 Design coefficients and factors for seismic force resisting systems. (Section of table from ASCE 7-05)

Seismic Force–Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, $\Omega_0{}^g$	Deflection Amplification Factor, $C_d{}^b$	Structural System Limitations and Building Height (ft) Limit[c]				
					Seismic Design Category				
					B	C	D[d]	E[d]	F[e]
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls	14.2 and 14.2.3.6	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls	14.2 and 14.2.3.4	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls	14.2 and 14.2.3.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls	14.2 and 14.2.3.1	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls	14.2 and 14.2.3.5	4	2½	4	NL	NL	40[k]	40[k]	40[k]
6. Ordinary precast shear walls	14.2 and 14.2.3.3	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4 and 14.4.3	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4 and 14.4.3	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1, 14.1.4.2, and 14.5	6½	3	4	NL	NL	65	65	65
14. Light-framed walls with shear panels of all other materials	14.1, 14.1.4.2, and 14.5	2	2½	2	NL	NL	35	NP	NP
15. Light-framed wall systems using flat strap bracing	14.1, 14.1.4.2, and 14.5	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames, moment resisting connections at columns away from links	14.1	8	2	4	NL	NL	160	160	100
2. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	14.1	7	2	4	NL	NL	160	160	100
3. Special steel concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100

Table 2.7 Design coefficients and factors for seismic force resisting systems. (Section of table from ASCE 7-05)

Seismic Force–Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, $\Omega_0{}^g$	Deflection Amplification Factor, $C_d{}^b$	Structural System Limitations and Building Height (ft) Limit[c]				
					Seismic Design Category				
					B	C	D[d]	E[d]	F[e]
22. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
23. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1, 14.1.4.2, and 14.5	7	2½	4½	NL	NL	65	65	65
24. Light-framed walls with shear panels of all other materials	14.1, 14.1.4.2, and 14.5	2½	2½	2½	NL	NL	35	NP	NP
25. Buckling-restrained braced frames, non-moment-resisting beam-column connections	14.1	7	2	5½	NL	NL	160	160	100
26. Buckling-restrained braced frames, moment-resisting beam-column connections	14.1	8	2½	5	NL	NL	160	160	100
27. Special steel plate shear wall	14.1	7	2	6	NL	NL	160	160	100
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Special steel moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Special steel truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Intermediate steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, 12.2.5.9, and 14.1	4.5	3	4	NL	NL	35[h,i]	NP[h]	NP[i]
4. Ordinary steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, and 14.1	3.5	3	3	NL	NL	NP[h]	NP[h]	NP[i]
5. Special reinforced concrete moment frames	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Special composite steel and concrete moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Intermediate composite moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP

Table 2.7 Design coefficients and factors for seismic force resisting systems. (Section of table from ASCE 7-05)

Seismic Force-Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations and Building Height (ft) Limit[c]				
					Seismic Design Category				
					B	C	D^d	E^d	F^e
E. DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
1. Special steel concentrically braced frames[f]	14.1	6	2½	5	NL	NL	35	NP	$NP^{h,i}$
2. Special reinforced concrete shear walls	14.2	6½	2½	5	NL	NL	160	100	100
3. Ordinary reinforced masonry shear walls	14.4	3	3	2½	NL	160	NP	NP	NP
4. Intermediate reinforced masonry shear walls	14.4	3½	3	3	NL	NL	NP	NP	NP
5. Composite steel and concrete concentrically braced frames	14.3	5½	2½	4½	NL	NL	160	100	NP
6. Ordinary composite braced frames	14.3	3½	2½	3	NL	NL	NP	NP	NP
7. Ordinary composite reinforced concrete shear walls with steel elements	14.3	5	3	4½	NL	NL	NP	NP	NP
8. Ordinary reinforced concrete shear walls	14.2	5½	2½	4½	NL	NL	NP	NP	NP
F. SHEAR WALL-FRAME INTERACTIVE SYSTEM WITH ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS	12.2.5.10 and 14.2	4½	2½	4	NL	NP	NP	NP	NP
G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR:	12.2.5.2								
1. Special steel moment frames	12.2.5.5 and 14.1	2½	1¼	2½	35	35	35	35	35
2. Intermediate steel moment frames	14.1	1½	1¼	1½	35	35	35^h	$NP^{h,i}$	$NP^{h,i}$
3. Ordinary steel moment frames	14.1	1¼	1¼	1¼	35	35	NP	$NP^{h,i}$	$NP^{h,i}$
4. Special reinforced concrete moment frames	12.2.5.5 and 14.2	2½	1¼	2½	35	35	35	35	35
5. Intermediate concrete moment frames	14.2	1½	1¼	1½	35	35	NP	NP	NP

2.2.3.3 Seismic Load Effects and Combinations

All members of a structure, including those that are not part of the seismic force resisting system, are to be designed using the seismic load effects described in this section. Seismic load effects are the axial, shear, and flexural member forces resulting from applied horizontal and vertical seismic forces. A redundancy factor, ρ, is assigned to the seismic force restraining system in each of two orthogonal directions for all structures, and is set to 1.3 for structures in Seismic Design Categories D, E or F. For all other structures it is set to 1.0.

The seismic load effect, E, is determined from the expression $E = E_h + E_v$, where E_h and E_v are the horizontal and vertical components of the seismic force. The horizontal load effect can be found from the equation

$Eh = \rho \cdot Q_E$,

where ρ is the redundancy factor defined above, and Q_E is the effects of horizontal seismic forces from the base shear, V, or the seismic design force, Fp. The vertical load effect is determined by the expression $E_v = 0.2 \cdot S_{DS} \cdot D$, where SDS is the design spectral response acceleration parameter defined earlier, and D is the effect of the dead load.(ASCE 7-05, Section 12.4)

2.2.3.4 Modeling Criteria

The effective seismic weight, W, of a structure includes the total dead load, 25 percent of the floor live load for areas used for storage, and the total weight of permanent equipment and partitions. For the purpose of determining forces and displacements resulting from applied loads and any imposed displacements or P-delta effects, it is necessary to construct a mathematical model of the structure.

The model must include the stiffness and strength of the elements that are important to the distribution of forces and deformations in the structure. It must also represent the distribution of mass and stiffness throughout the structure. When determining seismic loads, it is permitted to consider the structure to be fixed at the base (ASCE 7-05, Section 12.7).

2.2.3.5 Equivalent Lateral Force Procedure

2.2.3.5.1 Seismic Base Shear

The seismic base shear, V, in a given direction is determined using the expression:

$$V = C_S W \quad (2.5)$$

where C_S is the seismic response coefficient and W is the effective seismic weight.

2.2.3.5.2 Seismic Response Coefficient

The seismic response coefficient, CS, is determined from the equation:

$$C_S = \frac{S_{DS}}{\frac{R}{I}} \quad (2.6)$$

where SDS is the design response acceleration parameter defined earlier, R is the response modification factor, and I is the occupancy importance factor.

The coefficient does not need to be greater than:

: where T is the fundamental period of the structure.

$$C_S = \frac{S_{D1}}{T(\frac{R}{I})} \qquad (2.7)$$

The coefficient does not need to be greater than:

: where T is the fundamental period of the structure

The coefficient must not be less than:

$$C_S = 0.044.S_{ds}.I \qquad (2.8)$$

2.2.3.5.3 Period Determination

The fundamental period of the structure, T, in the direction under consideration is established using the structural properties and deformational characteristics of the resisting elements in a proper analysis.

As an alternative to performing an analysis to determine the fundamental period, it is permitted to use the approximate building period, Ta, calculated by using the equation:

$$Ta = C_t.H^x \qquad (2.9)$$

where hn is the height in feet above the base to the highest level of the structure .The coefficients C_t and x are determined from a table found in the code and depend on the structure type (see Table 2.9).

Table 2.8 Values of approximate period parameters Ct and x. (ASCE 7-05)

Structure Type	C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel moment-resisting frames	0.028 (0.0724)[a]	0.8
Concrete moment-resisting frames	0.016 (0.0466)[a]	0.9
Eccentrically braced steel frames	0.03 (0.0731)[a]	0.75
All other structural systems	0.02 (0.0488)[a]	0.75

[a]Metric equivalents are shown in parentheses.

2.2.3.5.4 Vertical Distribution of Seismic Forces

The lateral seismic force induced at any level can be determined using the equation:

$$F_x=C_{vx}V \tag{2.10}$$

where C_{vx} is the vertical distribution factor, and V is the total design lateral force or shear at the base of the structure. The vertical distribution factor is given by the equation:

$$C_{vx}=\frac{w_x h_x^k}{\sum_{i=1}^{N} w_i h_i^k} \tag{2.11}$$

where w is the portion of the total effective seismic weight of the structure, W, located at the assigned level, and h is the height from the base to the assigned level. The subscript i indicates the total number of levels in the structure, and the subscript x indicates the level under consideration. The exponent k relates to the period of the structure and is assigned a value of 1 for $T \leq 0.5$ s, a value of 2 for $T \geq 2.5$ s. For periods between 0.5 s and 2.5 s, the value k is determined by linear interpolation between 1 and 2.

2.2.3.5.5 Horizontal Distribution

The seismic design story shear in any story, V_x, can be determined from the equation $V_x = \Sigma F_i$, where F_i is the portion of the seismic base shear, V, induced at level i. Based on the relative lateral stiffness of the vertical resisting elements and the diaphragm, the story shear is distributed to the various vertical elements of the seismic force-resisting system.

For diaphragms that are not flexible, the distribution of lateral forces at each level must consider the effect of the inherent torsional moment, M_t, resulting from eccentricity between the locations of the centers of mass and rigidity. For flexible diaphragms, the distribution of forces to the vertical elements must account for the position and distribution of the supported masses (ASCE 7-05, Section 12.8).

2.2.3.6 Modal Response Spectrum Analysis

An analysis must be conducted to determine the natural modes of vibration of a structure. The analysis must include a sufficient number of modes to obtain a combined modal mass contribution of at least 90 percent of the actual mass in each of the orthogonal horizontal directions of response used in the model. The value for each force related design parameter of interest is computed using the properties of each mode and the response spectra divided by the quantity R/I. The value for displacement and drift quantities must be multiplied by the quantity C_d/I (ASCE 7-05, Section 12.9*)*.

2.2.4 Site Classification Procedure for Seismic Design

2.2.4.1 Definitions

The site soil must be classified in accordance with a table found in the code, which describes the site classes A through F, based on the soil properties of the upper 100 ft of the site (see Table 2.9). Where the soil properties are not known in sufficient detail to determine the site class,

site class D is to be used unless the authority having jurisdiction or geotechnical data determines site class E or F soils are present at the site. Site classes A and B are not to be assigned to a site if there is more than 10 ft of soil between the rock surface and the bottom of the spread footing or mat foundation. The properties of the soil for each site class is defined by the average shear wave velocity, vs, average field standard penetration resistance, N, and average undrained shear strength,

Table 2.9 Site classification. (ASCE 7-05)

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 feet, SEE SECTION 1613.5.5		
		Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \bar{v}_s \leq 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \leq 2{,}500$	$\bar{N} > 50$	$\bar{s}_u \geq 2{,}000$
D	Stiff soil profile	$600 \leq \bar{v}_s \leq 1{,}200$	$15 \leq \bar{N} \leq 50$	$1{,}000 \leq \bar{s}_u \leq 2{,}000$
E	Soft soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$\bar{s}_u < 1{,}000$
E	—	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ feet)		

For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929 m², 1 pound per square foot = 0.0479 kPa. N/A = Not applicable

2.3 Eurocode 8 (2004)

2.3.1 Design Criteria and Requirements

2.3.1.1 Fundamental Requirements

There are two basic requirements that are to be met in the design and construction of structures in seismic regions. The no-collapse requirement states that the structure is to withstand the seismic design loads defined in the standard without local or global collapse.

The seismic design loads are expressed in terms of the reference seismic action associated with a reference probability of exceedance, PNCR, in 50 years, and the importance factor γI.

In Norway the reference probability of exceedance, PNCR, is set to the recommended value of 10%. The other basic requirement is that of damage limitation. It states that a structure is to be designed and constructed to withstand a seismic load having larger probability of occurrence than the seismic design load, without causing damage of unreasonably high cost in comparison with the cost of the structure. The Norwegian

National Annex (NA) states that this requirement is not applicable in Norway *(Eurocode 8,* Section 2.1).

2.3.1.2 Compliance Criteria

The compliance criteria present two limit states to be checked in order to satisfy the fundamental requirements above. The ultimate limit states are those associated with collapse or other forms of structural failure that may endanger life safety. Damage limitation states are those associated with damage causing buildings to no longer meet the specified service requirements. Again, the latter limit state is not applicable in Norway (Eurocode 8, Section 2.2).

2.3.2 Ground Conditions and Seismic Loads

2.3.2.1 Ground Conditions

The ground conditions at a particular site must be identified by carrying out appropriate investigations of the soil. The construction site and the nature of the supporting ground should normally be free from risk of ground rupture, slope instability and permanent settlement caused by liquefaction in the event of an earthquake. The influence of local ground conditions on the seismic loads is accounted for using ground types A through E described by soil properties ranging from rock to soft soil.

The ground types are classified in a table, defining the appropriate soil properties in terms of parameters such as average shear wave velocity, standard penetration test blow count, and the undrained shear strength of the soil (see Table 2.11) (Eurocode 8*,* Section 3.1).

Table 2.10 Ground types. (Eurocode 8*)*

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s> 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

2.3.2.2 Seismic Loads

2.3.2.2.1 Seismic Zones

National territories are subdivided by the authorities into seismic zones depending on the local hazard. By definition, the hazard within each zone is assumed to be constant. The hazard is for the most part described by a single parameter, the reference peak ground acceleration on ground type A, a_{gR}.

The reference peak ground acceleration corresponds to the reference return period, T_{NCR}, of the seismic loads for the no-collapse requirement.

The importance factor of 1.0 is assigned to this reference return period. The local peak ground acceleration normalized to 1.0g at the frequency of

f = 40 Hz, a_{g40Hz}, is given on maps of southern and northern Norway in the National Annex. The reference peak ground acceleration, a_{gR}, is set equal to 0.8 a_{g40Hz}.

2.3.2.2.2 Basic Representation of the Seismic Loads

The earthquake motion at a given point on the surface is represented by an elastic response spectrum. The horizontal seismic action is described by two orthogonal components assumed to be independent. For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by expressions for various intervals of vibration period, T. The parameters used in the expressions vary by ground type and are found in a specified table. Similarly, expressions are outlined in the code to give the elastic response spectrum, $S_{ve}(T)$, which represents the vertical component of the seismic load. The design of structural systems for resistance to seismic forces generally permits the capacity in the non-linear range to be smaller than that

corresponding to the linear elastic response. By performing an elastic analysis based on a design spectrum, the capacity of the structure to dissipate energy is taken into account. This is done to avoid explicit inelastic structural analysis in design, and is accomplished by introducing the behavior factor q (Eurocode 8, Section 3.2).

2.3.3 Design of Buildings

2.3.3.1 Characteristics of Earthquake Resistant Buildings

2.3.3.1.1 Structural simplicity

Modeling, analysis, design, detailing and construction of simple structures have a much lower level of uncertainty than complex projects. As a result, the prediction of the seismic behavior of a simple building is much more reliable. Structural simplicity, which is characterized by the existence of clear and direct load paths, is therefore an important goal in design of buildings.

2.3.3.1.2 Uniformity

A uniform plan is recognized from the even distribution of structural elements. This results in short and direct load paths to the elements resisting the inertial forces created by the induced motions. Uniformity can be achieved by subdividing a building into dynamically independent units by the use of seismic joints.

2.3.3.1.3 Torsional resistance

In addition to lateral resistance and stiffness, buildings should possess adequate torsional resistance and stiffness to limit the development of torsional motions. The torsional moment of inertia is the resisting force of any structural element, and is highly dependent on the distance between the resisting element and the center of mass. Since the center of mass usually is located near the center of a building, the main elements resisting should be located as near the building perimeter as possible for increased effect.

2.3.3.1.4 Criteria for structural regularity

Structures are generally categorized as being either regular or non-regular. This distinction has implications for the structural model, which can be either a simplified planar model or a spatial model. It also affects the method of analysis, which can be either a simplified response spectrum analysis or a modal one. The criteria for structural regularity apply in both plan and elevation, with specific requirements for each.

2.3.3.1.5 Importance classes and factors

Buildings are classified in 4 importance classes, depending on the consequences for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse. The definitions of the importance classes are given in a table in the code, and are based on the seriousness of the consequences of failure (see Table 2.11). The values for the importance factors are given for the various importance classes in the National Annex. In addition, the importance classes for various types of buildings.(Eurocode 8, Section 4.2)

Table 2.11 Importance classes for buildings. *(*Eurocode 8)

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories.
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

2.3.3.2 Structural Analysis

2.3.3.2.1 Structural Modeling & Analysis According to EC8

The model of the building shall adequately represent the distribution of stiffness and mass in it so that all significant deformation shapes and inertia forces are properly accounted for the consideration of seismic action. The structure may be considered to consist of vertical and lateral load resisting systems connected by horizontal diaphragms.

When the floor diaphragms of the building may be taken as being rigid in their planes, the masses and the moments of inertia of each floor may be lumped at the center of gravity.

The structural analysis may be performed using two planar models for each main direction in the buildings conforming to the criteria for regularity in plan.

Accidental Torsional Effects: In order to account for uncertainties in the location of masses and in the spatial variation of seismic action, the calculated centre of mass at each floor shall be considered as being displaced from its nominal location in each direction by an accidental eccentricity.

where:

eai : The accidental eccentricity of storey mass i from its nominal location, applied in the same direction at all floors.

Li : The floor dimension perpendicular to the direction of seismic action.

Methods of Analysis: The seismic effects and the effects of the other actions included in the seismic design may be determined on the basis of the linear elastic behaviour of the structure. One of the following reference method for determining the seismic effects should be selected depending on the structural characteristics of the building.

- Lateral force method of analysis
- Modal response spectrum analysis

As an alternative to a linear method, a non-linear method may also be used such as non-linear static (pushover) analysis, non-linear time history (dynamic) analysis. Depending on the importance class of the building, linear elastic analysis may be performed using two planar models, one for each main horizontal direction, even if the criteria for regularity in plan are not satisfied, provided that all of the following special regularity conditions are met (EC8 , design of steel structure for earthquake) .

- The building shall have well-distributed and relatively rigid cladding and partitions.
- The building height shall not exceed 10 m

• The in-plane stiffness of the floors shall be large enough in comparison with the lateral stiffness of the vertical structural elements, so that a rigid diaphragm behaviour may be assumed.

• The centers of lateral stiffness and mass shall be each approximately on a vertical line and in the two horizontal directions of analysis satisfy the conditions:

$$r_x^2 > l_s^2 + e_{ox}^2 \tag{2.13}$$

$$r_y^2 > l_s^2 + e_{oy}^2 \tag{2.14}$$

where:

l_s is the radius of gyration.

r_x and r_y are the torsional radius.

e_{ox} and e_{oy} are the natural eccentricities

Lateral Force Method of Analysis: This type of analysis may be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction. And also, the structure should satisfy the criteria for regulation in plan and elevation and the following condition.

• The fundamental periods of vibration T1 in the two main directions which are smaller than the following values;

$$T_1 \leq \begin{cases} 4T_C \\ 2.0\ s \end{cases} \tag{2.15}$$

Tc: Upper limit of period of the constant spectral acceleration branch.

Base Shear Force: The seismic base shear force F_b for each horizontal direction in which the building analyzed shall be determined using the Eqn. 3.5

$$F_b = S_d(T_1)m\lambda \tag{2.16}$$

$S_d(T_1)$: Fundamental period of vibration of the building for lateral motion in the considered earthquake direction.

m : The total mass of the building above the foundation or above the top of a rigid basement computed as $\Sigma G_{k,j} + \Sigma\Psi_{E,i} * Q_{k,i}$ (where $\Psi_{E,i}$ is the combination coefficient for variable action).

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

The combination coefficients $\Psi E,i$ for the calculation of the effects of the seismic actions shall be computed from the Eqn. 2.17

$$\Psi_{EI}=\varphi\Psi_{2i} \qquad (2.17)$$

Ψ_{2i} : The combination coefficients for variable actions .

The values of φ are given in Table 2.12.

Table 2.12 Values of φ for calculating $\Psi E,i$

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

Rayleigh method may be used for the determination of the fundamental period of vibration period T1 of the building. For the buildings with heights of up to 40 m, the value of T1 may be approximated by the Eq. 3.7

$$T_I=C_t\, H^{0.75} \qquad (2.18)$$

C_t : 0.085 for the moment resistant space steel frames.

0.075 for the moment resistant space concrete frames and eccentrically braced steel frames.

0.050 for all other structures.

H : The height of the building from the foundation or from the top of a rigid basement.

Alternatively, the estimation of T1 may be made by using the Eqn. 3.8.

$$T_1=2\sqrt{d} \tag{2.19}$$

d : The lateral elastic displacement of the top of the building due to the gravity loads applied in the horizontal direction.

Distribution of the Horizontal Seismic Forces: The seismic action effects shall be determined by applying horizontal forces Fi to all stories. The horizontal force acting on storey i ;

$$F_i = F_b \frac{s_i\, m_i}{\sum s_j\, m_j} \tag{2.20}$$

Fb : The seismic base shear in accordance with Eqn. 3.10.

si, sj : The displacements of masses mi, mj in the fundamental mode shape.

mi, mj : The storey masses computed

When the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height, the horizontal forces Fi should be computed by using the heights of the masses instead of the displacements of masses.

$$Fi=Fb\frac{zi\ mi}{\sum zj\ mj} \tag{2.21}$$

zi, zj : The heights of the masses mi, mj above the level of application of the seismic action.

Torsional Effects: If the lateral stiffness and masses are symmetrically distributed in plan and unless the accidental eccentricity is taken into account by a more exact method, the accidental torsion effects may be accounted for by multiplying the action effects in the individual load resisting elements by a factor δ.

$$\delta=1+0.6\frac{x}{Le} \qquad (2.22)$$

x : The distance of the element under consideration from the centre of mass of the building in plan measured perpendicularly to the direction of the seismic action considered.

Le : The distance between the two outermost lateral load resisting elements measured perpendicularly to the direction of the seismic action considered.

If the analysis is performed using two planar models, torsional effects may be determined by doubling the accidental eccentricity e_{ai} of Eq. 3.1 and applying Eq. 3.11 with factor 1.2 instead of 0.6.

Modal Response Spectrum Analysis: This type analysis shall be applied to buildings which do not satisfy the criteria for regulation in plan and elevation. The response of all modes of vibration contributing significantly to the global response shall be taken into account.

The sum of the effective modal masses for the modes taken into account amounts in dynamic analysis of structure shall be more than 90% of the total mass of the structure. Beside to this, all modes with effective modal masses which are greater than 5% of the total mass shall be taken into account.

When using a spatial model, the above conditions should be verified for each relevant direction. If torsional modes make a significant contribution for the modes with effective modal masses in buildings, the minimum number of “k” of modes to be taken into account in a spatial analysis should satisfy both the following conditions.

$K \geq 3\sqrt{n}$ and $T_K \leq 0.20$ sec

k : The number of modes taken into account

n : The number of storeys above the foundation or the top of the a rigid basement

T_k : The period of vibration of mode k

Combination of Modal Responses: The response in two vibration modes i and j including both translational and torsional modes may be taken as independent of each other, if their periods Ti and Tj satisfy the following conditions.

$$E_E=\sqrt{\sum E_{EI}^2} \tag{2.23}$$

E_E : The seismic action effect under consideration (Force, displacement, etc.)

E_{Ei} : The value of seismic action effect due to the vibration mode i If the modes are not independent of each other, more accurate procedure for the combination of the modal maxima, such as “Complete Quadratic Combination” (CQC) shall be adopted.

Torsional Effects: Whenever a spatial model is used for the analysis, the accidental torsional effects may be determined as the envelope of the effects resulting from the application of static loadings, consisting of sets of torsional moments Mai about the vertical axis of each storey i. Mai shall be defined as the following Eqn. 2.24

$$M_{ai}=e_{ai}.Fi \tag{2.24}$$

M_{ai} : The torsional moment applied at storey i about its vertical axis

e_{ai} : The accidental eccentricity of storey mass i

F_i : The horizontal force acting on storey I

The effects of loadings should be taken into account with positive and negative signs. Combination of the Effects of the Components of the Seismic Action: The horizontal components of the seismic action shall be taken as acting simultaneously. The structural response to each component shall be evaluated separately, using the combination rules for modal responses as previously explained in modal response spectrum analysis.

Another method is that the square root of the sum of the squared values of the action effect due to each horizontal component. Beside to these, the action effects due to the combination of the horizontal components of the seismic action may be computed using both of the two following combination as an alternative method.

$$E_{Edx}+0.30E_{Edy} \tag{2.25}$$

$$0.30E_{Edx}+E_{Edy} \tag{2.26}$$

E_{Edx} : The action effects due to the application of the seismic action along the chosen horizontal axis x of the structure

E_{Edy} : The action effects due to the application of the seismic action along the chosen horizontal axis y of the structure

If the structural system or the regularity classification of the building in elevation is different in different horizontal directions, the value of the behaviour factor q may also be different. For the buildings satisfying the regularity in plan and in which walls or independent bracing systems in the two main horizontal directions are the only primary seismic elements, the seismic action may be assumed to act separately and without combination along the two main orthogonal horizontal axes of the structure.

Vertical Components of the Seismic Action: The vertical component of the seismic action should be taken into account in the cases listed below.

• For horizontal or nearly horizontal structural members spanning 20 m or more
• For horizontal or nearly horizontal cantilever components longer than 5 m
• For horizontal or nearly horizontal pre-stressed components
• For the beams supporting columns
• In base isolated structures

The effects of the vertical component shall be taken into account only for the elements under consideration stated above and their directly associated supporting elements or substructures.

If the horizontal components of the seismic action are also relevant for these elements, the following combinations may be used for the computation of the seismic action effects.

$E_{Edx}+0.30E_{Edy}+0.30E_{Edz}$ (2.27)

$0.30E_{Edx}+E_{Edy}+0.30E_{Edz}$ (2.28)

$0.30E_{Edx}+0.30E_{Edy}+E_{Edz}$ (2.29)

E_{Edx} : The action effects due to the application of the seismic action along the chosen horizontal axis x of the structure

E_{Edy} : The action effects due to the application of the seismic action along the chosen horizontal axis y of the structure

E_{Edz} : The action effects due to the application of the vertical component of the design seismic action

2.4 Definition of Load Combinations According to EC8 and IBC 2006

2.4.1 Load Combinations According to EC8

Combination of the Seismic Action with Other Actions According to EC8: The design value Ed of the effects of actions in the seismic design situation shall be determined in accordance with the following combination; [3]

Where:

G_{kj} : Characteristic values of dead loads j

γ_1 : Importance Factor

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

φ_{2i} : Combination coefficient of live loads

Q_{ki} : Characteristic value of live loads

The inertial effects of the design seismic action shall be evaluated by taking into account the presence of the masses associated with all gravity loads appearing in the following combination of actions

$$\sum G\text{kj} + \sum \Psi Ei\ Qki \qquad (2.31)$$

Where:

Ψ_{Ei} : The combination coefficient for variable action I

The values of variable action are stated in the calculation of base shear due to the seismic action in the further chapters.

The combination coefficient ΨE,i take into account the likelihood of the loads Qk,i not being present over the entire structure during the earthquake.

These coefficients may also account for a reduced participation of masses in the motion of structure due to the non-rigid connection between them.

2.4.2 Load Combinations According to ASCE7-05

2.4.2.1 Basic load combinations.

Where strength design or load and resistance factor design is used, structures and portions thereof shall resist the most critical effects from the following combinations of factored loads:

1.4(D+F) (2.32)

1.2(D+F+T)+1.6(L+H)+0.5(LrorSor R) (2.33)

1.2D+1.6(LrorSor R) +(L or 0.8 W) (2.34)

1.2D+1.6W+L+0.5(LrorSor R) (2.35)

1.2D+1.0E+L+0.2S (2.36)

0.9D+1.6W+1.6H (2.37)

0.9D+1.0E+1.6H (2.38

where

L = Live load, except roof live load, including any permitted live load reduction.

Lr = Roof live load including any permitted live load reduction.

R = Rain load.

S = Snow load.

T = Self-straining force arising from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep in component materials, movement due to differential settlement or combinations thereof.

W = Load due to wind pressure.

2.4.2.2 Load combinations using allowable stress design.

2.4.2.2.1 Basic load combinations.

Where allowable stress design (working stress design), as permitted by this code, is

used, structures and portions thereof shall resist the most critical effects resulting from the following combinations of loads:

D+F (2.39)

D+H+F+L+T (2.40)

D+H+F+(LrorS or R) (2.41)

D+H+F+0.75(L+T)+0.75(LrorS or R) (2.41)

D+H+F+(Wor 0.75 E) (2.42)

D+H+F+0.75(W OR 0.75 E)+0.75 L+0.75(Lr or S or R) (2.43)

0.6D+H+F (2.44)

$$0.6D+E+F \quad (2.34)$$

CHAPTER 3

GENERAL RULES

3.1 General rules for building structural system

3.1.1 Criteria for Structural Regularity According to EC8

The building structures are categorized into being regular or non-regular for the purpose of seismic design. This distinction has implications for the following aspects of the seismic design.

• The structure model, which can be either a simplified planar model or a spatial model.
• The method of analysis, which can be either a simplified response spectrum analysis (lateral force procedure) or a modal one.

• The value of the behavior factor q, which shall be decreased for buildings no regular in elevation. Separate consideration is given in Table 3.1 to the regularity characteristics of the building in plan and in elevation in terms of the implications of structural regularity on analysis and design.

3.1.1.1 Criteria for Regularity in Plan

The building structure shall be approximately symmetrical in plan with respect to two orthogonal axes in terms of the lateral stiffness and mass distribution. The plan configuration should be compact. For instance, each floor should be delimited by a polygonal convex line. If re-entrant corners or edge recesses exist in plan, regularity in plan may still be considered as being satisfied that these setbacks don not affect the floor in plan stiffness and the area between the outline of the floor and a convex polygonal line enveloping the floor does not exceed 5% of the floor area.

The L, C, H, I, and x plan shapes should be carefully examined with respect to the rigid diaphragm condition. The inplan stiffness of the floors should be sufficient in comparison with the lateral stiffness of the vertical structural elements. The slenderness $\lambda = L_{max} / L_{min}$ of the building shall not be higher than 4, where L_{max} and L_{min} are respectively the larger and smaller dimensions of the buildings in plan. The structural eccentricity in the direction of analysis considered shall not be greater than 30% of the torsional radius which is defined as the square root of the ratio of the global torsional stiffness to the lateral stiffness.

Regularity		Allowed Simplification		Behaviour factor
Plan	Elevation	Model	Linear elastic analysis	For linear analysis
Yes	Yes	Planar	Lateral force	Reference value
Yes	NO	Planar	Modal	Decreased value

No	Yes	Spatial	Lateral force	Reference value
No	No	Spatial	Modal	Decreased value

Table 3.1. Structural Regularity

3.1.1.2 Criteria for Regularity in Elevation

All lateral load resisting systems, such as cores, structural walls, or frames shall run without interruption from their foundations to the top of the building. Both the lateral stiffness and the mass of the individual stories shall remain constant or reduce gradually, without abrupt changes, from the base to the top of a particular building.

In framed buildings, the ratio of the actual storey resistance to the resistance required by the analysis should not vary disproportionately between adjacent stories. For the structures with setbacks in elevation, the following conditions should be considered.

• The setbacks at any floor shall not be greater than 20% of the previous plan dimension in the direction of the setback. (See Figure 3.1.a and 3.1.b)

• For a single setback within the lower 15% of the total height of the main structural system, the setback shall not be greater than 50% of the previous plan dimension. In this case the structure should be designed to resist at least 75% of the horizontal 20 shear forces that would developed in that zone in a similar building without the base enlargement. (See Figure 3.1.c)

• If the setbacks do not preserve symmetry, in each face of the sum of the setbacks at all stories shall be not greater than 30% of the plan dimension at the ground floor above the foundation or above the top of a rigid basement, and the individual setbacks shall be not greater than 10% of the previous plan dimension. (See Figure 3.1.d) (EC8 , design of steel structure for earthquake)

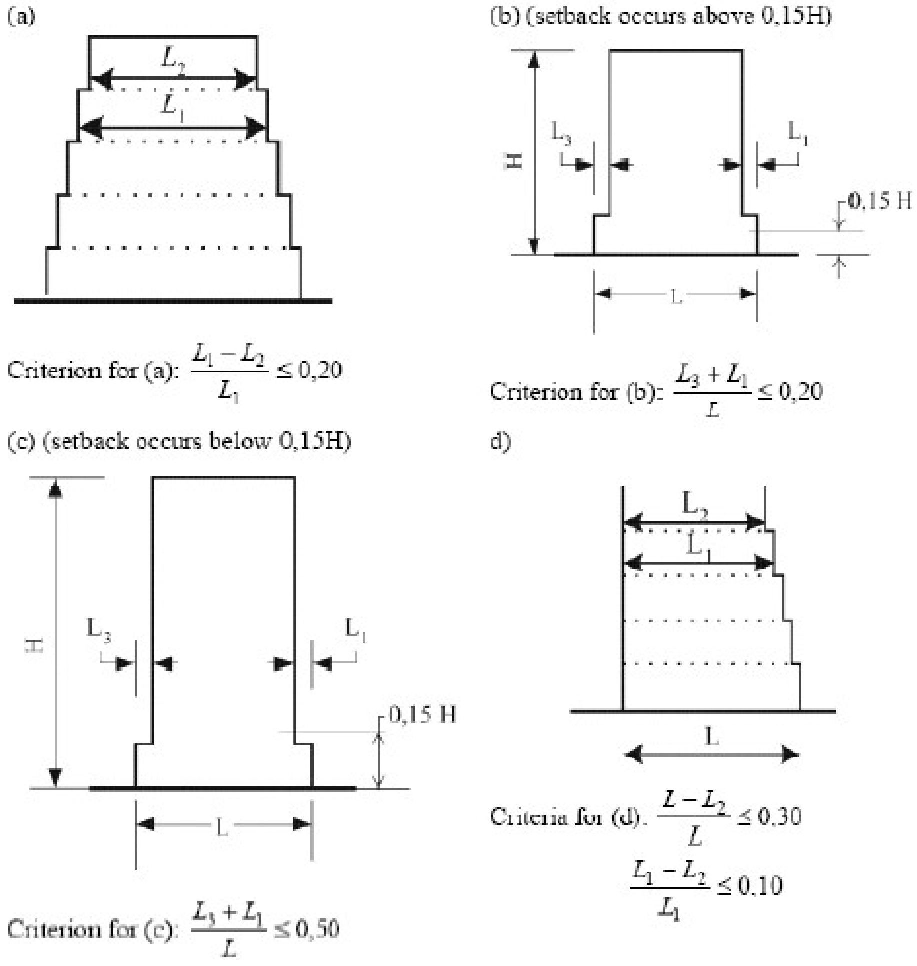

Figure 3.1 Criteria for regularity of buildings in elevation

3.1.2 Criteria for Structural Regularity According to ACSE

3.1.2.1 Irregular and Regular Classification.

Structures shall be classified as regular or irregular based upon the criteria in this section. Such classification shall be based on horizontal and vertical configurations.

3.1.2.1.1 Horizontal Irregularity.

Structures having one or tures assigned to Seismic Design Category are in more of the irregularity types listed in Table 3.2 shall be desiglight- frame construction. nated as having horizontal structural irregularity. Such structures assigned to the seismic design categories listed in Table 3.2

3.1.2.1.2 Vertical Irregularity.

Structures having one or more shall be capable of resisting at least 25 percent of the design story of the irregularity types listed in Table 3.3 shall be designated shear in every story. as having vertical irregularity. Such structures assigned to the seismic design categories listed in Table 3.3 shall comply with the requirements in-the sections referenced in that table.

Table 3.2 horizontal structural irregularities

	Irregularity Type and Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.8.4.3 12.7.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F C, D, E, and F B, C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D
2.	**Reentrant Corner Irregularity** is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
3.	**Diaphragm Discontinuity Irregularity** is defined to exist where there are diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
4.	**Out-of-Plane Offsets Irregularity** is defined to exist where there are discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	12.3.3.4 12.3.3.3 12.7.3 Table 12.6-1 16.2.2	D, E, and F B, C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
5.	**Nonparallel Systems-Irregularity** is defined to exist where the vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force–resisting system.	12.5.3 12.7.3 Table 12.6-1 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

Table 3.3 vertical structural irregularities

	Irregularity Type and Description	Reference Section	Seismic Design Category Application
1a.	**Stiffness-Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	Table 12.6-1	D, E, and F
1b.	**Stiffness-Extreme Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	12.3.3.1 Table 12.6-1	E and F D, E, and F
2.	**Weight (Mass) Irregularity** is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	Table 12.6-1	D, E, and F
3.	**Vertical Geometric Irregularity** is defined to exist where the horizontal dimension of the seismic force–resisting system in any story is more than 130% of that in an adjacent story.	Table 12.6-1	D, E, and F
4.	**In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity** is defined to exist where an in-plane offset of the lateral force-resisting elements is greater than the length of those elements or there exists a reduction in stiffness of the resisting element in the story below.	12.3.3.3 12.3.3.4 Table 12.6-1	B, C, D, E, and F D, E, and F D, E, and F
5a.	**Discontinuity in Lateral Strength–Weak Story Irregularity** is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 Table 12.6-1	E and F D, E, and F
5b.	**Discontinuity in Lateral Strength–Extreme Weak Story Irregularity** is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 12.3.3.2 Table 12.6-1	D, E, and F B and C D, E, and F

3.2 Torsional Effects

3.2.1 Torsional according EC8

Torsional Effects: Whenever a spatial model is used for the analysis, the accidental torsional effects may be determined as the envelope of the effects resulting from the application of static loadings, consisting of sets of torsional moments M_{ai} about the vertical axis of each storey i. Mai shall be defined as the following Eqn. 3.1

$$M_{ai}=e_{ai}.F_i \tag{3.1}$$

M_{ai} : The torsional moment applied at storey i about its vertical axis

e_{ai} : The accidental eccentricity of storey mass i

F_i : The horizontal force acting on storey I

The effects of loadings should be taken into account with positive and negative signs. Combination of the Effects of the Components of the Seismic Action: The horizontal components of the seismic action shall be taken as acting simultaneously. The structural response to each component shall be evaluated separately, using the combination rules for modal responses as previously explained in modal response spectrum analysis. Another method is that the square root of the sum of the squared values of the action effect due to each horizontal component. Beside to these, the action effects due to the combination of the horizontal components of the seismic action may be computed using both of the two following combination as an alternative method.

$E_{Edx}+0.30E_{Edy}$ (3.2)

$0.30E_{Edx}+E_{Edy}$ (3.3)

E_{Edx} : The action effects due to the application of the seismic action along the chosen horizontal axis x of the structure

E_{Edy} : The action effects due to the application of the seismic action along the chosen horizontal axis y of the structure

If the structural system or the regularity classification of the building in elevation is different in different horizontal directions, the value of the behaviour factor q may also be different. For the buildings satisfying the regularity in plan and in which walls or independent bracing systems in the two main horizontal directions are the only primary seismic elements, the seismic action may be assumed to act separately and without combination along the two main orthogonal horizontal axes of the structure (Eurocode 8, Design of Steel Structures for Earthquake Resistance, Part 1).

3.2.2 Torsional according ASCE7-5

3.2.2.1 Amplification of Accidental Torsional Moment.

Structures assigned to Seismic Design Category C, D, E, or F, where Type la or lb torsional irregularity exists as defined in Table 3.2 shall have the effects accounted for by multiplying M_{ta} at each level by a torsional amplification factor (A_x) as illustrated in Fig 3.2. and determined from the following equation (section 12.8.4.3 ASCE 7).

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \tag{3.4}$$

where:

δ_{max}= the maximum displacement at Level x (in. or mm) computed assuming A_x= 1

δ_{avg}= the average of the displacements at the extreme points of the structure at Level x computed assuming A_x= 1 (in. or mm)

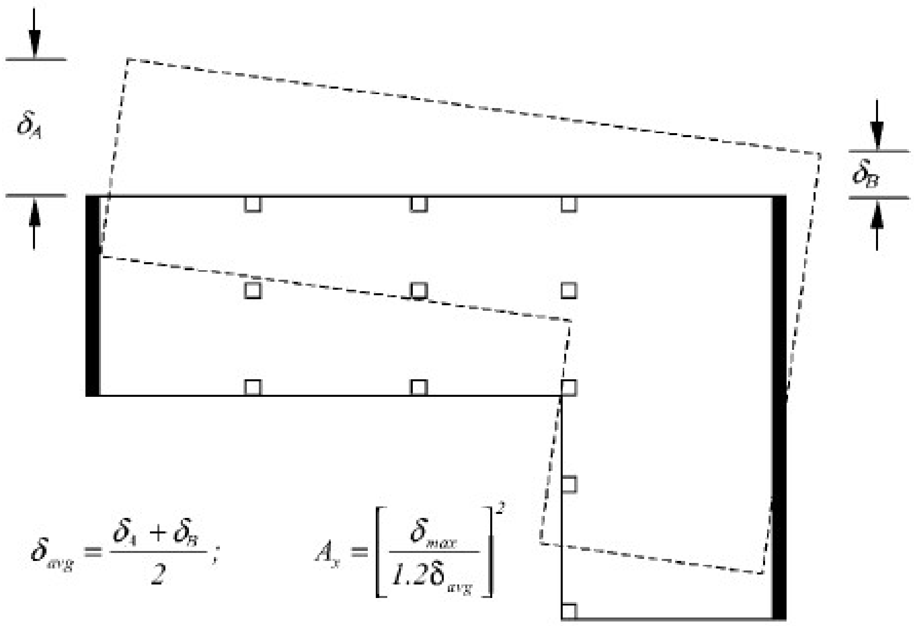

Figure 3.2 Torsional amplification factor, A_X

3.3 Limitation

3.3.1 Limitation according to EC8

3.3.1.1 Limitation of interstorey drift

(1) Unless otherwise specified, the following limits shall be observed:

a) for buildings having non-structural elements of brittle materials attached to the structure: $d_r\nu \leq 0{,}005h$ (3.5)

b) for buildings having ductile non-structural elements: $d_r\nu \leq 0{,}0075h$ (3.6)

c) for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

$$d_r\nu \leq 0{,}010\ h \tag{3.7}$$

where:

dr is the design interstorey drift

h is the storey height;

ν is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement.

(2) The value of the reduction factor ν may also depend on the importance class of the building. Implicit in its use is the assumption that the elastic response spectrum of the seismic action under which the "damage limitation requirement" should be met has the same shape as the elastic response spectrum of the design seismic action corresponding to the "ultimate limit state requirement" (Eurocode 8, Design of Steel Structures for Earthquake Resistance, Part 1).

3.3.2 Drift according ASCE 7-5

The design story drift (Δ) shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration. See Fig. 3.3(section 12.8.6 ASCE 7).

Where allowable stress design is used, (Δ) shall be computed using the strength level seismic forces specified in Section 12.8 without reduction for allowable stress design.

$$\delta_x = \frac{C_d\, \delta_{xe}}{I} \tag{3.8}$$

C_d = the deflection amplification factor in Table (3.2)

δ_{xe}= the deflections determined by an elastic analysis

I = the importance factor determined in accordance with Table (2.4)

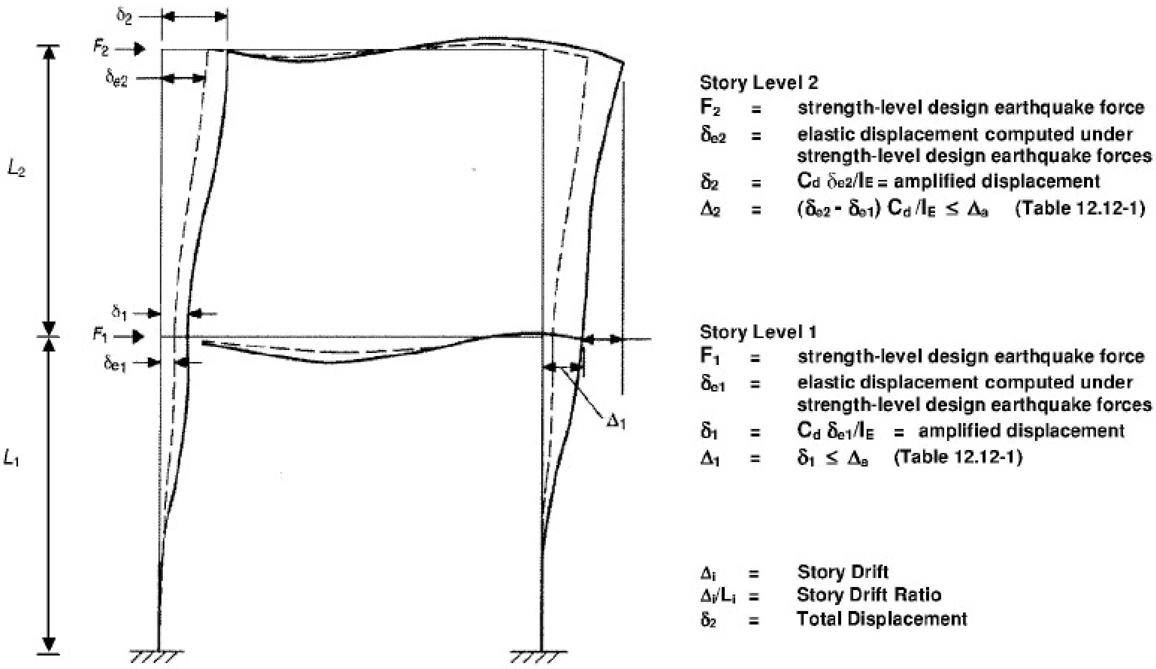

Figure 3.3 Story drift determination

3.4 Diaphragmatic

3.4.1 Diaphragmatic according to EC8

3.4.1.1 Diaphragmatic behavior at storey level

(1) In buildings, floors (including the roof) play a very important role in the overall seismic behavior of the structure. They act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action.

The action of floors as diaphragms is especially relevant in cases of complex and non-uniform layouts of the vertical structural systems, or where systems with different horizontal deformability characteristics are used together (e.g. in dual or mixed systems).

(2) Floor systems and the roof should be provided with in-plane stiffness and resistance and with effective connection to the vertical structural systems. Particular care should be taken in cases of non-compact or very elongated in-plan shapes and in cases of large floor openings, especially if the latter are located in the vicinity of the main vertical structural elements, thus hindering such effective connection between the vertical and horizontal structure.

(3) Diaphragms should have sufficient in-plane stiffness for the distribution of horizontal inertia forces to the vertical structural systems in accordance with the assumptions of the analysis (e.g. rigidity of the diaphragm, particularly when there are significant changes in stiffness or offsets of vertical elements above and below the diaphragm((Eurocode 8, DESIGN OF BUILDINGS ,SECTION 4.2.15).

3.4.2 Diaphragmatic according to ACSE7-5

3.4.2.1 Diaphragm Flexibility.

The structural analysis shall consider the relative stiff nesses of diaphragms and the vertical elements of the seismic force-resisting system. Unless a diaphragm can be idealized as either flexible or rigid, The structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semi rigid modeling assumption).

3.4.2.2 Flexible Diaphragm Condition.

Diaphragms constructed of untopped steel decking or wood structural panels are uermitted to be idealized as flexible in structures in which the vertical elements are steel or composite steel and concrete braced frames, or concrete, masonry, steel, or composite shear walls. Diaphragms of wood structural panels or untapped steel decks in one- and two-family residential buildings of light-frame con- struction shall also be permitted to be idealized as flexible.

3.4.2.3 Rigid Diaphragm Condition.

Diaphragms of concrete slabs or concrete filled metal deck with span-to-depth ratios of 3 or less in structures that have no horizontal irregularities are permitted to be idealized as rigid.

3.4.2.4 Calculated Flexible Diaphragm Condition.

Diaphragms not satisfying the conditions of Sections 12.3.1.1 or 12.3.1.2 (ACSE) are permitted to be idealized as flexible where the computed maximum in-plane deflection of the diaphragm under lateral load is more than two times the average story drift of adjoining vertical elements of the seismic force-resisting system of the associated story under equivalent tributary lateral load.

3.4.2.5 Irregular and Regular Classification.

Structures shall be classified as regular or irregular based upon the criteria in this section. Such classification shall be based on horizontal and vertical configurations.

3.4.2.6 Horizontal Irregularity.

Structures having one or more of the irregularity shall be desiglight- frame construction. nated as having horizontal structural irregularity. Such structures assigned to the seismic design categories shall comply with the requirements

3.4.2.7 Vertical Irregularity.

Structures having one or more of the irregularity types shall be designated as having vertical irregularity. Such structures assigned to the seismic design categories shall comply with the requirements (Figure 3.4)

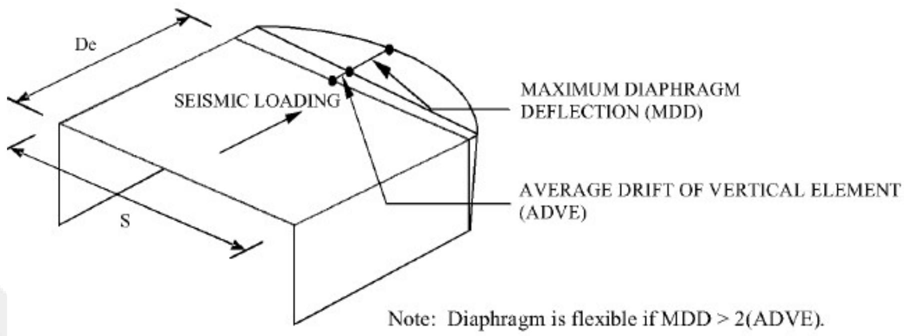

Figure 3.4 Flexible diaphragm

3.5.Non Structure

3.5.1.Non Structure according EC8

3.5.1.1. General

(1)P Non-structural elements (appendages) of buildings (e.g. parapets, gables, antennae, mechanical appendages and equipment, curtain walls, partitions, railings) that might, in case of failure, cause risks to persons or affect the main structure of the building or services of critical facilities, shall, together with their supports, be verified to resist the design seismic action.

(2)P For non-structural elements of great importance or of a particularly dangerous nature, the seismic analysis shall be based on a realistic model of the relevant structures and on the use of appropriate response spectra derived from the response of the supporting structural elements of the main seismic resisting system.

(3) In all other cases properly justified simplifications of this procedure

3.5.1.2 Verification

(1) The non-structural elements, as well as their connections and attachments or anchorages, shall be verified for the seismic design situation.

NOTE The local transmission of actions to the structure by the fastening of non-structural elements and their influence on the structural behaviour should be taken into account. The requirements for fastenings to concrete are given in EN1992-1-1:2004,

(2) The effects of the seismic action may be determined by applying to the nonstructural element a horizontal force *F*a which is defined as follows:

$$F_a=(S_a \cdot W_a \cdot \gamma_a)/q_a \qquad (3.9)$$

where:

F_a is the horizontal seismic force, acting at the centre of mass of the non-structural element in the most unfavourable direction;

W_a is the weight of the element;

S_a is the seismic coefficient applicable to non-structural elements, (see (3) of this subclause);

γ_a is the importance factor of the element,

q_a is the behaviour factor of the element

(3) The seismic coefficient *S*a may be calculated using the following expression:

$$Sa= \alpha.S.[3(1+\frac{z}{H}) / (1+(1-Ta/T1)-0.5] \qquad (3.10)$$

where

α is the ratio of the design ground acceleration on type A ground, *a*g, to the acceleration of gravity g;

S is the soil factor;

T_a is the fundamental vibration period of the non-structural element;

T_1 is the fundamental vibration period of the building in the relevant direction;

z is the height of the non-structural element above the level of application of the seismic action (foundation or top of a rigid basement); and

H is the building height measured from the foundation or from the top of a rigid basement.

3.5.2Non Structure According ACSE

3.5.2.1 Seismic Design Force.

The horizontal seismic design force (F_p) shall be applied at the component's center of gravity and distributed relative to the component's mass distribution and shall be determined in accordance with Eq. (3.11)

$$F_p = \frac{0.4 a_p S_{DS} W_p}{\left(\frac{R_p}{I_p}\right)} \left(1 + 2\frac{z}{h}\right) \tag{3.11}$$

F_p is not required to be taken as greater than

$$Fp = 1.6\, S_{DS}\, I_P\, W_P \tag{3.12}$$

and F_p shall not be taken as less than

$$Fp = 0.3\, S_{DS}\, I_P\, W_P \tag{3.13}$$

where

F_p = seismic design force

S_{Ds} = spectral acceleration, short period

a_p = component amplification factor that varies from 1.00 to 2.50

I_P = component importance factor that varies from 1 .OO to 1.50

W_P = component operating weight

R_P = component response modification factor that varies from 1 .OO to 12

z = height in structure of point of attachment of component with respect to the base. For items at or below the base, z shall be taken as 0. The value of z/h need not exceed 1.0

h = average roof height of structure with respect to the base

3.5.2.2 Seismic Relative Displacements.

The effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate. Seismic relative displacements (D_P) shall be determined in accordance with the equations set forth in (Sections 13.3.2.1 and 13.3.2.2 ACSE)

3.5.2.3Displacements within Structures.

For two connection points on the same Structure A or the same structural system, one at a height *h,* and the other at a height *h,,* D[, shall be determined

$$D_P = \delta_{XA} - \delta_{YA} \tag{3.14}$$

Alternatively, D_P is permitted to be determined using modal procedures described in (Section 12.9 ASCE),

using the difference in story deflections calculated for each mode and then combined using appropriate modal combination procedures.

D_P is not required to be taken as greater than

$$D_p = \frac{(h_x - h_y)\,\Delta_{aA}}{h_{sx}} \tag{3.15}$$

3.6.1 Design concepts EC8

3.6.1.1. Energy dissipation capacity and ductility classes

The design of earthquake resistant concrete buildings shall provide the structure with an adequate capacity to dissipate energy without substantial reduction of its overall resistance against horizontal and vertical loading. To this end, the requirements and criteria of Section 2 apply.

In the seismic design situation adequate resistance of all structural elements shall be provided, and non-linear deformation demands in critical regions should be commensurate with the overall ductility assumed in calculations.

Concrete buildings may alternatively be designed for low dissipation capacity and low ductility, by applying only the rules of EN 1992-1-1:2004 for the seismic design situation, and neglecting the specific provisions given in this section, provided the requirements set forth in 5.3 are met. For buildings which are not base-isolated (see Section 10), design with this alternative,

termed ductility class L (low), is recommended only in low seismicity cases (see 3.2.1(4)).

.Earthquake resistant concrete buildings other than those to which (2)P of this subclause applies, shall be designed to provide energy dissipation capacity and an overall ductile behaviour. Overall ductile behaviour is ensured if the ductility demand involves globally a large volume of the structure spread to different elements and locations of all its storeys. To this end ductile modes of failure (e.g. flexure) should precede brittle failure modes (e.g. shear) with sufficient reliability.

Concrete buildings designed in accordance with (3)P of this subclause, are classified in two ductility classes DCM (medium ductility) and DCH (high ductility), depending on their hysteretic dissipation capacity. Both classes correspond to buildings designed, dimensioned and detailed in accordance with specific earthquake resistant provisions, enabling the structure to develop stable mechanisms associated with large dissipation of hysteretic energy under repeated reversed loading, without suffering brittle failures.

To provide the appropriate amount of ductility in ductility classes M and H , specific provisions for all structural elements shall be satisfied in each class (see 5.4 - 5.6). In correspondence with the different available ductility in the two ductility classes, different values of the behaviour factor q are used for each class (see 5.2.2.2).

3.6.1.2 Structural types and behaviour factors

3.6.1.2.1 Structural types

(1) Concrete buildings shall be classified into one of the following structural types (see 5.1.2) according to their behaviour under horizontal seismic actions:

a) frame system;

b) dual system (frame or wall equivalent);

c) ductile wall system (coupled or uncoupled);

d) system of large lightly reinforced walls;

e) inverted pendulum system;

f) torsionally flexible system.

(2) Except for those classified as torsionally flexible systems, concrete buildings may be classified to one type of structural system in one horizontal direction and to another in the other.

(3) A wall system shall be classified as a system of large lightly reinforced walls if, in the horizontal direction of interest, it comprises at least two walls with a horizontal dimension of not less than 4,0 m or $2h_w/3$, whichever is less, which collectively support at least 20% of the total gravity load from above in the seismic design situation, and has a fundamental period T_1, for assumed fixity at the base against rotation, less than or equal to 0,5 s. It is sufficient to have only one wall meeting the above conditions in one of the two directions,

provided that: (a) the basic value of the behaviour factor, q_o, in that direction is divided by a factor of 1,5 over the value given in Table 5.1 and (b) that there are at least two walls meeting the above conditions in the orthogonal direction.

(4) The first four types of systems (i.e. frame, dual and wall systems of both types) shall possess a minimum torsional rigidity that satisfies expression (4.1b) in both horizontal directions.

(5) For frame or wall systems with vertical elements that are well distributed in plan, the requirement specified in (4)P of this subclause may be considered as being satisfied without analytical verification.

(6) Frame, dual or wall systems without a minimum torsional rigidity in accordance with (4)P of this subclause should be classified as torsionally flexible systems.

(7) If a structural system does not qualify as a system of large lightly reinforced walls according to (3)P above, then all of its walls should be designed and detailed as ductile walls.

3.6.2 Design Concept ASCE5-7

Design concept is an impressive term that we use to describe the intrinsic essentials of design. The concept encompasses reasons for our choice of design loads, analytical techniques, design procedures, preference for particular structural systems, and of course, our desire for economic optimization of the structure.

nature of loads and their effect on structural systems that paves the way for our understanding of structural behaviour and allows the designer to match structural systems to specific types of loading. For example, designers of tall buildings, recognizing the cost premium for carrying lateral loads by frame action alone, select a more appropriate system such as a belt and outrigger wall or a tubular system instead. And engineers designing for intense earthquakes know-ing that building structures must sustain gravity loads at large deformations, select moment frames and/or shear walls with ductile connections to provide for the deformation capacity.

As with other materials, the strength and deformation characteristics of reinforced concrete members are important in the design of buildings. In particular, buildings designed to resist seismic forces must have well-detailed members and joints such that the building can sustain large lateral deformations without losing its vertical load-carrying capacity.In reinforced concrete structures, the reinforcing bars and concrete are almost always subject to axial stress in tension or compression resulting from various load applications. However, they are usually stressed in a manner quite different from that in a simple axial compression or tensile test

CHAPTER 4

CASE STUDIES

4. Case Studies Using Both Codes

4.1 seismic analysis software (ETABS)

ETABS is a special purpose analysis and design program developed specifically for building systems. ETABS features a graphical interface coupled with modeling, analytical, and design procedures, all integrated using a common database. ETABS can handle the largest and most complex building models, including a wide range of nonlinear behaviors, making it an important tool for structural engineers in the building industry.

ETABS is a completely integrated system. Embedded beneath a simple, and fairly intuitive user interface are very powerful numerical methods, design procedures and international design codes, all working from a single comprehensive database.

This integration means that only one model of the floor systems and the vertical and lateral framing systems needs to be created to analyze and design the entire building. No external modules are required, because everything is integrated into one versatile analysis and design package with one Windows-based graphical user interface.

The effects on one part of the structure from changes in another part are instantaneous and automatic. The integrated components include a modeling module, a seismic and wind load generation module, a gravity load distribution module, a finite element based linear static and dynamic analysis module, and various design modules.

The ETABS building is idealized as a group of area, line and point objects. Those objects are used to represent wall, floor, column, beam, brace and link/spring physical members. The basic frame geometry is defined with reference to a simple three-dimensional grid system. With relatively simple modeling techniques, very complex framing situations may be considered.

The buildings may be unsymmetrical and non-rectangular in plan. Torsional behavior of the floors and interstory compatibility of the floors are accurately reflected in the results. Semi-rigid floor diaphragms may be modeled to capture the effects of inplane floor deformations.

The effects of the finite dimensions of the beams and columns on the stiffness of a frame system are included using end offsets that can be automatically calculated. The floors and walls can be modeled as membrane elements with in-plane stiffness only, plate bending elements with out-of-plane stiffness only or full shell-type elements, which combine both in-plane and out-of-plane stiffness. Floor and wall objects may have uniform load patterns in-lane or out-of-plane, and they may have temperature loads. The column, beam, brace, floor and wall objects are all compatible with one another.

Static analyses for user specified vertical and lateral floor or story loads are possible. If floor elements with plate bending capability are modeled, vertical uniform loads on the floor are transferred to the beams and columns through bending of the floor elements. Otherwise, vertical uniform loads on the floor are automatically converted to span loads on adjoining beams, or point loads on adjacent columns, thereby automating the tedious task of transferring floor tributary loads to the floor beams without explicit modeling of the secondary framing.

The program can automatically generate lateral wind and seismic load patterns to meet the requirements of various building codes. Three-dimensional mode shapes and frequencies, modal participation factors, direction factors and participating mass percentages are evaluated using eigenvector or Ritz-vector analysis. P-Delta effects can be included with static or dynamic analysis. Response spectrum analysis, linear time history analysis, nonlinear time history analysis, and static nonlinear (pushover) analysis are all possible.

Results from the various static load cases can be combined with each other or with the results from the dynamic response spectrum or time history analyses. Output can be viewed graphically, displayed in tabular output, or sent to a printer. Types of output include reactions and member forces, mode shapes and participation factors, static and dynamic story displacements and story shears, interstory drifts and joint displacements, and more. Import and export of data may occur between third-party applications such as Revit or AutoCAD from Autodesk (ETABS Reference).

4.2 Lateral Modeling Verifications

When the lateral system has been modeled using computer software like those described above, errors can easily occur in the process. For this reason, there arecertain verifications that must be made to ensure that the model is accurate and that the analysis will generate correct values for the imposed forces.

First of all the basic verifications must be performed to verify that the correct material properties, units, dimensions, boundary conditions and object assignments have been selected.

In lateral analysis, the mass of the structure is one of the most significant contributors to the design forces. The mass of the modeled structure should always be check by a hand calculation.

Each diaphragm should be checked to make sure that the mass is accounted for at each level. The assignment of the mass should be coordinated in the settings. When using superimposed loads instead of having the program calculated the self-weight, the settings must be adjusted to make sure that mass is not been accounted for twice.

Analysis of the model must always be run with static lateral loads in each direction. It is much simpler to verify the behavior of the model under static loads. It is important to verify that the applied lateral forces are accounted for and the load path makes sense. For example, if one wall seems to carry the entire load and the other wall carries none, then there must be something wrong.

Make sure that the calculated building period makes sense. The period should be within a reasonable range, and depending on the height of the building this could be between 2 and 10 seconds. If a tall building has a very short period, and a short building has a very long period, it is likely to be some errors in the modeling of the structure.

It is important to make sure that the mass participation factors are reasonable. If any modes have very little participation in any direction, there is probably a problem with the model. Boundary conditions, unconnected elements, unsupported elements, and null assignments must be checked. It is also important to make sure that enough modes are included in the analysis to achieve the code requirement of 90% mass participation.

Investigating the mode shapes can reveal serious problems with the model. It is important to make sure that the modes follow the general pattern for 1st order, 2nd order, etc. shapes. If a very symmetrical building has torsional modes as one of the primary modes, there is also reason for concern in regards to modeling errors.

Making sure that the base shear for dynamic analysis is scaled properly is also vital. According to ASCE 7-05, the dynamic base shear must be scaled up to 85% of static base shear, but it must not be scaled down if it exceeds this value.

It must be verified that the center of mass and center of rigidity appear to be in the correct locations. The use of cracked section properties must be verified if required. This can easily be accomplished by modifying the value for the Modulus of Elasticity, E, or by applying stiffness modification factors to the frame and shell elements.

Verification of proper assignment of diaphragms must be conducted. Any components that are not physically connected to the diaphragm should not be assigned to the diaphragm. It is also important to assign the proper rigidity to the diaphragm, considering aspect ratios, openings etc. A diaphragm is set to act either as rigid, or semi-rigid.

When using concrete coupling beams, it is important to properly account for stiffness degradation, i.e. cracking. It is typically more degraded than the assumed 50% of gross properties.

4.3 Practical Applications of the Codes

A concrete residential tower with different code has been selected for this section. The building has been using the most comma codes namely ,IBC 2006 and EC8.

4.3.1 Project description

4.3.1.1 Building description:

20 stories building (irregular shape)

- 1-12 stories with 1750 m^2 area
- 13-20 stories with 1225 m^2 area

2. Seismic lateral force resisting system moment shear wall resisting 75% of lateral force + frame resisting 25% of lateral force .(see fig 4.1)

3.Seismic design criteria per IBC 2006 and ASCE7-5:

- Importance Factor, $I_E = 1$
- Site Class: D
- Seismic Design Category: D
- Design Parameters:
- $S_S = 0.32$
- $S_1 = 0.22$
- $F_a = 1.544$
- $F_v = 1.96$
- $S_{DS} = 0.3294$
- $S_{D1} = 0.2875$
- Response Modification Factor
- $R_x = 6.5$
- $R_y = 6.5$
- Tx=1.4 sec ,Ty=1.4 sec

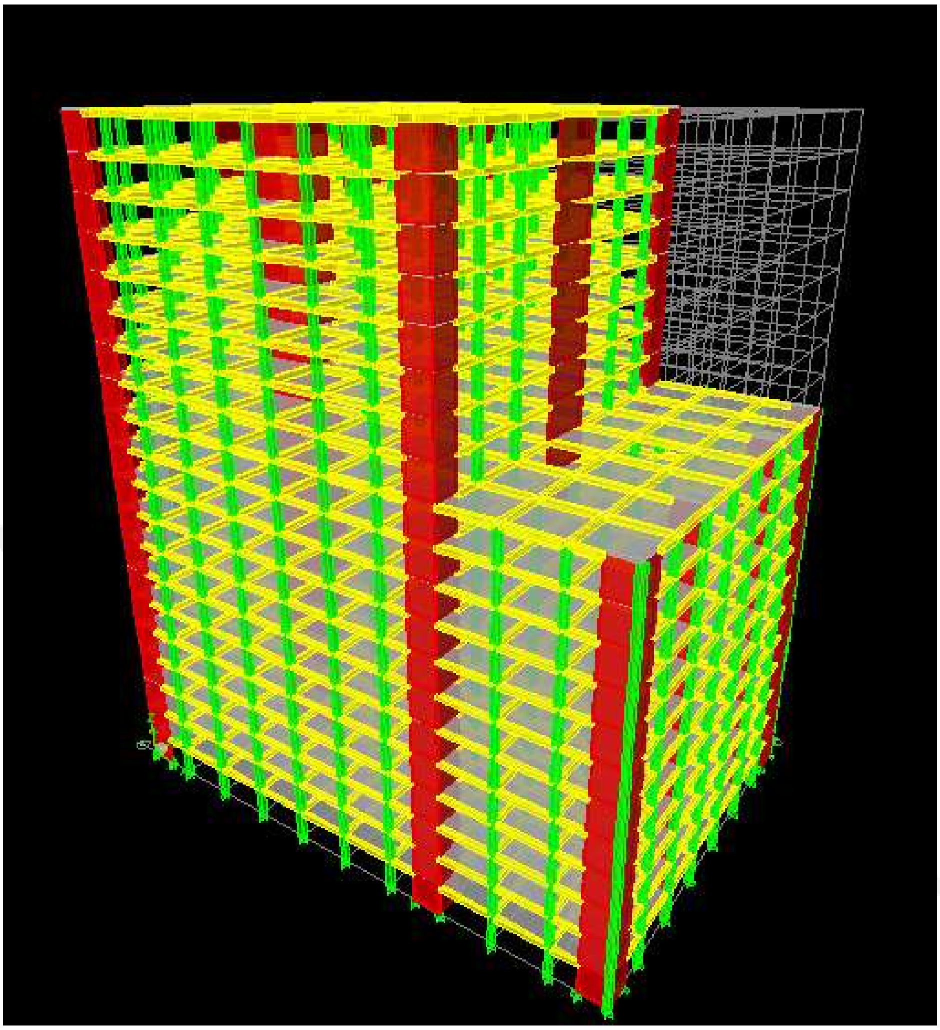

Figure 4.1 Structure modeled

4.Seismic Design Criteria per Eurocode 8:

- Importance Factor, γI = 1.0
- Ground Type: D
- Design Ground Acceleration, ag= 0.15
- Tx=1.4 sec
- Ty=1.4 sec

The seismic restraint system was established in around the center of the tower to reduce tensional irregularities. This means that the centers of mass and rigidity must be within close proximity .

The center of rigidity to not fall close to the center of mass. As a result of these modifications, the structure was categorized as a torsionally irregular building. it had been deemed torsionally irregular, additional provisions would have to be met.

We have shape irregular That because of the irregular area floors where floor space of 1 to 12 (1750 m^2)and the area of the upper floors(1225 m^2) .

This irregularity gives extra moment torsion to Origin, Is extracted values (AX) Amplification of Accidental Tensional Moment in order to see torsion within the allowable limits.

Also we make check about Dimensional vertical section according to ASCE7-5 table 3.3 and EC8 section 3.1.1.2 figure 3.1 .

Check torsion moment according ASCE7-5 and EC 8 :

$$Ax=\left(\frac{\delta_{max}}{1.2\,\delta_{avg}}\right)^2 \qquad (4.1)$$

$\delta_{max=}$the maximum displacement at Level x (in. or mm) computed assuming A, = 1

$\delta_{avg=}$the average of the displacements at the extreme points of the structure at Level x computed assuming A = 1 (in. or mm)

A_x minimum =1A_x maximum=3

Story level	Load	Direction	$Ax=(\frac{\delta_{max}}{1.2\ \delta_{avg}})^2$	$1 \leq Ax$
20	AX	X	0.811501	CHECK OK
19	AX	X	0.81	CHECK OK
18	AX	X	0.81	CHECK OK
17	AX	X	0.808501	CHECK OK
16	AX	X	0.807003	CHECK OK
15	AX	X	0.807003	CHECK OK
14	AX	X	0.805506	CHECK OK
13	AX	X	0.804011	CHECK OK
12	AX	X	0.802517	CHECK OK
11	AX	X	0.802517	CHECK OK
10	AX	X	0.801025	CHECK OK
9	AX	X	0.801025	CHECK OK
8	AX	X	0.801025	CHECK OK
7	AX	X	0.799534	CHECK OK
6	AX	X	0.799534	CHECK OK
5	AX	X	0.798044	CHECK OK
4	AX	X	0.798044	CHECK OK
3	AX	X	0.796556	CHECK OK
2	AX	X	0.7921	CHECK OK
1	AX	X	0.795069	CHECK OK

Table 4.1 Ax according ASCE7-5

Table 4.2 Ax according EC8

Story level	Load	Direction	$Ax=(\frac{\delta_{max}}{1.2\ \delta_{avg}})^2$	$1 \leq Ax$
20	AX	X	0.81	CHECK OK
19	AX	X	0.81	CHECK OK
18	AX	X	0.808501	CHECK OK
17	AX	X	0.808501	CHECK OK
16	AX	X	0.807003	CHECK OK
15	AX	X	0.805506	CHECK OK
14	AX	X	0.805506	CHECK OK
13	AX	X	0.804011	CHECK OK
12	AX	X	0.801025	CHECK OK
11	AX	X	0.802517	CHECK OK
10	AX	X	0.801025	CHECK OK
9	AX	X	0.7921	CHECK OK
8	AX	X	0.799534	CHECK OK
7	AX	X	0.799534	CHECK OK
6	AX	X	0.799534	CHECK OK
5	AX	X	0.798044	CHECK OK
4	AX	X	0.798044	CHECK OK
3	AX	X	0.796556	CHECK OK
2	AX	X	0.795069	CHECK OK

1	AX	X	0.802517	CHECK OK

Story level	Load	Direction	Drift	$\Delta=(\delta_{e1}-\delta_{e2})\,C_d/I_E$	$\leq 0.02h$

Check drift according ASCE7-5 and EC 8 :

- check drift according ASCE7-5

$\Delta=(\delta_{e1}-\delta_{e2})\,C_d/I_E$

δ_{xe}= elastic displacement computed under strength-level design earthquake forces

C_d=the deflection amplification factor

I=the importance factor (see figure 4.2)

- check drift according EC8

$d_r\nu \leq 0.010\ h$

where

*d*r is the design interstorey drift

h is the storey height;

ν is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement see figure (4.2),(4.3),(4.4),(4.5).

20	AX	X	1.425	7.125	CHECK OK
19	AX	X	1.53	7.65	CHECK OK
18	AX	X	[illegible]	[illegible]	CHECK OK
17	AX	X	1.77	8.85	CHECK OK
16	AX	X	1.893	9.465	CHECK OK
15	AX	X	2.001	10.005	CHECK OK
14	AX	X	2.088	10.44	CHECK OK
13	AX	X	2.133	10.665	CHECK OK
12	AX	X	2.1	10.5	CHECK OK
11	AX	X	2.133	10.665	CHECK OK
10	AX	X	2.157	10.785	CHECK OK
9	AX	X	2.163	10.815	CHECK OK
8	AX	X	2.148	10.74	CHECK OK
7	AX	X	2.1	10.5	CHECK OK
6	AX	X	2.016	10.08	CHECK OK
5	AX	X	1.887	9.435	CHECK OK
4	AX	X	1.701	8.505	CHECK OK
3	AX	X	1.44	7.2	CHECK OK
2	AX	X	1.074	5.37	CHECK OK
1	AX	X	0.516	2.58	CHECK OK

Table 4.3 Drift according ASCE7-5

Story level	Load			Drift .V	0.01h	≤ 0.02h

Table 4.4 Drift according EC8

		Direction	Drift	mm	mm	
20	AX	X	1.254	2.508	30	CHECK OK
19	AX	X	1.347	2.694	30	CHECK OK
18	AX	X	1.449	2.898	30	CHECK OK
17	AX	X	1.563	3.126	30	CHECK OK
16	AX	X	1.677	3.354	30	CHECK OK
15	AX	X	1.785	3.57	30	CHECK OK
14	AX	X	1.875	3.75	30	CHECK OK
13	AX	X	1.929	3.858	30	CHECK OK
12	AX	X	1.92	3.84	30	CHECK OK
11	AX	X	1.968	3.936	30	CHECK OK
10	AX	X	2.007	4.014	30	CHECK OK
9	AX	X	2.031	4.062	30	CHECK OK
8	AX	X	2.034	4.068	30	CHECK OK
7	AX	X	2.007	4.014	30	CHECK OK
6	AX	X	1.944	3.888	30	CHECK OK
5	AX	X	1.833	3.666	30	CHECK OK
4	AX	X	1.665	3.33	30	CHECK OK
3	AX	X	1.422	2.844	30	CHECK OK
2	AX	X	1.071	2.142	30	CHECK OK
1	AX	X	0.519	1.038	30	CHECK OK

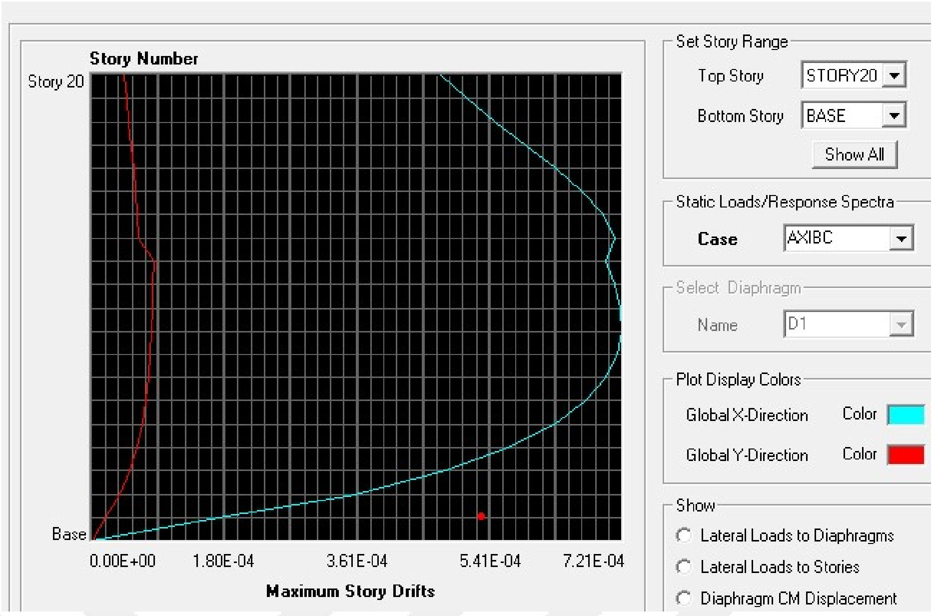

Figure 4.2 Drift in story according to ASCE7-5

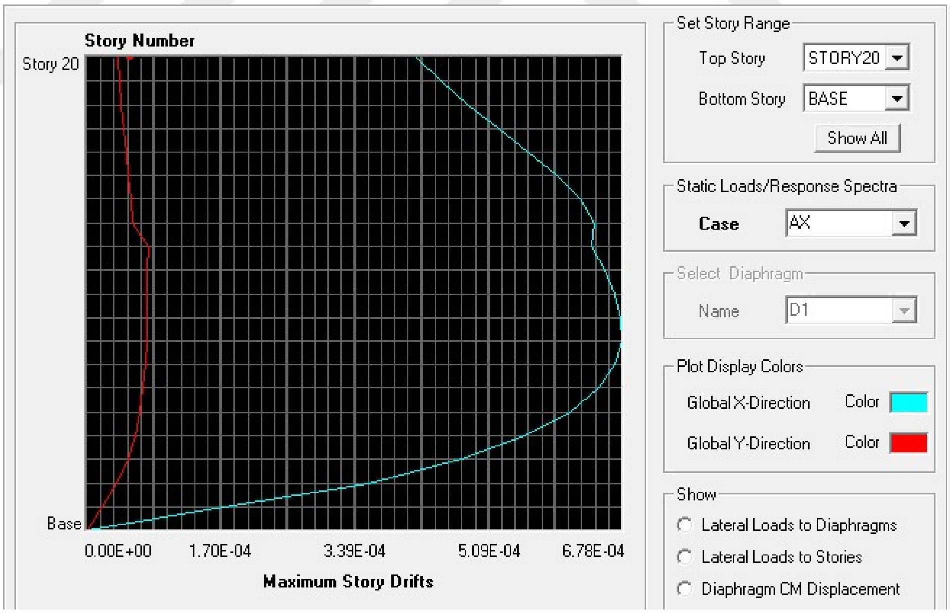

Figure 4.3 Drift in story according to EC8

The seismic base was set at ground level, and the below grade levels where simplified since they would not be considered in the seismic analysis.

When all structural members had been modeled, including columns, beams, shear wall,and slabs, the program ran a check to make sure that the model was functioning properly without any discrepancies. The first time this check was run, it displayed some components that were malfunctioning as a result of modeling errors.

These problems were repaired so that the model would perform as desired. Part of the modeling process is also applying loads. The program calculates the selfweight of the modeled components, but other dead loads can also occur, such as the weight of partition walls, and flooring.

The Live loads were also added to each floor depending on the type of occupancy. This building primarily consists of office space, and a live load of 0.4 ton/m was applied to all levels. Seismic loads is lateral loads and is therefore applied in the Frame analysis module.

4.3.2.1 Modeling

When modeling this building the analysis software ETABS was used. Since this analysis was restricted to seismic behavior of the building, only the contributing components were modeled. This is a building with a dual system, i.e. a seismic restraint system consisting of both concrete shear walls and moment resisting frames.

When using such a system, the 2006 IBC requires that the frames alone must be able to withstand 25% of the seismic load. This building was therefore first modeled with moment frames only, for use in the frames analysis.

Later the model was modified to also include the shear walls, which is the main resisting system. Because of the irregular shape of the diaphragms of the tower, the architectural plans were imported to the model.

This gave an accurate floor plan to the various levels without the excessive work required to duplicate the outline of the floor slabs. Even with the irregular shape of the typical floor, the centers of mass and rigidity not corresponded well. For the levels near the base, the extent of the podium level diaphragms lead to an additional shear wall being required throughout the podium levels (see Fig. 4.1). The seismic base was set at ground level for this building as well.

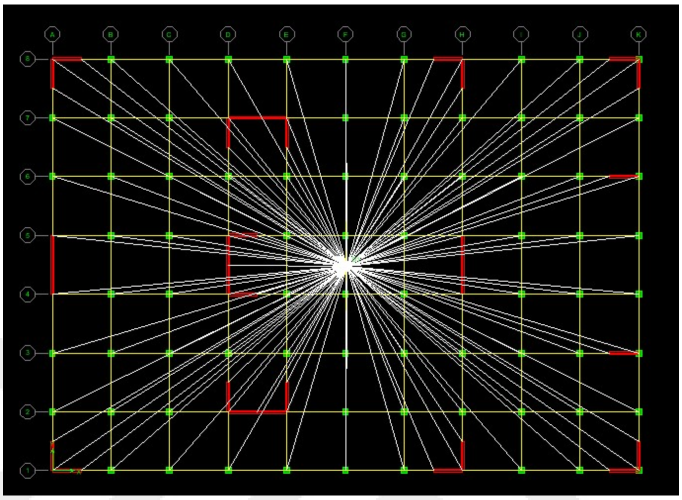

Figure 4.4 Center of mass in level floor (1 to 12)

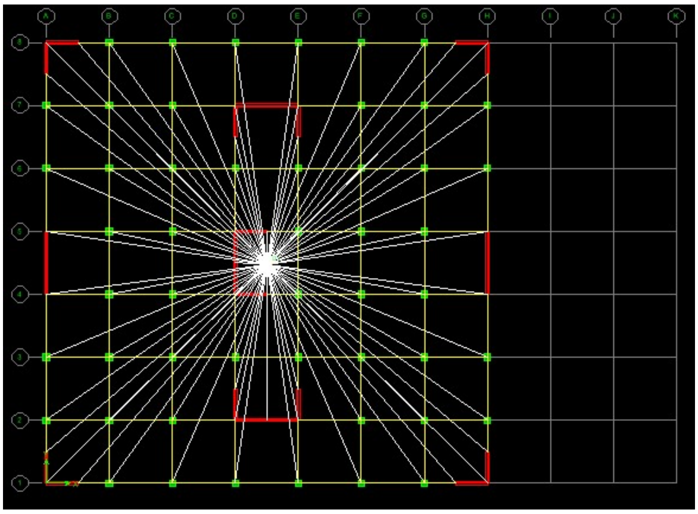

Figure 4.5 Center of mass in level floor (13 to 20)

4.3.2.2 Analysis

The first thing to do in this module was to define the load cases. This was done by selecting the type of load, and what code to use in the calculation of the load on the structure. For example, when choosing the seismic load case, the option of several different codes were given. For this case study, both the 2006 IBC and the Eurocode were used.

4.3.2.3 Results

Force in basic shear	ASCE7-5	EC8
FX	-614.11	-626.66
MZ	11864.929	12108.669
MY	-25631.47	-24298.101

Table 4.5 The force in the Basic shear(ton.m)

Table 4.6 Calculate the center of mass and center of rigidity

Story level	XCM	YCM	XCR	YCR
20	17.422	17.568	20.004	17.538
19	17.417	17.56	20.306	17.536
18	17.417	17.56	20.611	17.533
17	17.417	17.56	20.925	17.531
16	17.417	17.56	21.25	17.528
15	17.417	17.56	21.58	17.526
14	17.417	17.56	21.896	17.523
13	17.417	17.56	22.17	17.52
12	24.123	17.589	22.357	17.517
11	24.789	17.586	22.455	17.515
10	24.789	17.586	22.48	17.511
9	24.789	17.586	22.455	17.508
8	24.789	17.586	22.393	17.504
7	24.789	17.586	22.3	17.5
6	24.789	17.586	22.18	17.495
5	24.789	17.586	22.038	17.489
4	24.789	17.586	21.885	17.483
3	24.789	17.586	21.745	17.475
2	24.789	17.586	21.67	17.469

1	24.789	17.586	21.73	17.48

Table 4.7 Calculate the displacement according to ASCE5-7 and EC8

Story level	ASCE7-5(mm)	EC8 (mm)
20	35.9289	33.2928
19	34.5031	32.0379
18	32.9726	30.6920
17	31.3259	29.2427
16	29.5545	27.6797
15	27.6617	26.0023
14	25.6598	24.2185
13	23.5707	22.3444
12	21.4364	20.4144
11	19.3361	18.4948
10	17.2017	16.5279
9	15.0440	14.5217
8	12.8798	12.4910
7	10.7327	10.4581
6	8.6329	8.4519
5	6.6172	6.5091
4	4.7310	0.5183
3	3.0310	4.6757
2	0.5163	1.5884
1	1.5916	3.0100

Table 4.8 Force in the story (ton) according ASCE7-5 and EC8

Story level	ASCE5-7	EC8
20	57.18	46.61
19	60.03	50.1
18	55.47	47.47
17	51.02	44.83
16	46.70	42.2
15	42.49	39.55
14	38.42	36.92
13	34.47	34.28
12	41.38	42.7
11	37.55	40.33
10	32.67	36.67
9	28.00	33
8	23.57	29.33
7	19.39	25.67
6	15.48	22
5	11.86	18.33
4	8.56	14.67
3	.62	11
2	5	7.33
1	1.13	3.67

Table 4.9 Calculate moment in column (ton.m) according ASCE7-5 and EC8

Story level	ASCE7-5	EC8
20	0.945	0.856
19	0.843	0.763
18	0.853	0.772
17	0.844	0.764
16	0.836	0.757
15	0.825	0.749
14	0.821	0.745
13	0.804	0.731
12	0.746	0.682
11	0.734	0.671
10	0.702	0.643
9	0.659	0.606
8	0.609	0.561
7	0.551	0.509
6	0.487	0.45
5	0.417	0.387
4	0.348	0.326
3	0.281	0.265
2	0.289	0.282
1	0.232	0.232

Table 4.10 Calculate shear column(ton) according to ASCE7-5 and EC8

Story level	ASCE7-5	EC8
20	0.69	0.63
19	0.56	0.51
18	0.58	0.53
17	0.57	0.52
16	0.57	0.51
15	0.56	0.51
14	0.55	0.5
13	0.54	0.49
12	0.5	0.46
11	0.49	0.45
10	0.47	0.44
9	0.44	0.41
8	0.41	0.38
7	0.37	0.34
6	0.32	0.3
5	0.27	0.25
4	0.22	0.21
3	0.17	0.16
2	0.15	0.15
1	0.12	0.12

Table 4.11 Calculate moment story (ton.m) according ASCE5-7 and EC8

Story level	ASCE5-7	EC8
20	-171.541	-139.827
19	-523.174	-429.969
18	-1041.22	-862.514
17	-1712.33	-1429.55
16	-2523.53	-2123.17
15	-3462.21	-2935.45
14	-4516.14	-3858.5
13	-5673.5	-4884.39
12	-6954.98	-6038.37
11	-8349.11	-7313.34
10	-9841.24	-8698.32
9	-11417.4	-10182.3
8	-13064.2	-11754.3
7	-14769.3	-13403.2
6	-16520.8	-15118.2
5	-18307.8	-16888.2
4	-20120.6	-18702.2
3	-21950.2	-20549.2
2	-23789.1	-22418.1
1	-25631.5	-24298.1

Table 4.12 Calculate story torsion (ton.m) according to ASCE5-7 and EC8

Story level	ASCE5-7	EC8
20	1104.598	900.385
19	2263.783	1867.904
18	3334.889	2784.5
17	4320.149	3650.175
16	5221.861	4464.927
15	6042.404	5228.758
14	6784.238	5941.667
13	7449.919	6603.653
12	8250.084	7429.329
11	8976.124	8209.219
10	9607.751	8918.21
9	10149.23	9556.301
8	10605.08	10123.49
7	10980.1	10619.79
6	11279.47	11045.18
5	11508.8	11399.68
4	11674.32	11683.27
3	11783.01	11683.27

2	11843.11	11895.97
1	11864.93	11895.97

Table 4.13 Calculate story shear (ton) according to ASCE5-7 and EC8

Story level	ASCE7-5	EC8
20	-57.18	-46.61
19	-117.21	-96.71
18	-172.68	-144.18
17	-223.7	-189.01
16	-270.4	-231.21
15	-312.89	-270.76
14	-351.31	-307.68
13	-385.78	-341.96
12	-427.16	-384.66
11	-464.71	-424.99
10	-497.37	-461.66
9	-525.38	-494.66
8	-548.95	-523.99
7	-568.35	-549.66
6	-583.83	-571.66
5	-595.69	-589.99
4	-604.25	-604.66
3	-609.87	-604.66
2	-612.98	-615.66
1	-614.11	-615.66

Table 4.14 modal mass participation

The modal direction factor in global X direction	The modal direction factor in global y-direction	SUM The modal direction factor in global X-direction	SUM The modal direction factor in global y-direction
0	63.7149	0	63.7149
69.757	0	69.757	63.715
0.0013	6.9664	69.7583	70.6814
0.036	12.0803	69.7943	82.7617
14.4895	0.0344	84.2837	82.7962
0.0013	1.4876	84.285	84.2837
0.0171	5.8436	84.3022	90.1273
6.1662	0.0175	90.4684	90.1448
0.0008	0.1567	90.4691	90.3015
2.9034	0.0382	93.3726	90.3397
0.0402	2.8247	93.4127	93.1643

4.3.2.4 Conclusion

In this study the analysis of a high building reinforced concrete according to ASC57-5 and EC8 using program computer ETAB under the influence earthquakes. Although the two codes have certain differences, it is clear that they are both based on a common understanding of earthquake behavior.

1. From tables 4.5 we can see the shear force in foundation calculated by using EC8 is more than ASCE7-5 where EC8 give -626.66 ton and ASCE7-5 give -614.11 ton .

2.From tables 4.5 we can see the torsion in foundation calculated by using EC8 is more than ASCE7-5 where EC8 give 12108.669 ton/m and ASCE7-5 give 11864.929 ton/m.

3.From tables 4.5 we can see the moment in foundation calculated by using ASCE7-5 is more than EC8 where ASCE7-5 give -25631.47 ton/m and EC8 give -24298.101 ton/m.

4.From tables (4.9) ,(4.10) we can obtained the distribution of shear forces and moment in column is irregular, that happen because of the non-applicability center of rigidity with center of mass, not irregular shape of origin from and the force is not applied in center of mass. And in table (4.6) we see the center of mass between floor 12 and 13 is different , this different produce extra torsion and make the distribution of moments and shear forces in column irregular.

5.From tables 4.7 we can see the displacement calculated by using ASEE7-5 is more than EC8.

6.Finally from table (4.14) we can see the combined model mass participation of direction-X is 93.4127 % of the actual mass for two codes and the combined model mass participation of direction-Y is 93.1643 % of the actual mass for two codes. the codes limit the above value to 90 %.

4.3.2.5 Future work

My recommendation are to study effect of different between center of mass and center of rigidity on distribution of moment and shear force of the structure .

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