MOSTAFA K. A. HAMED

COMPARATIVE STUDY OF DIFFERENT SEISMIC CODES FOR REINFORCED CONCRETE BUILDINGS IN NORTHERN CYPRUS

NEU 2018

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A THESIS SUBMITTED TO THE GRADUATE SCHOOL OF APPLIED SCIENCES OF NEAR EAST UNIVERSITY

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In Partial Fulfillment of the Requirements for The Degree of Master of Science in Civil Engineering

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Mostafa K. A. Hamed: **Comparative Study of Different Seismic Codes for Reinforced Concrete Buildings in Northern Cyprus**

Approval of Director of Graduate School of Applied Sciences

Prof. Dr. Nadire ÇAVUŞ

We certify that, this thesis is satisfactory for the award of the degree of Master of Science in Civil Engineering

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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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To my parents

ABSTRACT

This study presents a comparative evaluation between three seismic design codes, the International Building code (IBC 2009) and Eurocode 8 (EC 8) which are well known and the seismic design code for northern Cyprus which was established in 2015. In order to make possible comparison among the codes, a particular location and the most common residential frame model has been chosen. In this research, a building of moment-resisting frame and moment-resisting frame with shear wall plan of reinforced concrete (RC) frames were analysed for low-rise to mid-rise structures. Response spectrum method (RSM) and equivalent lateral force method (ELFM) were performed using extended three dimensional analysis of building system (ETABS) software package. The main objective of this study is to examine the seismic provisions of the first edition of the northern Cyprus seismic code to determine whether it provides a generic level of safety that incorporate in well established code. The results obtained from both static and dynamic analysis are presented in the form of base shear, story shear, displacement, axial forces and bending moments for selected columns for three different codes.

Keywords: Seismic design code; equivalent lateral force method; response spectrum method; moment-resisting frame; moment-resisting frame with shear wall; north Cyprus

ÖZET

Bu çalışmada, üç farklı deprem yönetmeliği için karşılaştırmalı değerlendirmeler yapılmıştır. Kuzey Kıbrıs'ta 2015 yılında hazırlanmış deprem bölgelerinde yapılacak binalar hakkındaki yönetmelik, iyi bilinen ve yaygın olarak kullanılan IBC2009 ve EC 8 yönetmelikleri ile karşılaştırılmış ve değerlendirmeler yapılmıştır. Yönetmelikler arasında olası karşılaştırmaların yapılabilmesi için, belirli bir yer ve en yaygın konut çerçeve modeli seçilmiştir. Bu araştırmada, az ve orta yükseklikteki yapılar için, moment dayanımlı çerçeveve perde duvarlı moment dayanımlı betonarme çerçevelerin yapısal analizleri yapılmıştır.Bunun için ETABS yazılım paketi yardımı ile , tepki spektrumu yöntemi ve eşdeğer yanal kuvvet yöntemi kullanılarak üç boyutlu analiz gerçekleştirilmiştir.Bu çalışmanın temel amacı kuzey Kıbrıs'ta kullanılmaya başlanan sismik tasarım yönetmeliğinin ilk baskısının sismik hükümlerini inceleyip, iyi hazırlanmış yönetmeliklerin dahil edildiği kapsamlı bir güvenlik seviyesi sağlayıp sağlamadığını tespit etmektir. Üç farklı yönetmelik için statik ve dinamik analizden elde edilen sonuçlar, taban kesme kuvveti, kat kesme kuvveti, yerdeğiştirme, ve bazı seçilmiş kolonlarda,eksenel kuvvetler ve eğilme momentleri şeklinde sunulmuştur.

Anahtar Kelimeler: Sismik tasarım yönetmeliği; eşdeğer yanal kuvvet yöntemi; tepki spektrum yöntemi; momente dayanımlı çerçeve; perde duvarlı moment dayanımlı çerçeve; Kuzey Kıbrıs

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LIST OF ABBREVIATIONS

ABSSUM	Absolute Sum
CQC	Complete Quadratic Combination
DCH	Higher Ductility Classes
DCM	Medium Ductility Classes
EC 8	Eurocode 8
ELFM	Equivalent Lateral Force Method
ETABS	Extended Three Dimensional Analysis of Building System
HDL	High Ductility Level
IBC 2009	International Building Code 2009
MCE	Maximum Considered Earthquake
MRF	Moment Resisting Frame
MRF+SW	Moment Resisting Frame With Shear Wall
NCSC 2015	Northern Cyprus Seismic Code 2015
NDL	Nominal Ductility Level
PGA	Peak Ground Acceleration
RC	Reinforced Concrete
RSM	Response Spectrum Method
SDC	Seismic Design Category
SDOF	Single Degree of Freedom
SRSS	Square Root of the Sum of Squares

LIST OF SYMBOLS

DL	Dead load
LL	Live load
E	Earthquake load
S_{MS}	Response accelerations for short periods
S_{M1}	Response accelerations for long periods
S_{DS}	Design spectral acceleration for short periods
S_{D1}	Design spectral acceleration for long periods
V	Design seismic base shear
W	Effective weight
C_S	Seismic response coefficient
I	Importance factors
R	Response modification coefficients
F_x	Design lateral force applied at story *x*
w_x or w_i	Portion of the total effective weight of the structure, W, assigned to level *x* or *i*, respectively
k	Exponent related to the structure period
V_{S30}	Shear wave velocity
S_S	Measure of how strongly the MCE affects structures with a short period 0.2 sec
S_1	Measure of how strongly the MCE affects structures with a longer period 1 sec
F_a	Corresponding site coefficients for short periods
F_v	Corresponding site coefficients for long periods
T_a	First natural vibration period of the building
N	Number of stories
T	Fundamental period
T_L	Long-period transition period
P_{NCR}	Reference probability of exceedance in 50 years of the reference seismic action for the no-collapse requirement
$S_e(T)$	Elastic response spectrum

$\mathbf{a_g}$	Design ground acceleration
$\mathbf{a_gR}$	Reference peak ground acceleration on type A ground
$\boldsymbol{T_B}$	Lower limit of the period of the constant spectral acceleration branch
$\boldsymbol{T_C}$	Upper limit of the period of the constant spectral acceleration branch
$\boldsymbol{T_D}$	Value defining the beginning of the constant displacement response range of the spectrum
$\boldsymbol{S}$	Soil factor
$\boldsymbol{\eta}$	Damping correction factor
Ms	surface wave magnitude
$\boldsymbol{\xi}$	Viscous damping ratio of the structure expressed as a percentage
$\boldsymbol{S_{De}(T)}$	Elastic displacement response spectrum
$\boldsymbol{q}$	Behaviour factor
$\boldsymbol{S_d(T)}$	Design spectrum
$\boldsymbol{\beta}$	Lower bound factor for the horizontal design spectrum
$\boldsymbol{F_b}$	Design seismic base shear
$\boldsymbol{S_d(T_1)}$	Design spectrum at period T_1
$\boldsymbol{T_1}$	Fundamental period of vibration
$\boldsymbol{m}$	Total mass of the building
$\boldsymbol{\lambda}$	The correction factor
$\boldsymbol{F_i}$	Horizontal force acting on story *i*
$\boldsymbol{S_i, S_j}$	Displacements of masses m_i, m_j in the fundamental mode shape
$\boldsymbol{m_i, m_j}$	Story masses
$\boldsymbol{Z_i, Z_j}$	Height of the masses m_i, m_j above the level of application of the seismic action (foundation or top of a rigid basement)
$\boldsymbol{H}$	Building height
$\boldsymbol{A(T)}$	Spectral acceleration coefficient
$\boldsymbol{A_0}$	Effective ground acceleration coefficient
$\boldsymbol{S(T)}$	Spectrum coefficient
$\boldsymbol{S_{ae}(T)}$	Elastic spectral acceleration
$\boldsymbol{T_A, T_B}$	Spectrum characteristic periods

$\boldsymbol{R_{\mathrm{a}}(T)}$	Seismic load reduction factor
R	Behaviour factor
$\boldsymbol{F_i}$	Equivalent seismic load shall be distributed to stories of the building
$\boldsymbol{\Delta F_N}$	Additional equivalent seismic load
$\boldsymbol{V_t}$	Base shear
$\boldsymbol{w_i, w_j}$	Story weights
$\boldsymbol{H_i, H_j}$	Height of building
$\boldsymbol{DL_i}$	Total dead load at *i*'th story of building
$\boldsymbol{LL_i}$	Total live load at *i*'th story of building
$\boldsymbol{n}$	Live load participation factor
$\boldsymbol{H_N}$	Total height of building measured from the top foundation level
Sf	Scale factor
$\boldsymbol{\mathrm{Sf}_N}$	New scale factor
g	Acceleration due to gravity

CHAPTER 1

INTRODUCTION

1.1 Background

Earthquake is one of the most destructive natural hazard. Earthquakes do not destroy the settlement area only. It may be de-stabilize the economy and social structure of the economy. Earthquakes occur several times a day in various parts of the world. Major earthquakes occur most frequently in particular areas of the earth's surface that are called zones of high probability. The global seismic hazard map shown in Figure 1.1 which is based on data from the Global seismic hazard assessment program, highlights the areas where there is an increased risk of seismic activity. In the countries, which are placed on the major earthquake zone of the world, designing and constructing earthquake resistance structures is of great importance. Highly destructive earthquakes hit around the world resulting in injuries and deaths of humans and left a lot of constructions with extensive damage. The main reason of substantial damage is due to the weakness of buildings to withstand with earthquake effects due to the insufficient detailing of the seismic resisting building according to inadequate detailing. Therefore, to improve the safety of the constructions, numerous of seismic codes were provided worldwide(Ozcebe et al., 2004). All over the world countries placed on earthquake zones, publish their own codes to improve the safety and to control the design and construction of the structures.

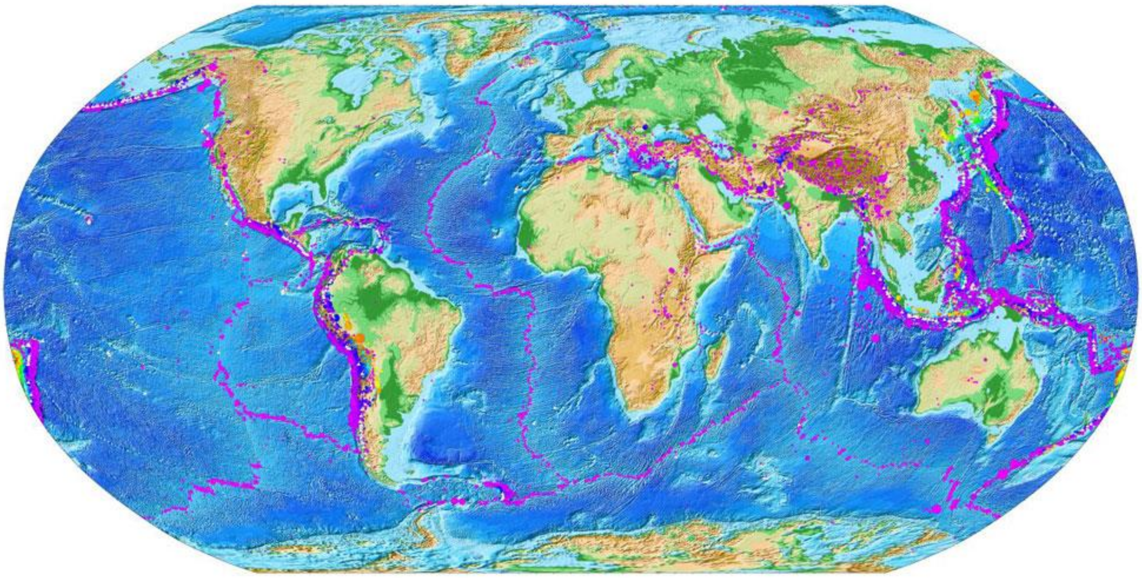

Figure 1.1: Map of global seismic hazard (Giardini, Grünthal, Shedlock, & Zhang, 1999)

In the last years, there are many disastrous earthquakes occurred which caused a big human tragedy all around the world. Recent most massive earthquakes around the world they have surface wave magnitude (M_S) above 5.0 in the last ten years is shown in Table 1.1.

Map of seismicity of the eastern Mediterranean region show clearly that Cyprus experiences fewer earthquakes than the surrounding regions. This does not necessarily mean that the earthquakes are less damaging.

Every region has a different seismic potential, different seismic past, different geological and topographical structure and pattern. Thereby their seismic risks will be different.

In order to reduce the seismic risk in a region, the damage possibility must be reduced since the seismic hazard of the region cannot be changed.

Table 1.1: Recent earthquakes in the last 10 years in the world (Motamedi, 2012)

Years	**Location**	**Magnitude**
2006	Mozambique	7.0
2007	Indonesia	8.5
2008	China	7.9
2009	Honduras	7.3
2010	Spain	8.8
2010	China	6.9
2011	Japan	9.1
2011	Turkey	7.2
2012	Iran	6.4
2013	Pakistan	8.3
2014	Thailand	6.1
2015	Nepal	7.8
2016	Italy	6.2
2017	Mexico	8.1
2017	Iran	7.3

Cyprus, as many countries in this part of the world, has a long recorded history.

Cyprus lies in one of the active seismic regions of the eastern Mediterranean basin and the island has been struck by numerous strong earthquakes in its history. Figure 1.2 shows the history of several earthquakes that hits the island.

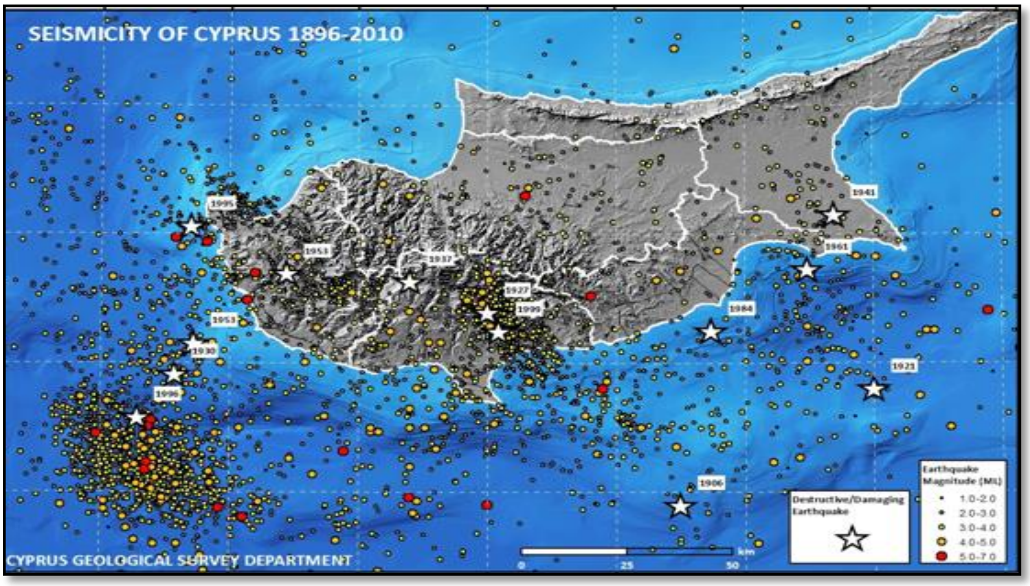

Figure 1.2: Seismicity of Cyprus region between 1896-2010 (GSD, 2010)

A list of some major earthquakes (magnitude, M > 5.0) experienced between the years 1947 to 2015 are listed below.

Table 1.2: Largest earthquakes in Cyprus (GSD, 2015)

Years	Location	Magnitude
1947	Nicosia and Famagusta	5.4
1953	Pafos	6.5
1961	Larnaca	5.7
1995	Pafos	5.7
1996	Pafos	6.8
1999	Lemesos	5.6
2015	Pafos	5.6

Since Cyprus is located in a seismically active zone, the entire island has always been vulnerable to earthquakes which is the most hazardous kind of disaster. Cyprus is the third biggest island in the Mediterranean Sea with an area of 9251 km^2. It has a northern and southern part as shown in Figure 1.3. North Cyprus is divided into five districts namely; Nicosia (Lefkoşa), Famagusta (Gazimağusa), Kyrenia (Girne), Iskele and Guzelyurt as shown in Figure 1.3. Nicosia is the capital city of the north and south Cyprus. It is the only divided capital city in Europe. A case study is chosen for the northern half of Nicosia (Lefkoşa). The Lefkoşa has a total population of 94824, where around one-third of the northern part whole population lives, according to the latest census which was performed by the State Statistical Institute (Statistics and Research Department, 2015).

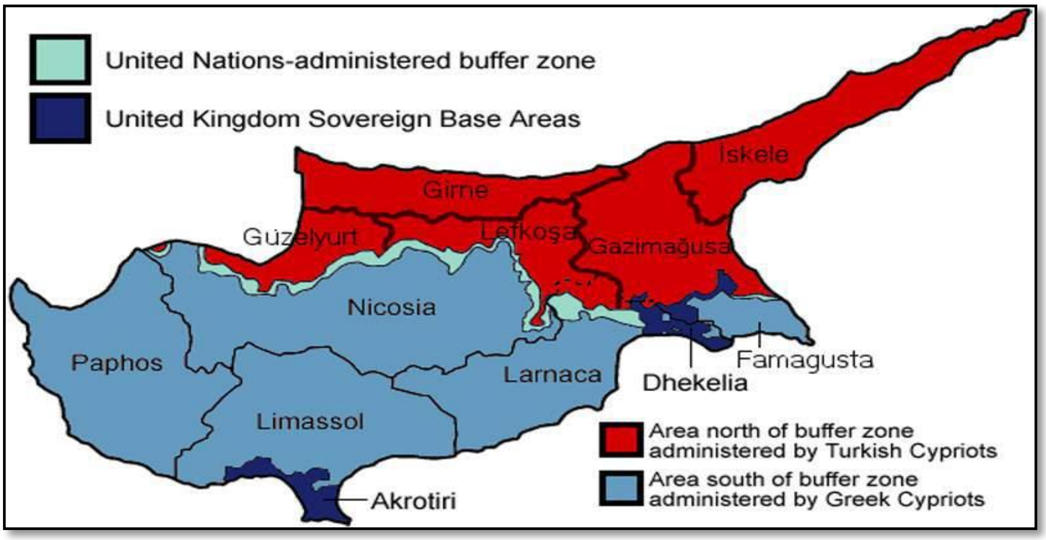

Figure 1.3: Main districts of Cyprus (Yglesias, 2013)

The island is known to have accommodated many communities and cultures throughout its history. As a result of the movement of the population, because of the partition of the island into two, a housing necessity started to take places especially after 1974 (Ozay et al., 2005). Unfortunately, no current scientific building inventory information that conveys the current situation in northern Cyprus. The information obtained in the census carried out by the State Statistical Institute can be used to evaluate the total urban constructions, the building types according to usage and the residential buildings in Lefkoşa. This statistical information's are all presented in Figures1.4 - 1.6, respectively.

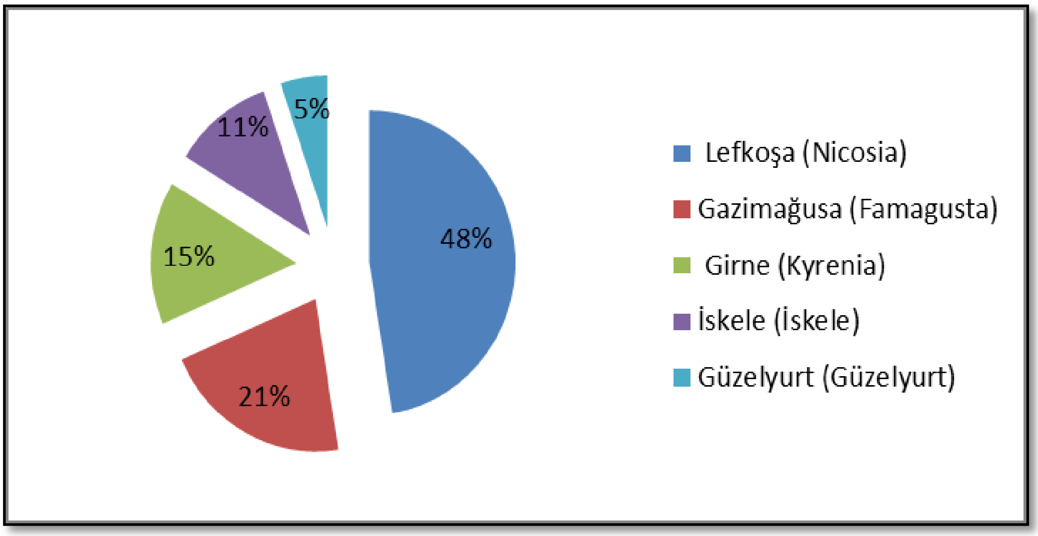

Figure 1.4: Total urban constructions in northern Cyprus 2015 (Statistics and Research Department, 2015)

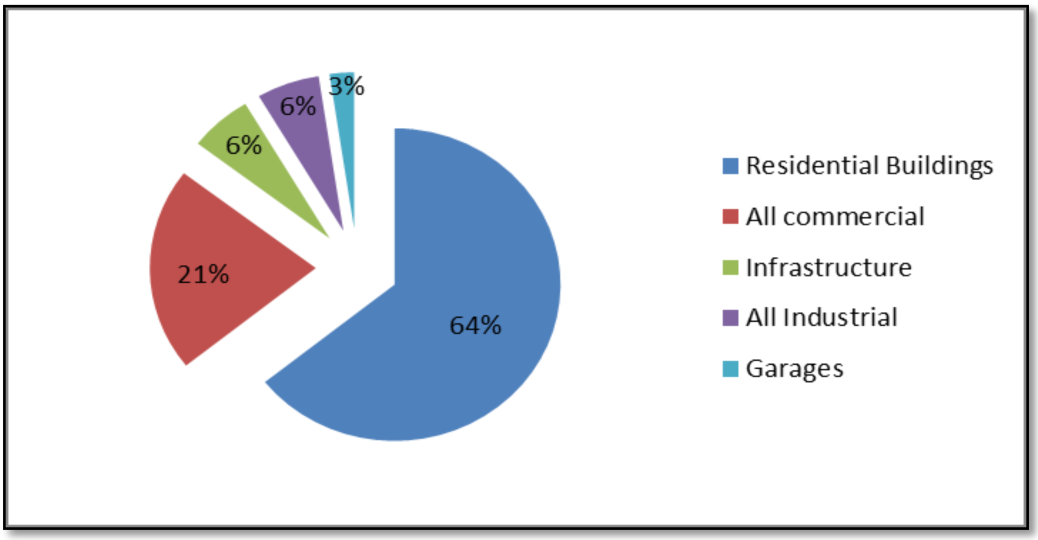

Figure 1.5: Building types in Lefkoşa city according to usage 2015 (Statistics and Research Department, 2015)

The majority of the existing building stock in case study region is low-rise and mid-rise reinforced concrete (RC) buildings. RC buildings are very popular in Northern Cyprus. This method of construction is applied in Northern Cyprus as it is applied in many countries because the implementation of this method is convenient (Yakut, 2004). Besides common loads applied on RC buildings, earthquake is one of the most hazardous actions they have to withstand. Many scientists have carried out several studies to understand the behaviour of this composite material and to propose better solution against natural event. No doubt, Cyprus will continue to be hit with powerful earthquakes in the future as well.

Civil engineers and architects play a major role in improving the seismic capacity of buildings. It has been accepted by engineers and architects that a building configuration, its size and shape and that of its component elements has a significant effect on its behaviour in earthquakes.

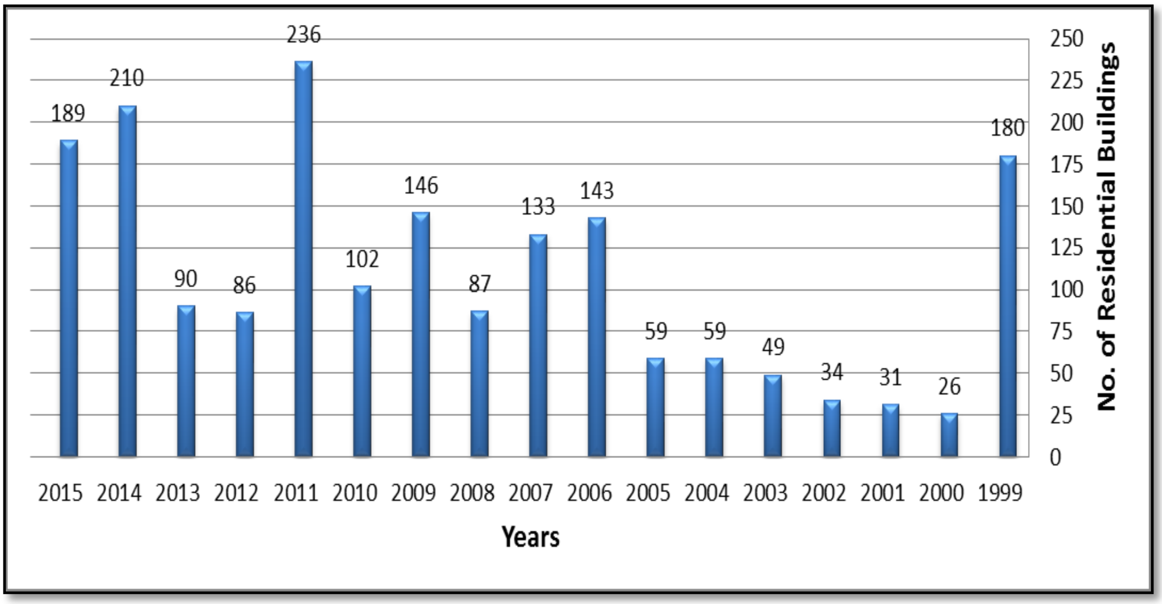

Figure 1.6: Number of residential buildings in Lefkoşa city (Statistics and Research Department, 2015)

To this effect Turkish earthquake regulation is being used for the northern part of the island and peak ground acceleration (PGA) values have been adopted to the northern Cyprus in the time period until 2015. The recent version of the seismic design code in Turkey includes the issue on seismic safety assessment and retrofitting which was published in 2007, (Turkish Earthquake Code, 2007). The further information on Turkish seismic design code and its evolution by time can be found elsewhere (AIJ/JSCE/BU, 2001; Aydınoglu M.N., 2007; Bayülke, 1992; Gülkan, 2000; Ilki & Celep, 2012).

The first seismic design code for structures in Northern Cyprus was established in 2015, which is called “Regulation on buildings to be built in earthquake zones for northern Cyprus”. That was the first national code where the government of that time felt the need for a legal enactment. This code will be nominated as northern Cyprus seismic code (NCSC 2015) in this study.

1.2 Problem Statement

The regulation on buildings to be built in earthquake zones for northern Cyprus (NCSC 2015) has not been studied before. Therefore, there is need to conduct study, in order to determine the performance of the seismic design code, NCSC 2015, with other well-known global codes such as IBC 2009 and EC 8.

1.3 Objective of the Study

The primary objective of this study is to create a comparative evaluation among three seismic design codes including; International building code (IBC2009), Eurocode (EC8) and North Cyprus seismic code (NCSC 2015), to achieve the main aim of this study, the following objectives will be performed:

- To investigate moment-resisting frame (MRF) in regular form and moment-resisting frame with shear wall (MRF+SW) in regular form RC framed buildings.
- To explore the variation in the results obtained.
- To perform equivalent lateral force methods (ELFM) and response spectrum methods (RSM) using ETABS 2015 software.
- To verify the seismic design base shear, story shear, displacement, axial force and bending moments for selected columns under different parameters suggested by codes mentioned above.

1.4 Significance of the Study

This study attempt to examine the first seismic design code of north Cyprus (NCSC 2015), which will be useful in providing a generic level of safety for buildings.

CHAPTER 2

LITERATURE REVIEW

2.1 Overview

Several types of investigations related to comparisons between structural codes are readily available. Since the past decade, many papers and academic research works have been published, mainly as journal articles and conference proceedings which have been reviewed as a part of this study.

Doğangün, Adem, & Livaoğlu (2006) investigated the seismic verification, and dynamic analysis of given types of buildings located at code defined different sites using different codes namely; (TEC, UBC, IBC and EC 8), to investigate the seismic response of the structures, elastic analyses were implemented by the response spectrum method using the SAP2000 program. The result showed that EC 8 gives the higher base shear for similar ground types defined in the other codes. The maximum base shears occurred for ground types of D or E defined in EC 8. Also, it was noted that the ground types have a significant role in occurring the maximum shear force.

Safkan (2012) has presented comparative study between two codes which used different seismic zoning and different PGA values for the same region. These codes include Eurocode 8 (used in the Southern region the island) and Turkish Earthquake Code 2007 (used in the Northern region of the island). The study focused on this point where that cause that TEC 2007 gives much lower base shear values compare to EC 8, and this matter becomes a point of judgment in the design codes. Moreover, SAP2000 software has been used to analyse in two site locations which have the same building in Nicosia and

Famagusta cities. The results indicate that use of TEC 2007 Code with present seismic zoning map results at an unsafe level for estimation of seismic loads in Famagusta region, while soil amplification factors provided by EC 8 lead to in higher values, also the PGA value found to slighter compared to TEC 2007 map.

Landingin, Rodrigues, Varum, Arêde, & Costa (2012) have been presented a comparative study on the seismic provisions using three seismic design codes. The European code (Eurocode 8 or EC 8), the Philippine code (National Structural Code of the Philippines or NSCP 2010), and American code (International Building Code 2009 or IBC 2009), to the most ordinary popular residential construction of standard occupancy.

SAP2000 was used to create the structural model for the RC frames. It was observed that the EC 8 was found to be conservative as compared to NSCP 2010 and IBC 2009. Most of the representative columns need an additional increase of 20% to 40% more reinforcements as compared with NSCP 2010 and IBC 2009. It was noted that EC 8 considered the influences of earthquake actions of the load combination cases in both directions, while it was not found in other codes.

Resatoglu & Atiyah (2016) examined the design rules of TEC 2007 and EC 8 using STA4-CAD V12.1 to analysis and design of four stories reinforced concrete constructions, according to EC 8 and TEC 2007. It was found that a high ductility reduction factor affects base shear which causes an increase in the cost of construction within increase of some stories. The design the outputs of research shows that with TEC 2007 percentage steel reinforcement has increased if compared with and EC 8 in the two cases that studied in the research.

Zasiah, Johinul, & Tameem (2016) investigated the seismic performance of a multi-storied reinforced concrete moment resisting framed building under static and dynamic loading as per Bangladesh National Building Code (BNBC, 2006) by launching a comparative study has been carried out between static and dynamic analysis.

A ten storied reinforced concrete (RC) multi-storied building has been modelled and then analysed using ETABS 2015 software package. Based on computing modelling output data, it has been found that the base shear obtained from response spectrum method analysis is less compared to equivalent lateral force method, the whereas, maximum story-displacement obtained from dynamic response spectrum analysis is about 78% of that of static analysis. At the same time, in case of the maximum bending moment in an interior column, the dynamic value is approximately 87% of the static value.

Kumar (2017) has been presented the comparison between equivalent static technique & response spectrum technique to analyse the model for observing the lateral displacement of the structure in a regular and irregular structure in various zones.

The lateral forces are calculated by using the STAAD Pro, and the building model was analysed using ETABS. However, parameters such as base shear, time period, natural frequency, story drift and bending moments are studied.

The study conducted that linear static analysis observed that there is an increase of lateral displacement in the regular frame more than in irregular frame in respect of different zones.

Bagheri, Firoozabad, & Yahyaei (2012) has been presented the accuracy and exactness of time history analysis in comparison with the most commonly adopted response spectrum analysis and equivalent static analysis. Moreover, ETABS and SAP 2000 were used to model the Multi-story irregular structures with 20 stories. The results show that the static analysis was greater than dynamic analysis including response spectrum and time history analysis. Also, it was noted that for high-rise building the static analysis is not satisfactory and it is essential to provide dynamic analysis. The consequences of the static analysis were uneconomical as the values of displacement are greater than dynamic analysis.

CHAPTER 3

METHODOLOGY

3.1 Overview

This chapter presents the selected case study and discuss the modelling of RC framed structures and explore the variations in the results obtained using the three seismic design codes.

3.2 Case Study

The location of the building is assumed to be at Lefkoşa city in northern Cyprus as shown in Figure 3.1. The RC frame building in this study was designed with consideration of seismic codes. It's well known that earthquake is one of the most hazardous actions on buildings which must be studied, besides common loads applied on RC buildings have to withstand, where the biggest earthquake with surface wave magnitude 6.5 struck the island in the 1953 and caused 40 fatalities (Ambraseys, 2009). According to a recent united nations seismic hazard research in Lefkoşa region, the estimated peak ground acceleration is 0.32g with %10 probability exceedance in 50 years and the lowest shear wave velocity for Lefkoşa is 209 m/s (Resatoglu & Atiyah, 2016).

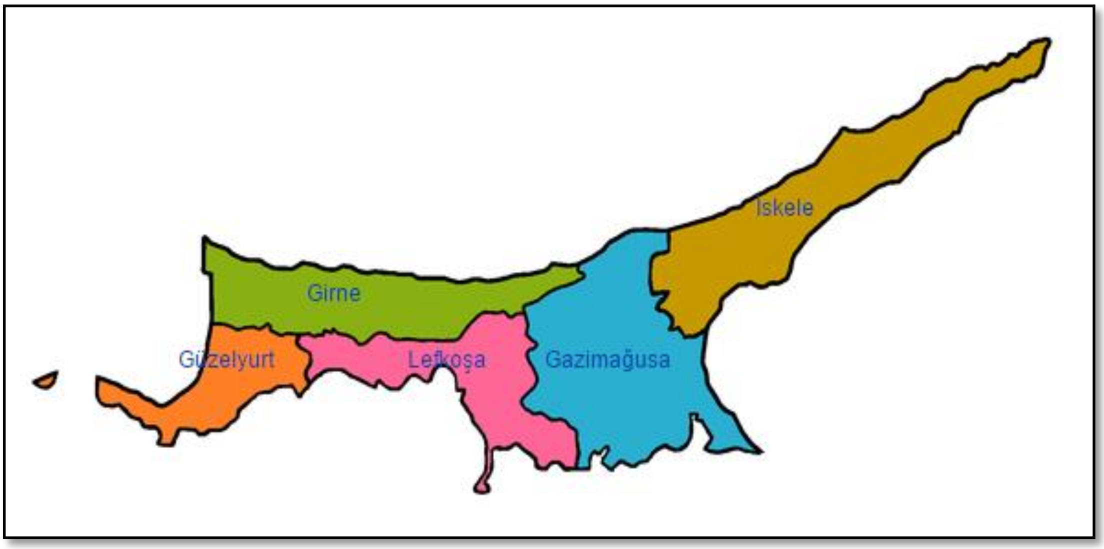

Figure 3.1: Northern part of Cyprus and its districts (Makris et al., 1983)

Recently, due to political issues in Cyprus, two different design codes were provided. These are regulation on buildings to be built in earthquake zones for northern Cyprus (NCSC 2015) for northern part and Eurocode 8 (EC 8) for the southern part of the island, where both codes use different seismic zone map and different peak ground acceleration (PGA). Seismic zones cited in this specification are the second and third seismic zones depict in seismic zoning map of northern Cyprus prepared and mutually consulted by the Chamber of Cyprus Turkish Civil Engineers and Ministry of Public Works and Transport department.

Figure 3.2 shows the seismic zoning map of Cyprus according to EC8 Cyprus National Annex. It was observed that Lefkoşa city have the PGA value of 0.2g (CEN, 2004). Also, Figure 3.3 shows the seismic zoning map that has been adapted to the northern part of the island with a PGA value between 0.2 - 03g for Lefkoşa city (Chamber of Civil Engineers, 2015). Compared to the EC 8 map, higher ground shaking values can be seen in the NCSC 2015 map.

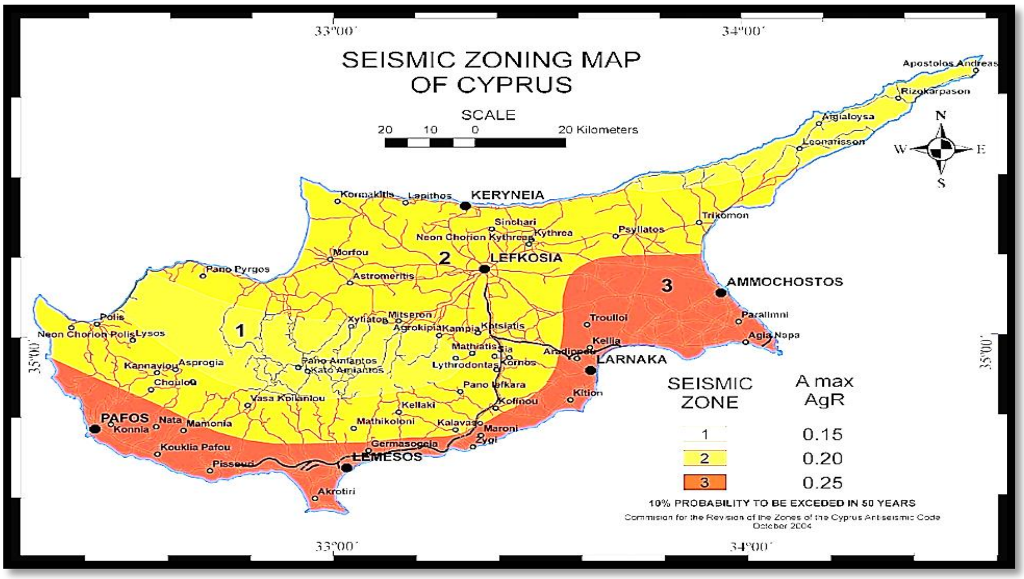

Figure 3.2: Seismic map zoning according to EC 8 national annex Cyprus EN 1998-1:2004 (GSD, 2004)

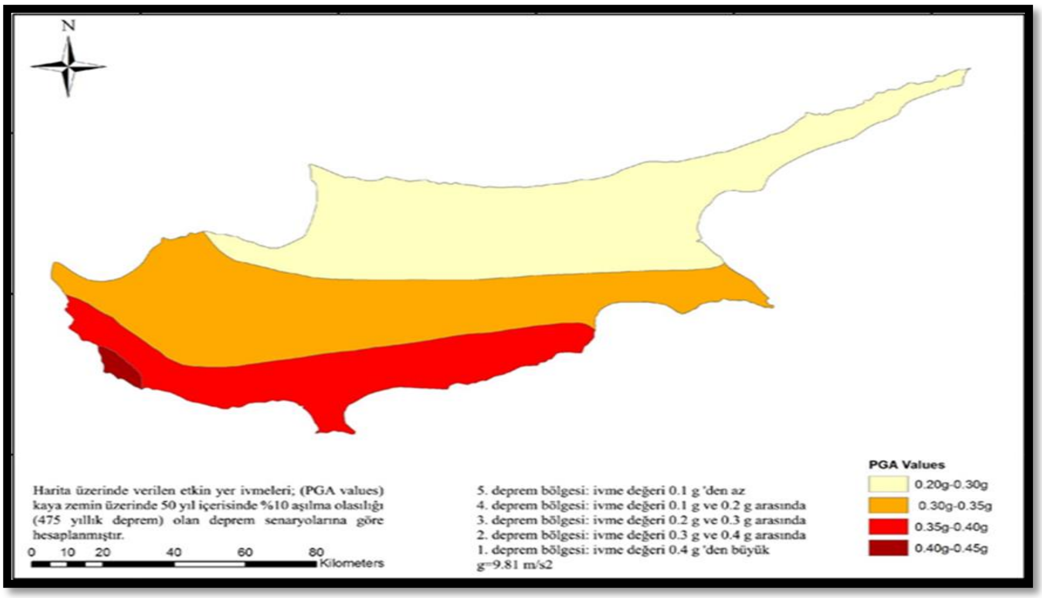

Figure 3.3: Seismic map zoning according to NCSC 2015 (Chamber of Civil Engineers, 2015)

3.3 Modelling of RC Framed Structures

The total height of the building above the ground level considered for the study is 15.6 m. In the present study, five stories (ground +4) reinforced concrete residential building of 21.5 m × 14.5 m in the plan has been considered for the comparison, as shown in Figures 3.4 and 3.5, respectively.

Four types of RC buildings have been modelled and analysed in this study, namely:

1) Five-story moment-resisting frame (MRF) in regular form analysis using ELFM
2) Five-story moment-resisting frame (MRF) in regular form analysis using RSM
3) Five-story moment-resisting frame with shear wall (MRF+SW) in regular form analysis using ELFM
4) Five-story moment-resisting frame with shear wall (MRF+SW) in regular form analysis using RSM

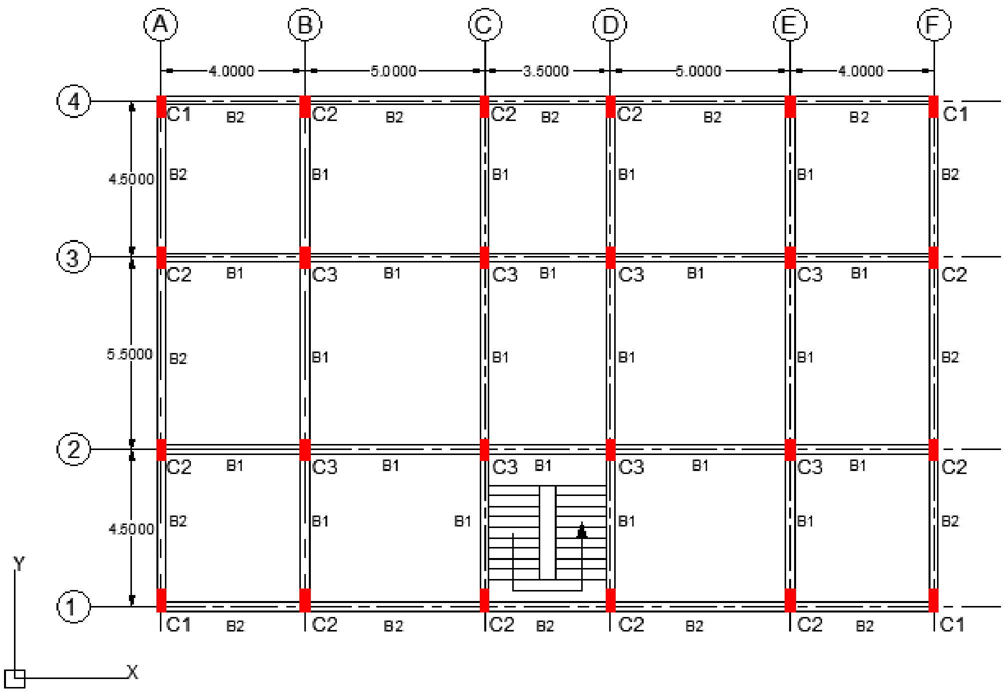

Figure 3.4: Floor plan for five story moment-resisting frame in regular form

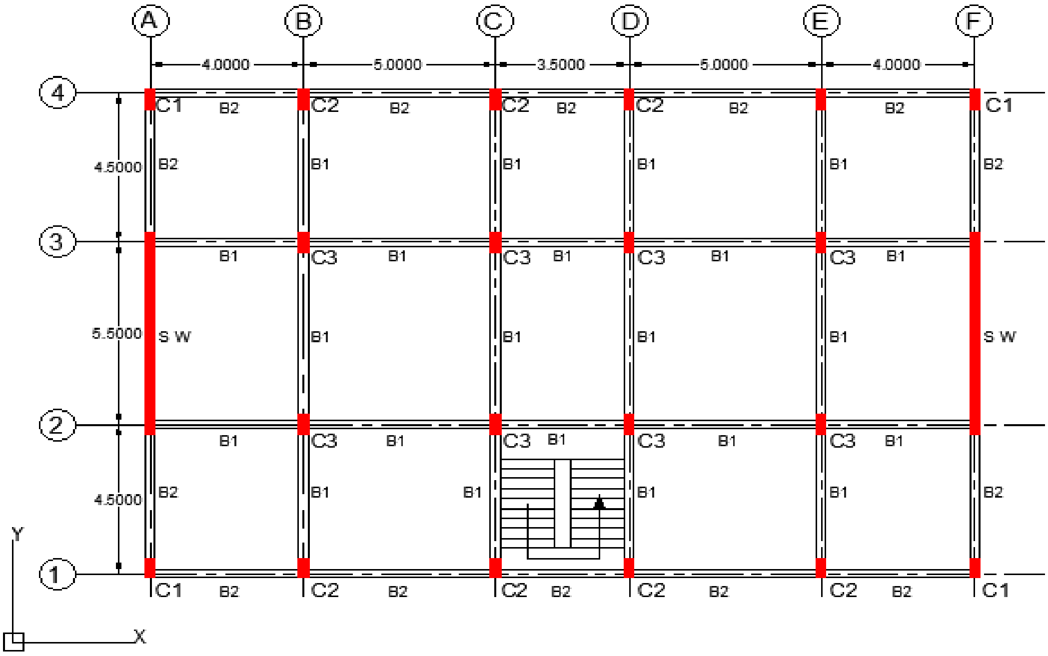

Figure 3.5: Floor plan for five story moment-resisting frame with shear wall in regular form

Typical floor height of RC building is 3 m, and all stories are considered as typical floors. The compressive strength of concrete was considered as 30 MPa, and the yield strength of the steel was selected as 420 MPa. The damping ratio was taken as 0.05. The dimensions of slabs, beams and columns are listed in Tables 3.1-3.3 respectively. In the frame buildings, some members were selected for the aim of the analysis. The selected column members (corner, exterior, interior) are shown in Table 3.3.

Table 3.1: Layout of slab for the residential building

Number of floors	Type of slab	Thickness (mm)	Description of slab
G, 1, 2, 3 and 4	S1	150	Slab for floors
	S2	150	Slab for stairs

Table 3.2: Layout of beams for the residential building

Number of floors	Type of beam	Dimensions (mm)	Carrying
G, 1, 2, 3 and 4	B1	500*250	Internal walls
	B2	500*250	External walls

Table 3.3: Layout of columns for the residential building

Number of floors	Type of column	bx (mm)	by (mm)
G, 1, 2, 3 and 4	Corner Column (C1)	300	400
	Exterior Column (C2)	300	500
	Interior Column (C3)	300	600

The three-dimensional (3D) analysis is carried out under static and dynamic seismic analysis in both x and y directions. ELFM and RSM have been used for performing static and dynamic analysis respectively. The methods were used to verify the seismic design base shear, displacement, story shear, axial force and maximum bending moments for selected columns under different parameters suggested by codes mentioned above.

The ETABS 2015 software package was used for analysis and design of the RC buildings. ETABS 2015 is integrated software capable of carry out 3D.

3.4 Load Combination

The load combinations for each seismic code were also utilised in the modelling of RC framed buildings. The different load combinations for 3D analysis are considered in both seismic codes that is shown in Table 3.4. IBC2009, EC 8 and NCSC 2015, considered the effects of lateral forces in two directions.

Table 3.4: Load Combinations

Case	IBC2009	EC8	NCSC 2015
DL & LL	1.2 DL + 1.6 LL	1.35 DL + 1.5 LL	1.4 DL + 1.6 LL
DL, LL & E	1.2DL+1.0LL ± 1.0E_X	1.0DL+0.3LL ± 1.0E_X	1.0DL+1.0LL ± 1.0E_X
	1.2DL+1.0LL ± 1.0E_Y	1.0DL+0.3LL ±1.0E_Y	1.0DL+1.0LL ± 1.0E_Y
	—	1.0DL+0.3LL ± 1.0E_X ± 0.3E_Y	1.0DL+1.0LL ± 1.0E_X ± 0.3E_Y
	—	1.0DL+0.3LL ± 0.3E_X ± 1.0E_Y	1.0DL+1.0LL ± 0.3E_X ± 1.0E_Y
DL & E	0.9 DL ± 1.0 E_X	—	0.9 DL ± 1.0 E_X
	0.9 DL ± 1.0 E_Y	—	0.9 DL ± 1.0 E_Y

CHAPTER 4

SEISMIC DESIGN CODES

4.1 Overview

This chapter present and discuss the seismic design codes including soil class, seismic zones, importance factors and seismic design loads according to IBC 2009, EC 8, and NCSC 2015.

4.2 Seismic Design Code According to International Building Code (IBC 2009)

International Building Code (IBC) is a comprehensive set of building standards that supply several of profits that govern the design of structures such as the international assembly for building professionals to talk about functioning and normative code necessities. Understanding provisions in the IBC assists to assembly supplies a superior field to argument suggested rescripts in addition to advances international consistency in the application of victuals. Hence, the IBC can govern the design of structures in an attempt to remove conflicting or duplicate standards to achieve minimal rules for construction systems utilizing prescriptive and functioning-pertained victuals. Moreover, it is established on broad-based rules that produce potentially to utilize of Modern materials and new construction designs (ACI, 2015).

According to (ASCE 7-05) which specified minimum design loads for buildings and other Structures that the seismic design loads for constructions and additional structures which are susceptible to building code necessaries require a minimal load. The load and its combinations have been evolved that defined for strength design and acceptable stress design to be used as combined as should be in seismic design loads. In that document, the severity of the design earthquake motion for a concrete structure is described regarding the structure's seismic design category (SDC), which depends on the structure's geographic location and also the soil on which it is built (ACI, 2015).

4.2.1 Soil site class

They are six types of soil to be considered according to IBC 2009 to represent the most common soil conditions are given in Table 4.1. To determining the soil class depend on shear wave velocity. Wherever, the shear wave velocity is unknown to determine the soil class, shall be used soil class D unless the authority having jurisdiction or geotechnical data determines soil class E or F are present at the site (McCormac, 2005).

Table 4.1: Soil site class (McCormac, 2005)

Site Class	**Soil Description**	**Shear Wave Velocity V_{S30} (m/s)**
A	Hard rock	V_S >1500 m/s
B	Rock	760 < V_S < 1500
C	Very dense soil and soft rock	360 < V_S < 760
D	Stiff soil (default site class)	180 < V_S < 360
E	Soft clay soil	V_S < 180
F	Liquefiable soils, quick highly sensitive clays, collapsible weakly cemented soils. These require site response analysis.	

4.2.2 Maximum considered earthquake (MCE)

According to (McCormac, 2005), the severity of maximum considered earthquake level ground is shaking is described regarding the spectral response acceleration parameters S_S and S_1. The parameter S_S is a measure of how strongly the MCE affects structures with a short period 0.2 sec. The parameter S_1 is a measure of how strongly the MCE affects structures with a longer period 1 sec. Once the soil site class is assigned, the corresponding site coefficients for short and long periods, F_a and F_v, respectively, are determined using Table 4.2 and 4.3.

Table 4.2: Mapped MCE spectral response acceleration parameter at short period F_a (McCormac, 2005)

Site Class	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
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	A site response analysis must be performed				

Table 4.3: Mapped MCE spectral response acceleration parameter at long period F_v (McCormac, 2005)

Site Class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
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	A site response analysis must be performed				

The MCE spectral response accelerations for short periods S_{MS} and for long periods S_{M1} are defined by the following:

$$S_{MS} = F_a S_S \tag{4.1}$$

$$S_{M1} = F_v S_1 \tag{4.2}$$

The design spectral acceleration parameters are defined by the following:

$$S_{DS} = \left(\frac{2}{3}\right) S_{MS} \tag{4.3}$$

$$S_{D1} = \left(\frac{2}{3}\right) S_{M1} \tag{4.4}$$

4.2.3 Importance factors and risk

The occupancy of a building is an important consideration in determining its SDC. These risk categories are correlated to important factors that range from 1.0 to 1.5. The importance factor and risk categories are given below in Table 4.4.

Table 4.4: Importance factor and risk categories (McCormac, 2005)

Occupancy of Buildings and Structures	Risk Category	Importance Factor *I*
Buildings and other structures that represent a low risk to human life in the event of failure	I	1.00
All buildings and other structures except those listed in Risk Categories I, III, and IV	II	1.00
Buildings and other structures, the failure of which could pose a substantial risk to human life	III	1.25
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and /or mass disruption of day-to-day civilian life in the event of failure		
Buildings and other structures not included in Risk, Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where there quantity exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.		
Buildings and other structures designated as essential facilities.	IV	1.50
Buildings and other structures, the failure of which could pose a substantial hazard to the community.		
Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous, chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity exceeds a threshold quantity established by the authority having jurisdiction to be dangerous to the public if released and is sufficient to pose a threat to the public if released.		
Buildings and other structures required to maintain the functionality of other Risk Category IV structures.		

4.2.4 Seismic design categories (SDC)

To determine SDC depend on the seismic hazard level, soil type, risk category, and Occupancy as shown in Table 4.5 and 4.6.

Table 4.5: SDC based on short period S_{DS} (McCormac, 2005)

Value	Risk 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 4.6: SDC based on long period S_{D1} (McCormac, 2005)

Value	Risk 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

4.2.5 Seismic design loads

The design seismic base shear V, in each principal plan direction is defined by the following:

$$V = C_S W \tag{4.5}$$

where

W = the effective weight

C_S = the seismic response coefficient

The seismic response coefficient C_S, is defined by the following:

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \tag{4.6}$$

and need not exceed

$$C_S = \frac{S_{D1}}{\left(\frac{R}{I}\right)T} \quad \text{for } T \leq T_L \tag{4.7}$$

or

$$C_S = \frac{S_{D1\,.\,T_L}}{\left(\frac{R}{I}\right)T^2} \quad \text{for } T > T_L \tag{4.8}$$

In no case is C_S, permitted to be less than 0.044 I S_{DS} or less than 0.01. When $S_1 \geq 0.6$ g

$$C_S = \frac{0.5\,S_1}{\left(\frac{R}{I}\right)} \tag{4.9}$$

The total design base shear V is distributed to each building level is defined by the following:

$$F_x = \frac{w_x\, h_x^k}{\sum_{i=1}^{n} w_i h_i^k}\, V \qquad (4.10)$$

where

F_x = Design lateral force applied at story *x*

w_x or w_i = Portion of the total effective weight of the structure, W, assigned to level *x* or *i*, respectively

k = an exponent related to the structure period as follows:

- for structures having a period of 0.5 sec or less, *k* = 1
- for structures having a period of 2.5 sec or more, *k* = 2
- for structures having a period between 0.5 sec and 2.5 sec, *k* shall be 2 or shall be determined by linear interpolation between 1 and 2

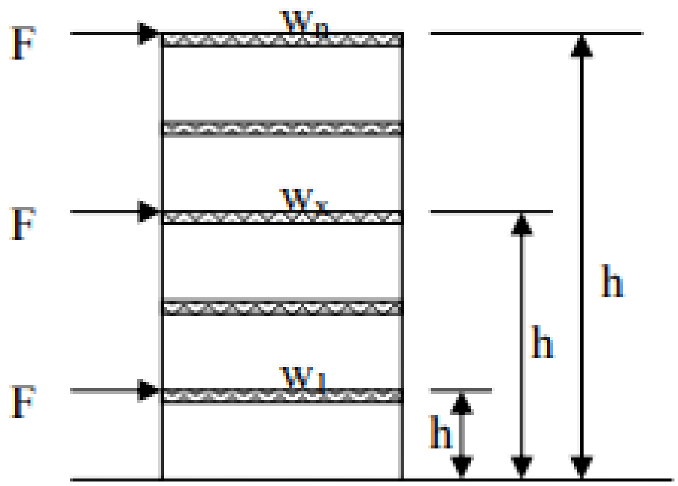

Figure 4.1: Lateral force applied at stories

The approximate first natural vibration period of the building T_a in the two directions is defined by the following:

$$T_a = C_t\, h_n^x \qquad (4.11)$$

where

h_n = The building height above the base to the highest level of the building

Ct = 0.0466 for concrete

x = 0.9

As an alternative, the approximate fundamental period T_a, for structures less than 36 m in height in which the seismic force resisting system consists of concrete moment-resisting frame is defined by the following:

$$T_a = 0.1\,N \tag{4.12}$$

where

N = Number of stories

The response modification coefficient R, reduces the seismic design force for structures capable of responding inelastically. In Table 4.7, the terms ordinary, intermediate and special.

Table 4.7: Response modification coefficients (McCormac, 2005)

Structure Type	R
Building Frame System	
Special reinforced concrete shear wall	6
Ordinary reinforced concrete shear wall	5
Special reinforced concrete moment frames	8
Intermediate reinforced concrete moment frames	5
Ordinary reinforced concrete moment frames	3

Response acceleration S_a, depends on the fundamental period T, as shown in Figure 4.2.

$$S_a = S_{DS}\left[0.4 + 0.6\frac{T}{T_0}\right] \tag{4.13}$$

$$S_a = S_{DS} \tag{4.14}$$

$$S_a = \frac{S_{D1}}{T} \tag{4.15}$$

$$S_a = \frac{S_{D1}T_L}{T^2} \tag{4.16}$$

where

S_{DS} = Design spectral response acceleration parameter at short period

S_{D1} = Design spectral response acceleration parameter at long period

T = The fundamental period

T_0 = $0.2\ S_{D1}/S_{DS}$

T_S = S_{D1}/S_{DS}

T_L = Long-period transition period. To determine, T_L, from Table 4.8

Table 4.8: Long-period transition period (Council, 2015)

$\mathbf{M_S}$	$\boldsymbol{T_L}$ **sec**
6.0 - 6.5	4
6.5 - 7.0	6
7.0 - 7.5	8
7.5 - 8.0	12
8.0 - 8.5	16
8.5 - 9.0	20

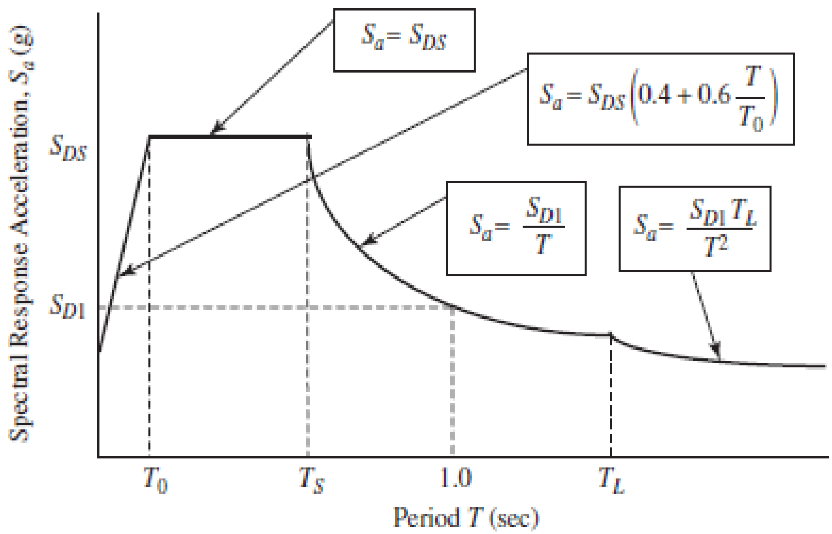

Figure 4.2: Design response spectrum (McCormac, 2005)

4.3 Seismic Design Code According to Eurocode (EC 8)

Eurocode 8 specified as Design of structures for earthquake resistance, which has prepared CEN/TC250 on behalf of Technical Committee, the responsible for all structural Eurocodes (BSI) has grouped formulas for buildings with universal set and seismic activities (CEN, 2004).

The European Committee for Standardisation has developed code for the structural design of construction works in the European Union which is known as Eurocodes.

At the present time, Eurocode is mandatory for the specification of European public works including the European continent in general. In addition, each country is expected to issue a national annex to the European rules that will need to be referred to a particular country.

4.3.1 Ground condition

They are five types of soil to be considered according to EC 8 to represent the most common soil conditions are given in Table 4.9. To determining the soil type which depend on shear wave velocity (CEN, 2004).

Table 4.9: Ground types (CEN, 2004)

Ground Type	Soil Description	Shear Wave Velocity V_{S30} (m/s)
A	Rock or other rock like geological formation.	$V_S > 800$
B	Deposits of very dense sand, gravel or very stiff clay, at least tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	$360 < V_S < 800$
C	Deep deposits of dense or medium dense sand, gravel or stiff clay with a thickness from several tens to many hundreds of metres.	$180 < V_S < 360$
D	Deposits of loose to medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft to firm cohesive soil.	$V_S < 180$
E	A soil profile consisting of a surface alluvium layer with vs values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with Vs > 800 m/s.	
S1	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.	$V_S < 100$ (indicative)
S2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S1	

4.3.2 Seismic zones

The national territories are divided by national authorities into seismic zones, according to the local hazard for Cyprus as shown in Figure 3.2. The reference peak ground acceleration a_gR according to national authorities to the requirement for no collapse can be chosen for any seismic zone (CEN, 2004). Also, according to national authorities can choose the peak ground acceleration reference from P_{NCR} which is the reference probability of exceedance in 50 years (Solomos, Pinto, & Dimova, 2008). Within the scope of EC 8, the movement of the earthquake at a certain point on the surface due to the spectrum of elastic earth acceleration response is called (elastic response spectrum), as shown in Figure 4.3.

The elastic response spectrum $S_e(T)$, for horizontal components of the seismic action is defined by the following:

$$S_e(T) = \mathrm{a_g}\, S \left[1 + \frac{T}{T_B}(\eta\ 2.5 - 1)\right] \qquad 0 \leq T \leq T_B \qquad (4.17)$$

$$S_e(T) = \mathrm{a_g}\, S\, \eta\ 2.5 \qquad T_B \leq T \leq T_C \qquad (4.18)$$

$$S_e(T) = \mathrm{a_g}\, S\, \eta\ 2.5 \left[\frac{T_C}{T}\right] \qquad T_C \leq T \leq T_D \qquad (4.19)$$

$$S_e(T) = \mathrm{a_g}\, S\, \eta\ 2.5 \left[\frac{T_C\, T_D}{T^2}\right] \qquad T_D \leq\ T\ \leq 4_S \qquad (4.20)$$

where

$S_e(T)$ =	Elastic response spectrum
T =	The vibration period
$\mathrm{a_g}$ =	Design ground acceleration ($\mathrm{a_g} = I\, \mathrm{a_g}$R)
T_B =	Lower limit of the period of the constant spectral acceleration branch
T_C =	Upper limit of the period of the constant spectral acceleration branch
T_D =	Value defining the beginning of the constant displacement response range of the spectrum
S =	Soil factor
η =	Damping correction factor, η = 1 for 5% viscous damping

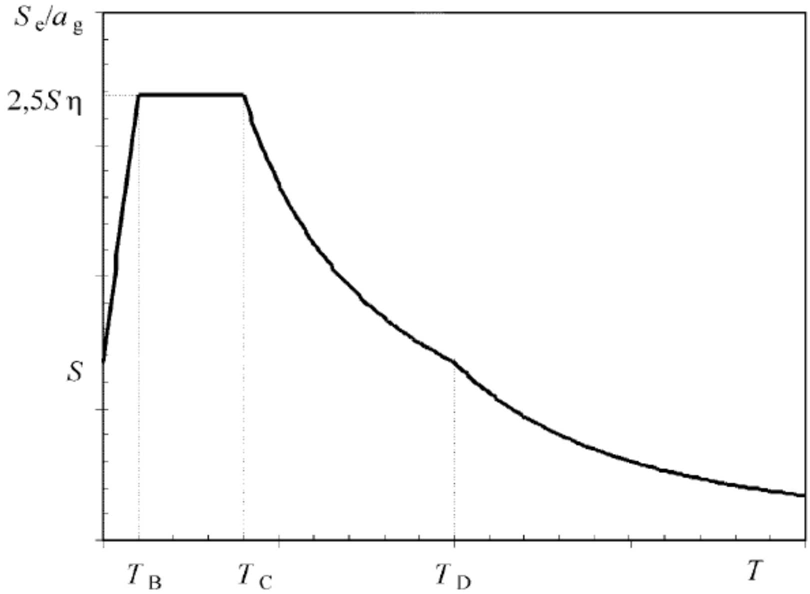

Figure 4.3: Elastic response spectrum

There are two types of elastic response spectra which are type 1 and type 2 taken into account for varying seismicity conditions (Schott & Schwarz, n.d.). In this regard, the provisions of EC 8 provide the following: (If the earthquakes that contribute most to the seismic hazard defined for the site for probabilistic hazard assessment have a surface wave magnitude, Ms, not greater than 5.5, it is recommended that the type 2 spectrum is adopted) (CEN, 2004). The values of soil factor S and periods T_B, T_C, T_D which describes the shape of the elastic response spectrum, depending on the soil type are given in Table 4.10 values elastic response spectrum for type 1 and in Table 4.11 values elastic response spectrum for type 2 (Schott & Schwarz, n.d.).

Table 4.10: The values for type 1 (CEN, 2004)

Ground Type	***S***	T_B	T_C	T_D
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.20	0.6	2.0
D	1.35	0.20	0.8	2.0
E	1.4	0.15	0.5	2.0

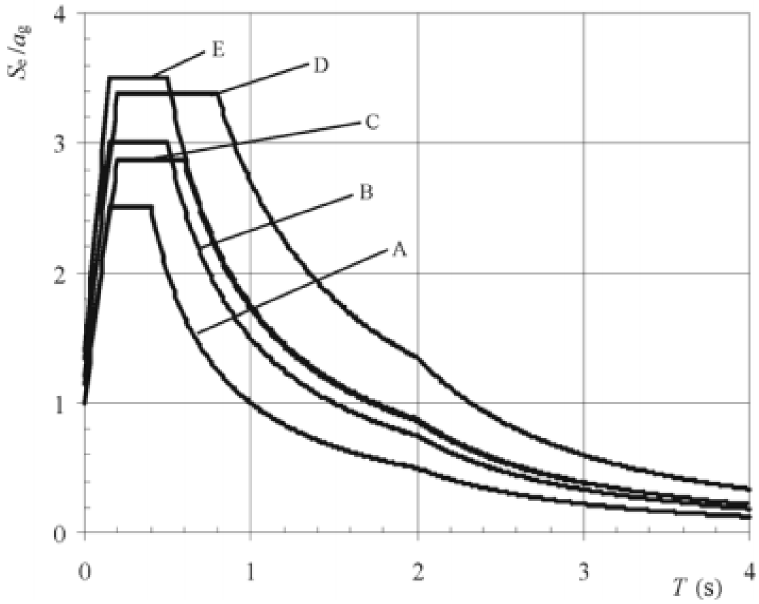

Figure 4.4: Elastic response spectrum for ground types for type 1

Table 4.11: The values for type 2 (CEN, 2004)

Ground Type	S	T_B	T_C	T_D
A	1.0	0.05	0.25	1.2
B	1.35	0.05	0.25	1.2
C	1.5	0.10	0.25	1.2
D	1.8	0.10	0.30	1.2
E	1.6	0.05	0.25	1.2

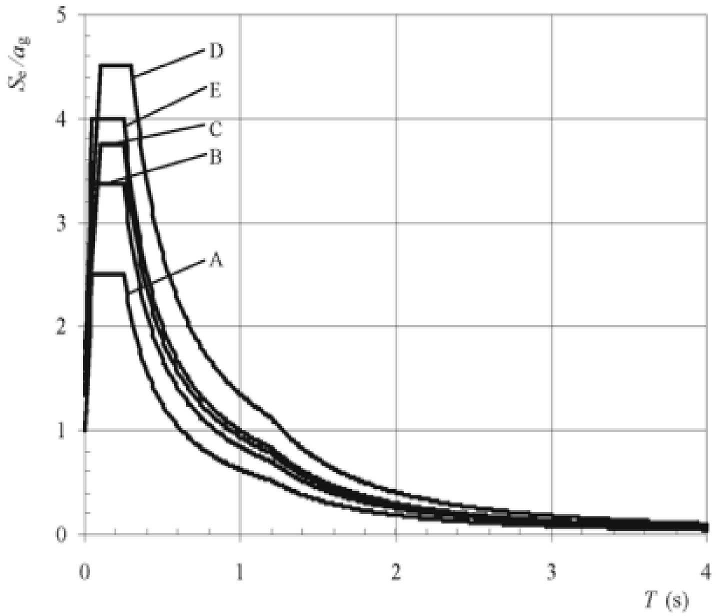

Figure 4.5: Elastic response spectrum for ground types for type 2 (CEN, 2004)

The damping correction factor η is defined by the following:

$$\eta = \sqrt{\frac{10}{(5+\xi)}} \geq 0.55 \tag{4.21}$$

where

ξ = Viscous damping ratio of the structure expressed as a percentage

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

$$S_{De}(T) = S_e(T)\left[\frac{T}{2\pi}\right]^2 \tag{4.22}$$

4.3.3 Importance classes

The building importance classes *I*, is given below in Table 4.12.

Table 4.12: The values of importance classes (CEN, 2004)

Importance classes (*I*)	Buildings and Structures	The value
I	Buildings of minor importance for public safety agricultural buildings, etc.	0.8
II	Ordinary buildings, not belonging to the other categories.	1.0
III	Building whose seismic resistance is important given the consequence associated with a collapse. school, assembly halls, cultural institutions.	1.2
IV	Building whose integrity during earthquakes is of vital importance for civil protection, hospitals, fire stations, power plants.	1.4

4.3.4 Design spectrum for elastic analysis

The reduction of response spectrum that accomplished by insert the behaviour factor q concerning elastic one is known as an elastic analysis which is termed as design spectrum. However, the behaviour factor q may be used in the elastic analysis model in the structure in case it is response completely elastic with 5% viscous damping and according to the relevant ductility classes in the various Parts of EN 1998 the factor q can be as well utilized to account the effect of the viscous damping being different from 5% whose given to different materials and structural systems. The classification should be considered softness in each direction. In different horizontal directions of the structure, the value of the q behavior factor may be different.

Definitions and requirements for structural systems of higher ductility classes (DCH) and structural systems of medium ductility classes (DCM). The terms of structural behaviour factors q are given in Table 4.13.

Table 4.13: The values of behaviour factor q (CEN, 2004)

Type of Structure	DCM	DCH
Uncoupled wall system	3.0	4.0 α_u/α_1
Torsionally flexible system	2.0	3.0
Inverted pendulum system	1.5	2.0
Frame system, dual system, coupled wall system	3.0 α_u/α_1	4.5 α_u/α_1

Table 4.14: The values of factor (α_u/α_1) (CEN, 2004)

Frames	α_u/α_1
Multi-Story, multi bay frames or frame equivalent dual structures	1.3
Multi - Story, one bay frames	1.2
One story buildings	1.1

Table 4.15: The values of factor (α_u/α_1) (CEN, 2004)

Wall	α_u/α_1
Wall equivalent dual or coupled wall systems	1.2
Other uncoupled wall systems	1.1
Wall systems with only two uncoupled walls per horizontal direction	1.0

The design spectrum $S_d(T)$, for the horizontal components of the seismic action is defined by the following (Pitilakis, Gazepis, & Anastasiadis, 2006):

$$S_d(T) = \mathrm{a_g}\, S \left[\frac{2}{3} + \frac{T}{T_B}\left(\frac{2.5}{q} - \frac{2}{3}\right)\right] \qquad 0 \leq T \leq T_B \qquad (4.23)$$

$$S_d(T) = \mathrm{a_g} S\, \frac{2.5}{q} \qquad T_B \leq T \leq T_C \qquad (4.24)$$

$$S_d(T) \begin{cases} = \mathrm{a_g}\, S\, \dfrac{2.5}{q} \left[\dfrac{T_C}{T_B}\right] \\ \geq \beta\, \mathrm{a_g} \end{cases} \qquad T_C \leq T \leq T_D \qquad (4.25)$$

$$S_d(T) \begin{cases} = \mathrm{a_g}\, S\, \frac{2.5}{q} \left[\frac{T_C\, T_D}{T^2}\right] \\ \geq \beta\, \mathrm{a_g} \end{cases} \qquad T_D \leq T \qquad (4.26)$$

where

$\mathrm{a_g}$, S, T_C and T_D =	Are as defined
$S_d(T)$ =	Design spectrum
q =	Behaviour factor
β =	Lower bound factor for the horizontal design spectrum

4.3.5 Seismic design loads

The design seismic base shear F_b, in each horizontal direction, is defined by the following:

$$F_b = S_d(T_1) m\, \lambda \qquad (4.27)$$

where

$S_d(T_1)$ = Design spectrum at period T_1

T_1 = Fundamental period of vibration

m = Total mass of the building

λ = The correction factor. (λ = 0.85 if $T_1 \leq 2Tc$, λ = 1.0 otherwise)

To determine seismic load effect to all stories is defined by the following:

$$F_i = F_b \frac{S_i\, m_i}{\sum S_j m_j} \tag{4.28}$$

where

F_i= Horizontal force acting on story i

F_b = Seismic base shear

S_i , S_j = Displacements of masses m_i, m_j in the fundamental mode shape

m_i, m_j = Story masses

When the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height, the horizontal forces is defined by the following:

$$F_i = F_b \frac{Z_i\, m_i}{\sum Z_j m_j} \tag{4.29}$$

where

Z_i , Z_j = Height of the masses m_i, m_j above the level of application of the seismic action (foundation or top of a rigid basement)

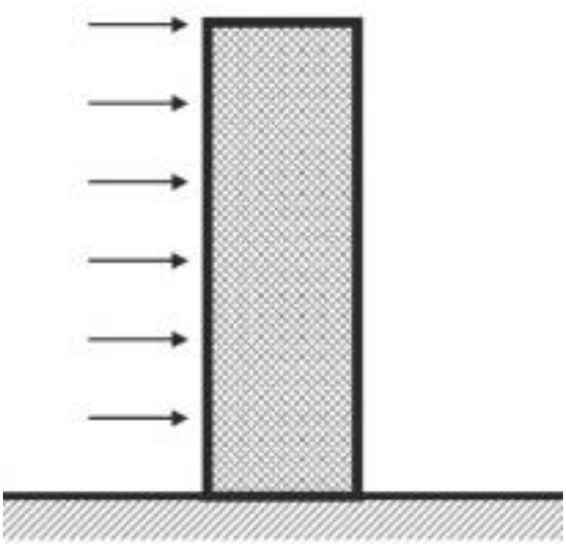

Figure 4.6: Horizontal force acting on stories

The approximate first natural vibration period of the building, T_1, in the two directions is defined by the following:

$$T_1 = C_t . H^{\frac{3}{4}} \tag{4.30}$$

where

C_t = 0.075 for concrete frames, 0.085 for steel frames and 0.05 for all other structures
H = Building height

4.4 Seismic Design Code According to Northern Cyprus Seismic Code (NCSC 2015)

Based on this specifications, the earthquake resistant design general principle is the prevention of elements (both structural and non-structural) of buildings from damage by any low intensity earthquakes, the damage limitation of structural and non-structural elements in medium-intensity earthquakes to repairable levels, and to prevent buildings from high intensity earthquake from partial or total collapse for the loss of life avoidance (Chamber of Civil Engineers, 2015).

4.4.1 Ground condition

Soil types to be considered according to NCSC 2015 to represent the most common local soil conditions are given in Table 4.16. To determining the ground type which depend on shear wave velocity.

As shown in Table 4.17, lists the categories of local sites that should be considered as the basis for determining local soil conditions.

Table 4.16: Ground types (Chamber of Civil Engineers, 2015)

Ground Type	**Soil Description**	**Shear Wave Velocity V_{S30} (m/s)**
A	Massive volcanic rocks, unweathered sound metamorphic rocks, stiff cemented sedimentary rocks	> 1000
	Very dense sand, gravel	> 700
	Hard clay and silty clay	> 700
B	Soft volcanic rocks such as tuff and agglomerate weathered cemented sedimentary rocks with planes of discontinuity	700 – 1000
	Dense sand, gravel	400 - 700
	Very stiff clay, silty clay	300 - 700
C	Highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity	400 - 700
	Medium dense sand and gravel	200 - 400
	Stiff clay and silty clay	200 - 300
D	Soft, deep alluvial layers with high groundwater level	< 300
	Loose sand	< 200
	Soft clay and silty clay	< 200

Table 4.17: Local site classes (Chamber of Civil Engineers, 2015)

Local Site Class	Soil Group and Topmost Soil Layer Thickness (h_1)
Z1	Group (A) soils. Group (B) soils with $h_1 < 15$m
Z2	Group (B) soils with $h_1 > 15$m. Group (C) soils with $h_1 < 15$m
Z3	Group (C) soils with 15 m $< h_1 <$ 50m. Group (D) soils with $h_1 < 10$m
Z4	Group (C) soils with $h_1 > 50$ m. Group (D) soils with $h_1 > 10$m

Note: In the case where the thickness of the topmost soil layer under the foundation is less than 3 m, the layer below may be considered as the topmost soil layer indicated in this table.

4.4.2 Seismic zones

The first, second, third, and fourth seismic zones are mentioned in this specifications for seismic zones depict in seismic zoning map of northern Cyprus prepared and mutually consulted by the Chamber of Cyprus Turkish Civil Engineers and Ministry of Public Works and Transport department, as shown in Figure 3.3.

4.4.3 Importance factor

The building importance factor *I*, is given below in Table 4.18.

Table 4.18: Building importance factor (Chamber of Civil Engineers, 2015)

Occupancy or Type of Building	Importance Factor (I)
Buildings required to be utilised after the earthquake and buildings containing hazardous materials	
a. Buildings required to be utilized immediately after the earthquake (Hospitals, dispensaries, health wards, fire fighting buildings and facilities, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities; governorate, county and municipality administration buildings, first aid and emergency planning stations) b. Buildings are containing or storing toxic, explosive and flammable materials, etc.	1.5
Intensively and long-term occupied buildings and buildings preserving valuable goods	
a. Schools, other educational buildings and facilities, dormitories and Hostels, military barracks, prisons, etc. b. Museums.	1.4
Intensively but short-term occupied buildings	
a. Sports facilities, cinema, theatre and concert halls, etc.	1.2
Other buildings	
a. Buildings are other than above defined buildings. (Residential and office buildings, hotels, building like industrial structures, etc.)	1.0

4.4.4 Definition of elastic seismic loads

The spectral acceleration coefficient $A(T)$, which shall be considered as the basis for the determination of seismic loads is defined by the following:

$$A(T) = A_0 \, I \, S(T) \tag{4.31}$$

where

A_0 = Effective ground acceleration coefficient

I = Building importance factor

$S(T)$ = Spectrum coefficient

Elastic spectral acceleration, $S_{\text{ae}}(T)$, which is the ordinate of elastic acceleration spectrum defined for 5 % damped rate is derived by multiplying spectral acceleration coefficient with gravity g, is defined by the following:

$$S_{\text{ae}}(T) = A\,(T)\text{g} \tag{4.32}$$

The effective ground acceleration coefficient, A_0, is specified in Table 4.19.

Table 4.19: Effective ground acceleration coefficient (Chamber of Civil Engineers, 2015)

Seismic Zone	A_0
1	0.4
2	0.3
3	0.2
4	0.1

4.4.5 Spectrum coefficient

The spectrum coefficient $S(T)$, is defined by the following:

$$S(T) = 1 + 1.5\frac{T}{T_A} \qquad 0 \leq T \leq T_A \tag{4.33}$$

$$S(T) = 2.5 \qquad T_A < T \leq T_B \tag{4.34}$$

$$S(T) = 2.5\left(\frac{T_B}{T}\right)^{0.8} \qquad T_B < T \tag{4.35}$$

where

T_A and T_B = Spectrum characteristic periods are given in Table 4.20

Table 4.20: Spectrum characteristic periods (Chamber of Civil Engineers, 2015)

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

In the case where the requirements specified before are not met, spectrum characteristic periods defined in Table 4.20 for local site class Z4 shall be used.

4.4.6 Special design acceleration spectra

Where required, the flexible acceleration spectrum can be determined through special investigations into local seismic conditions and location conditions. However, the spectral acceleration coefficients corresponding to the acceleration spectrum measurements obtained are by no means less than those specified in Equation (4.31) based on the relevant characteristic periods specified in Table 4.20.

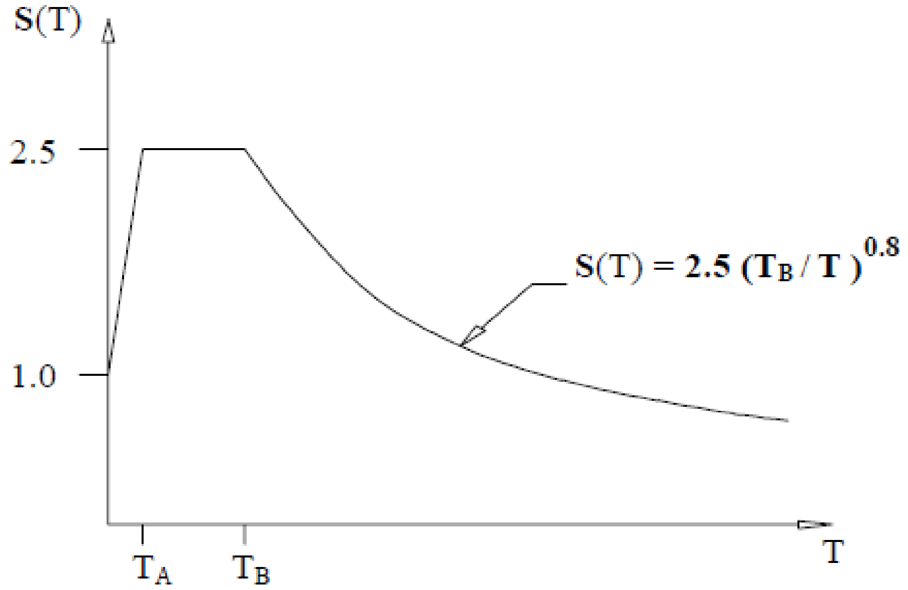

Figure 4.7: Design acceleration spectra (Chamber of Civil Engineers, 2015)

To consider the specific nonlinear behaviour of the structural system during an earthquake, elastic seismic loads to be determined regarding spectral acceleration coefficient shall be divided to below defined seismic load reduction factor to account for. Seismic load reduction factor shall be defined by the Equation (4.36) or (4.37) regarding structural system behaviour factor R, defined in Table 4.12 for various structural systems, and the natural vibration period *T*.

$$R_{\mathrm{a}}(T) = 1.5 + (R - 1.5)\frac{T}{T_A} \qquad 0 \leq T \leq T_A \qquad (4.36)$$

$$R_{\mathrm{a}}(T) = R \qquad T > T_A \qquad (4.37)$$

Definitions and requirements for structural systems of high ductility level (HDL) and structural systems of nominal ductility level (NDL). The terms of structural behaviour factors (R) are given in Table 4.21.

Table 4.21: Structural behaviour factors R (Chamber of Civil Engineers, 2015)

Structural System	Systems of (NDL)	Systems of (HDL)
CAST IN SITU REINFORCED CONCRETE BUILDINGS		
Buildings in which seismic loads are fully resisted by frames	4	8
Buildings in which seismic loads are fully resisted by coupled structural walls	4	7
Buildings in which seismic loads are fully resisted by solid structural walls	4	6
Buildings in which seismic loads are jointly resisted by frames and solid and /or coupled structural walls	4	7

4.4.7 Seismic design loads

The total equivalent seismic load shall be distributed to stories of the building by the following:

$$F_i = (V_t - \Delta F_N)\frac{w_i H_i}{\sum_{j=1}^{N} w_j H_j} \tag{4.38}$$

The additional equivalent seismic load, ΔF_N, acting at the i'th story shall be defined by the following:

$$\Delta F_N = 0.0075\, N\, V_t \tag{4.39}$$

where

V_t = Base shear

N = Total number of stories

$\mathrm{w_i}, \mathrm{w_j}$ = Story weights

H_i, H_j = Height of building

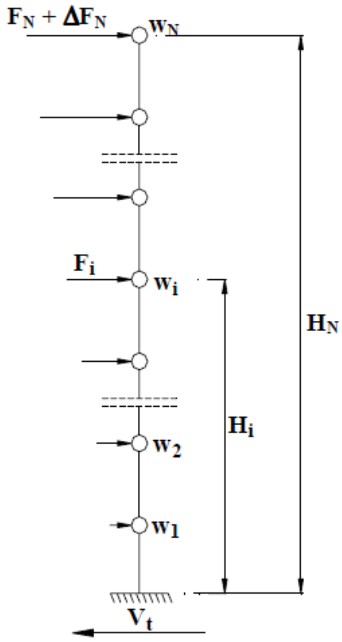

Figure 4.8: The sum of lateral seismic loads acting at story levels (Chamber of Civil Engineers, 2015)

The base shear V_t, acting on the entire building in the earthquake direction is defined by the following:

$$V_t \;=\; W\,\frac{A(T_1)}{R_\mathrm{a}(T_1)} \;\; \geq \;\; 0.1\, A_0\, I\, W \tag{4.40}$$

Total building weight W, is defined by the following:

$$W \;=\; \sum_{i=1}^{N} w_i \tag{4.41}$$

Story weights, w_i , is defined by the following:

$$w_i = DL_i + n\,LL_i \tag{4.42}$$

where

DL_i = Total dead load at story i

LL_i = Total live load at story i

n = Live load participation factor are given in Table 4.22

Table 4.22: Live load participation factor n (Chamber of Civil Engineers, 2015)

Occupancy of Building	$\boldsymbol{n}$
Depot, warehouse, etc.	0.8
School, dormitory, sports facility, cinema, theatre, concert hall, car park, restaurant, shop, etc.	0.6
Residence, office, hotel, hospital, etc.	0.3

The approximate first natural vibration period of the building is defined by the following:

$$T_1 = C_t\, H_N^{3/4} \tag{4.43}$$

where

C_t = 0.07 for RC frames. and 0.08 for steel frames. and 0.05 for all other buildings

H_N = Total height of building measured from the top foundation level

In buildings in which N > 13 excluding basement(s), natural period is not taken more than

$$T_1 = 0.1\,N \tag{4.44}$$

where

N = Total number of stories

CHAPTER 5

SEISMIC ANALYSIS METHODS

5.1 Overview

This chapter presents the seismic analysis methods as shown in Figure 5.1, and how to calculate the design force of each pattern by ETABS.

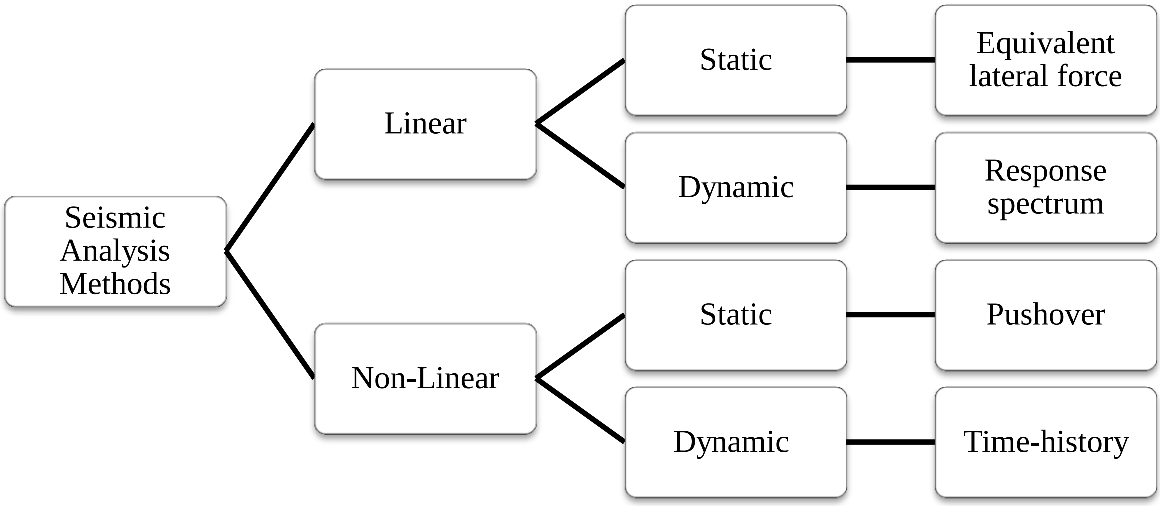

Figure 5.1: Seismic analysis methods (Touqan & Salawdeh, 2013)

A linear method has been used for performing static and dynamic analysis which are ELFM and RSM respectively.

5.2 Equivalent Lateral Force Method

The ELFM analysis is a simple, static and acceptable for most regular structures having specified height for analysis. It is based on the dynamics of a single degree of freedom oscillator or multi degree of freedom system vibrating by the specific shape. The ELFM analysis is one of the common approaches used for seismic demand analysis of structures, which is one of the fastest and most practical methods in most codes. This procedure is used for common structures (residential buildings with low-rise to medium-rise buildings). It can be regarded as the easiest method of analysis due to the forces dependence on the code, which serves as structures' significant period combined with some experimental modifiers. The base shear is to be calculated as a total, followed by its distribution along the height of the building on the basis of a simple formula suitable for mass and stiffness for regular distribution of buildings. The obtained design lateral force depending on the action of the floor diaphragm, shall be distributed to singular lateral load resisting elements in each floor (Yimer, 2014).

Factors such as the size and other earthquake characteristics, site geology, distance from the error, and lateral load resisting type of system influence the seismic forces in a structure. The inclusions of these factors are important in seismic design forces specification. In the procedure for static force, using empirical formulas, the static forces are specified by inertia forces. The representation of a "dynamic characteristics" is not explicitly by experiential formulas for the design or analysis of a particular structure (Di Julio, 2001).

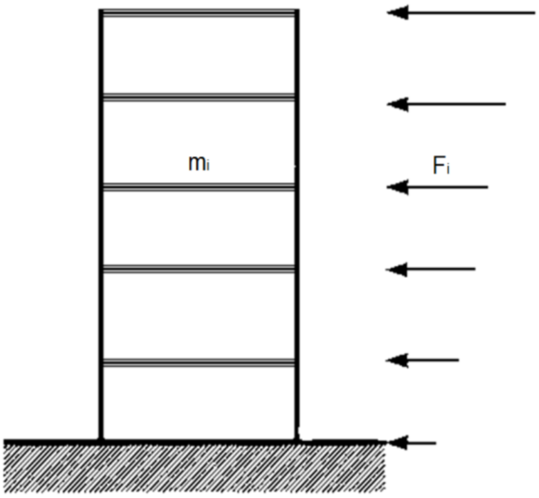

Figure 5.2: Series forces acting on a building to represent the effect of earthquake

5.3 Response Spectrum Method

The curves of response spectrum subjected to specified earthquake are plotted between the time period or frequency and maximum response of SDOF system subjected to ground motion, where assist in getting the peak structural responses under linear range, which can be applied in obtaining lateral forces developed in structure due to earthquake and hence motivates the earthquake-resistant design of structures. Considering the oscillation of its natural frequency, the resulting plot of response spectrum may then be capable of picking up any response of linear system. This application can be in the assessment of earthquake due to peak response of buildings. If the steady state periodic is obtained by calculation of response spectrum using input, then is recorded the steady state result. Damping must be included, otherwise the response will be infinite (Teja & Shahab, 2017).

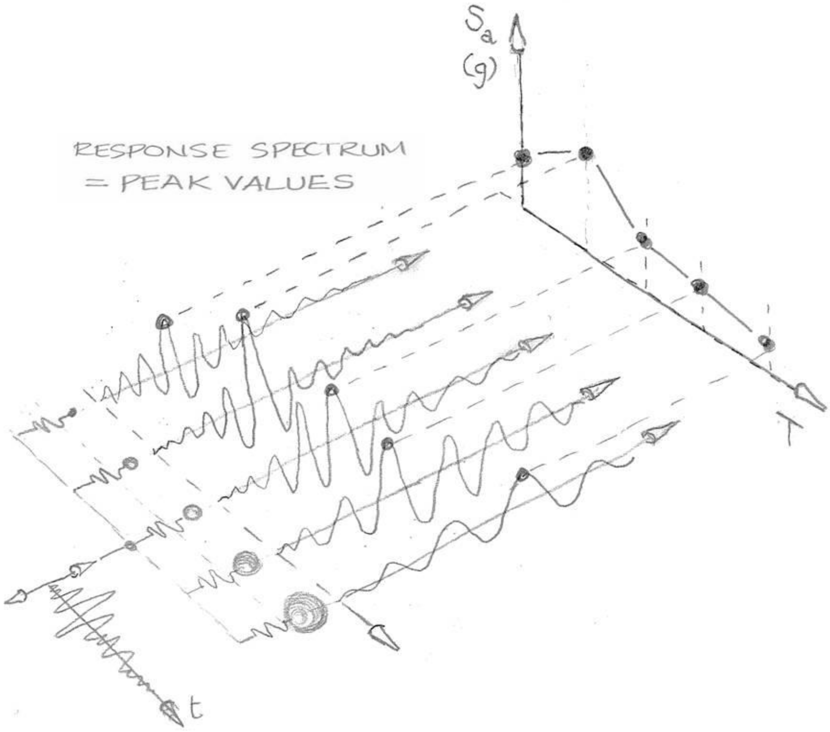

Figure 5.3: Response spectrum curve

5.3.1 Modal analysis

The model response spectrum method is superposition method and is applicable to the analysis of dynamic response of complex structures in their linear range of behavior, in particular analysis of forces and deformations in multi-story buildings. The method depend on certain principles of damping which are clarified models, for many structures, an independent computations and the rest of model's responses can be gathered in determining the total response in each natural mode of vibration, as shown in Figure 5.3. Every model responds according to its own unique pattern of deformation (the modes shape), and frequency (the model frequency), its own modal damping as well as the modal response can be determined by SDOF analysis of an oscillator with their properties (damping and ductility) selected to be represent a particular mode and extent to which the earthquake motion is excited (Yimer, 2014).

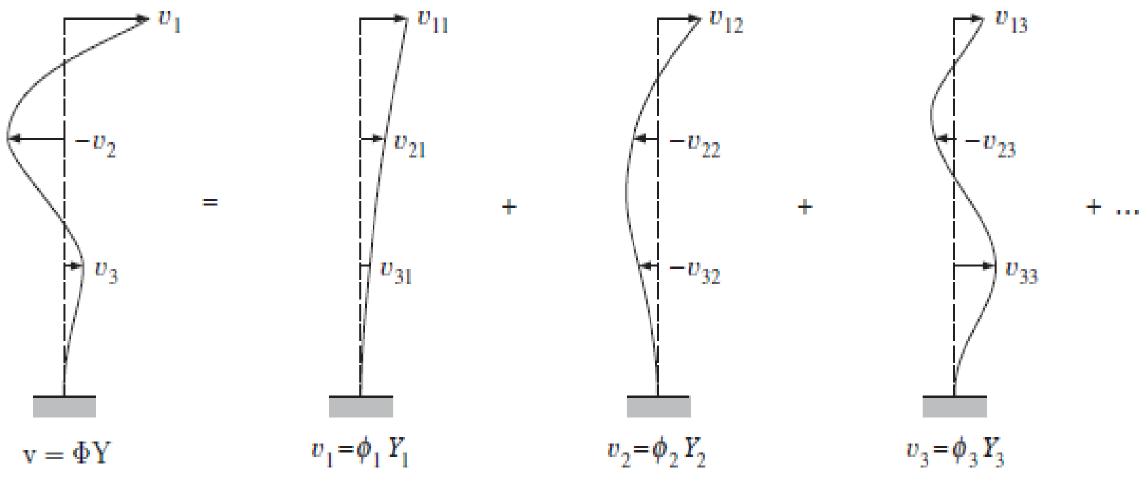

Figure 5.4: The modal components to determine the total response (CSI, 2014)

The compulsory response amount of interest, such as replacement, the structure of the bending moment can be determined by each mode of vibration using the obtained highest response and the shear force. Furthermore, the final highest response shall be obtained by the combination of responses in every mode of vibration by applying the modal combinations rules (Gupta, 1984).

5.3.2 Modal combination rules

The most applied procedure for obtaining the peak response are as follows (Gupta, 1984):

- Absolute Sum Method (ABSSUM), the algebraic summation of all the modes of the peak response by assuming same time occurrence of modal peaks.
- Square root of the sum of squares method (SRSS), by taking in each modal the root of sum of square the maximum response is determined in each mode of vibration.
- Complete quadratic combination method (CQC), the computation of maximum response from all the modes.

If frequency of the model is not close, an appropriate combination method is the SRSS. Since the phase information of the input is lost while response spectrum generation, the result will typically be different from that which would be calculated. However, it is believed that for many buildings, satisfactory approximations to the design forces and deformations can be obtained from the modal method by using the modified design response spectrum for inelastic history systems (CSI, 2014).

Response spectrum method is favoured by earthquake engineering community because of:

1) It provides a technique for performing an equivalent static lateral load analysis.
2) It allows a clear understanding of the contributions of different modes of vibration.
3) It offers a simplified method for finding the design forces for structural members for an earthquake.

The horizontal design force at each floor in each mode is calculated by ETABS. The ETABS outcomes; design values, base shear, story shear, axial force and modal masses.

5.4 ETABS

ETABS has powerful features and are models designed completely for the design of both reinforced concrete structures integrated modules of steel. Moreover, the program enables the user to be able to create, analyse, modify, and make design especially for structural models, all to be done in the same user interface (HabibuIlah, 2000).

Moreover, the software depends on a collection of modules that should be used in building analysis such as static analysis and dynamic analysis. The main program carries the nodes, members, and loads on it, and other modules do the subroutine used for every analysis you need to perform (HabibuIlah, 2000).

5.4.1 Modelling using ETABS

5.4.1.1 Model initialization

One of the international code according to design is chosen from model initialization.

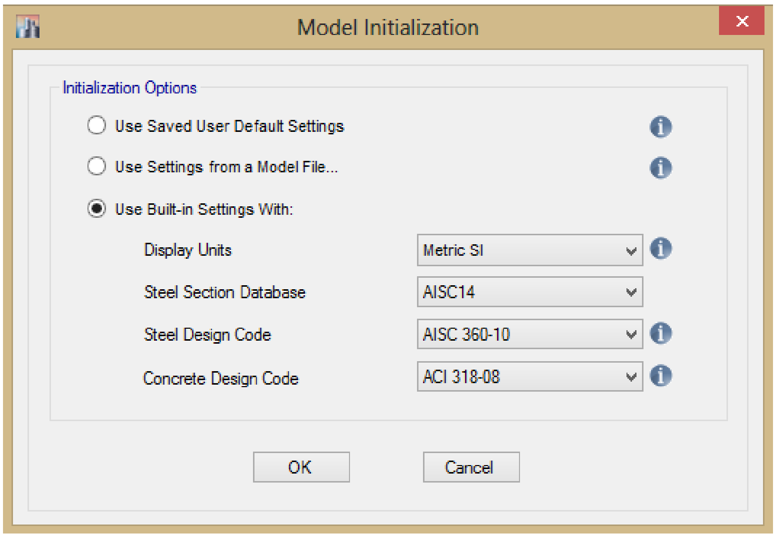

Figure 5.5: Determine the unit and international code

5.4.1.2 Material properties

ETABS enforces design of beams torsion, columns, beams, and slabs for flexure and shear design of the upper material strength limits. The upper limits are defined as the input material strengths if they are considered in the material properties as being higher than the limits. The responsibility of the user is to ensure the satisfaction of minimum strength.

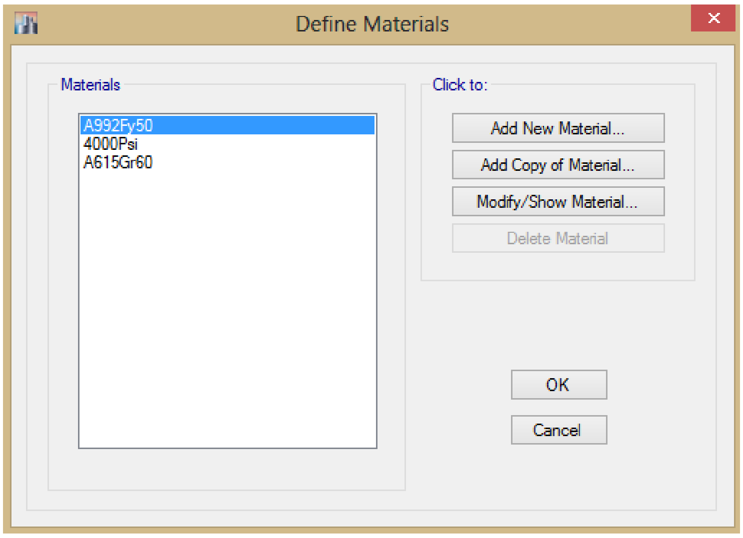

Figure 5.6: Material properties

5.4.1.3 Define loads patterns

Loads represent actions upon the structures, such as force, pressure, support settlement, thermal effects, ground acceleration, and others. An unlimited number of load patterns can be defined in ETABS. Typically, separate load patterns would be defined for dead load, live load, wind load and seismic load, and can include automated loads, such as self-weight or code-specified wind or earthquake lateral force distributions. Loads that need to vary independently, for design purposes or because of how they are applied to the structure, should be defined as a separate load pattern. ETABS uses the type of load pattern to create design load combinations automatically.

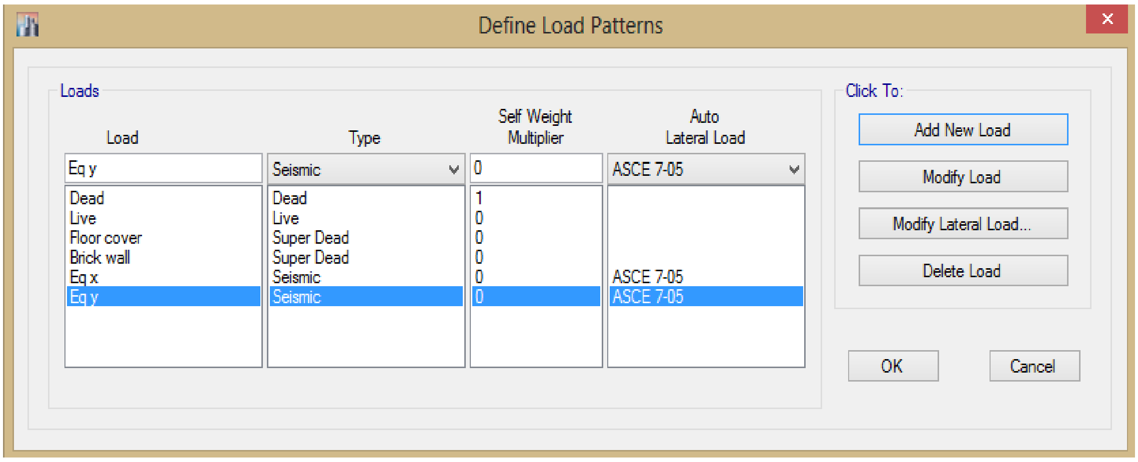

Figure 5.7: Load patterns

5.4.1.4 Mass source data

Mass values are calculated for structural elements according to volume and material density. Mass is then automatically concentrated at joint locations, where the mass is determined by the load pattern according to the code conditions. If is dead load only or dead load with a percentage of live load.

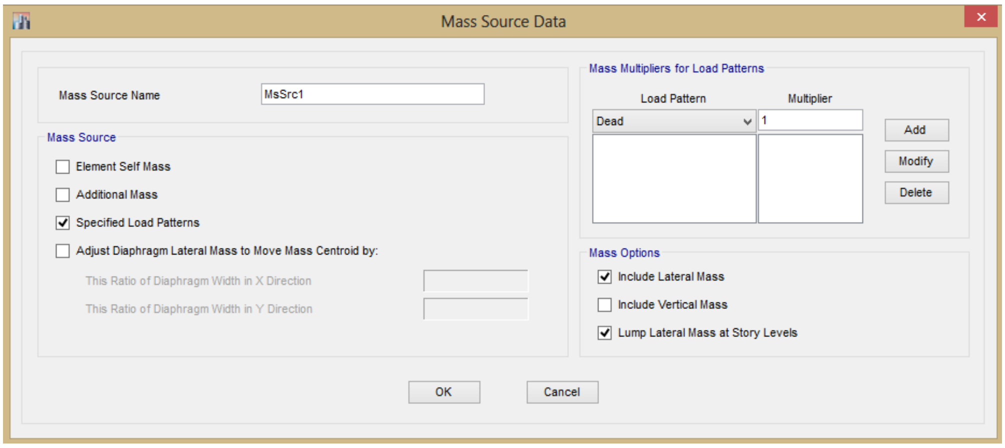

Figure 5.8: Mass source

5.4.1.5 Response spectrum function

The parameters and factors according to codes are inserted into the ETABS 2015 program for drawing response spectrum curve as shown in Figure 5.9 to 5.11.

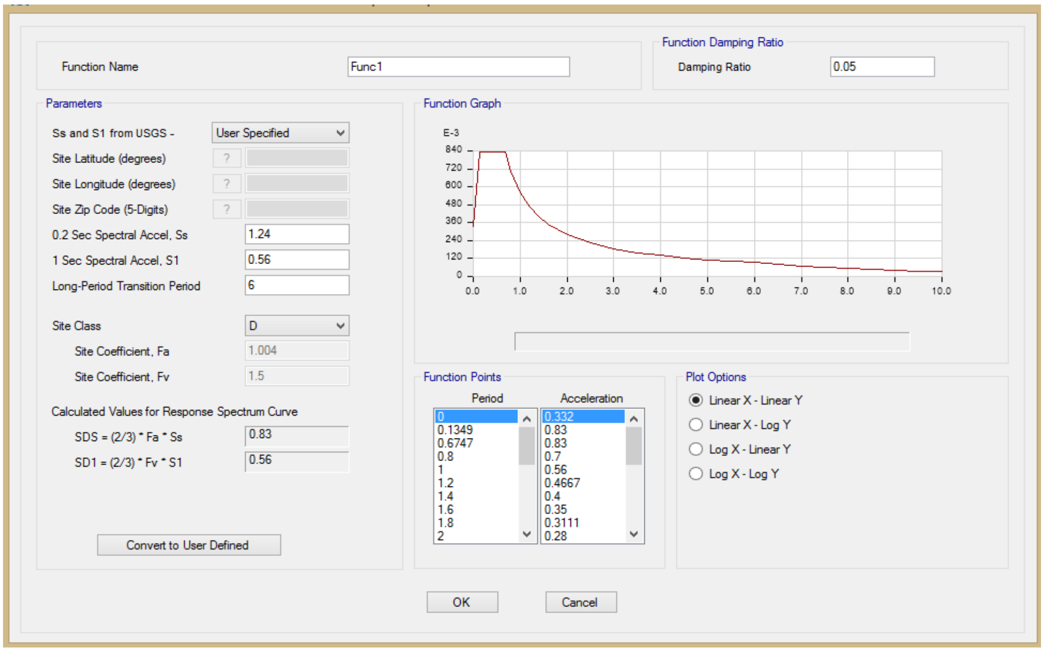

Figure 5.9: Response spectrum function definition according to IBC 2009

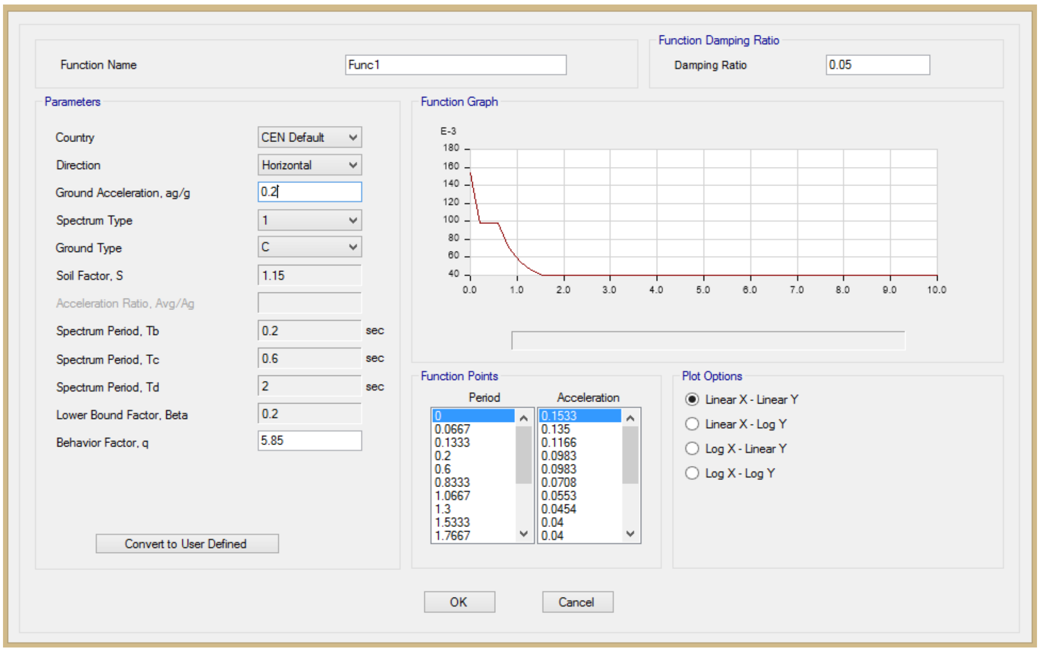

Figure 5.10: Response spectrum function definition according to EC 8

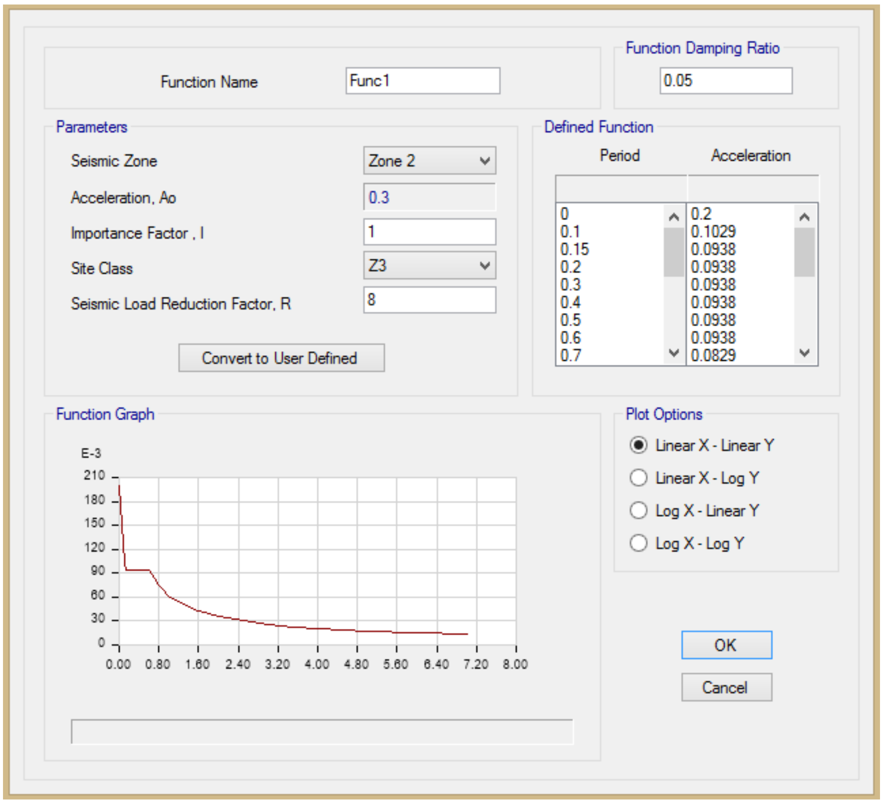

Figure 5.11: Response spectrum function definition according to NCSC 2015

5.4.1.6 Equivalent lateral force

The factors and parameters are inserted into the ETABS 2015 program according to codes as shown in Figure 5.12 to 5.14.

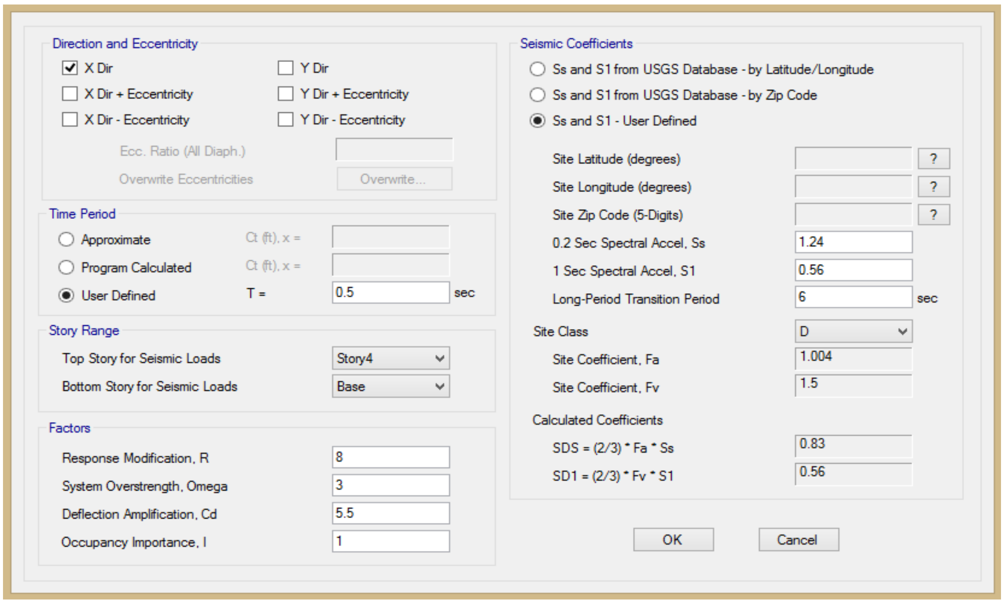

Figure 5.12: Equivalent lateral force according to IBC 2009

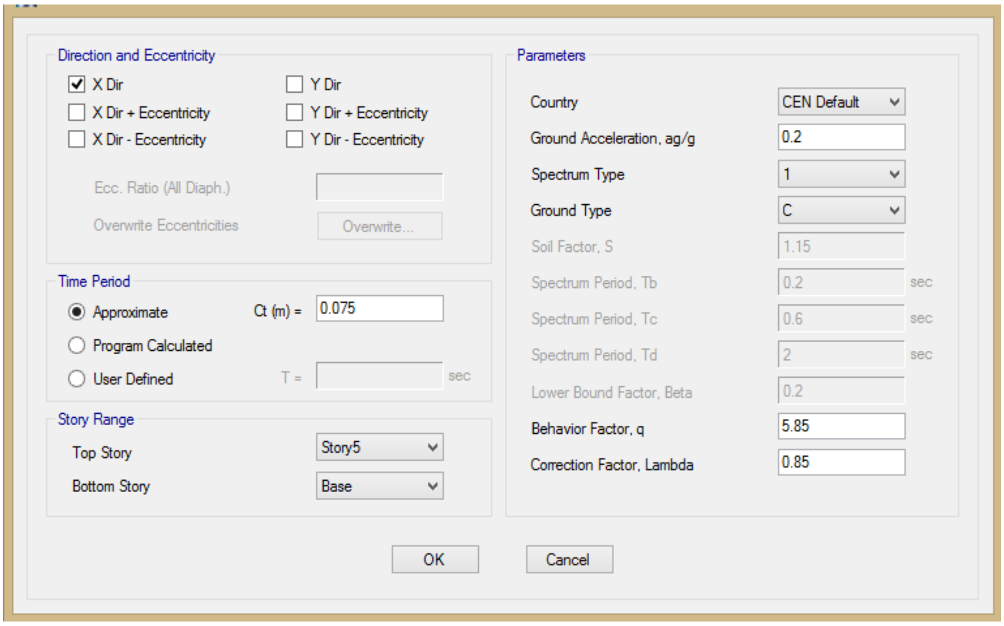

Figure 5.13: Equivalent lateral force according to EC 8

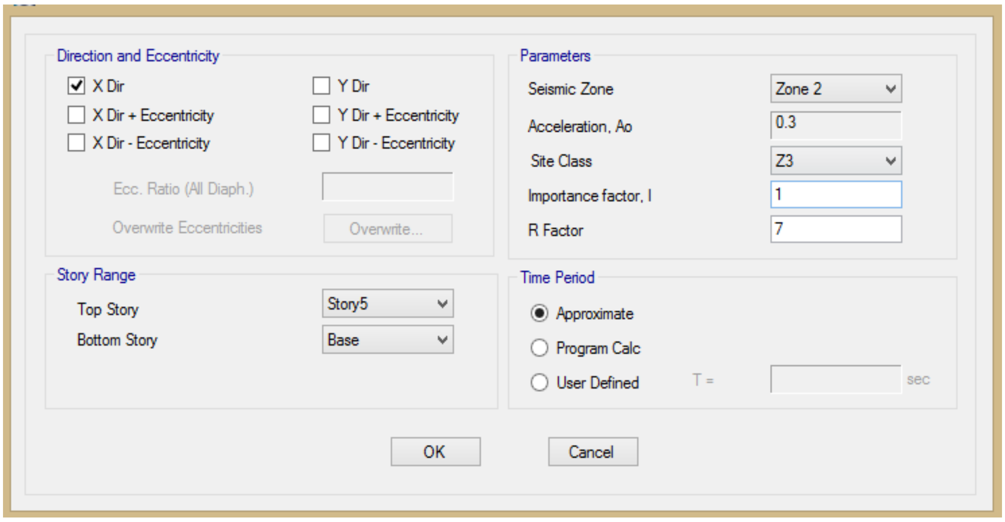

Figure 5.14: Equivalent lateral force according to NCSC 2015

5.4.1.7 Scale factor

The response spectrum scale factor is defined by the following:

$$Sf = \frac{I\,g}{R} \tag{5.1}$$

where

g = Gravity (9.81 m/sec^2 for kN-m)

I = Importance factor

R = Response Modification or behavior factor

Following analysis, users should check the base shear due to all modes, reported in the base reaction table. If the dynamic base shear reported is extra than 85% of the static base shear, no additional action is wanted. Conversely, if dynamic base shear is less than 85% of the static base shear, then the scale factor must be adjusted to the dynamic base shear equivalent 85% of the static base shear. In this case, the new scale factor should be

$$Sf_N = \frac{I\,g}{R} * 0.85 * \frac{ELFM}{RSM} \tag{3.2}$$

The analysis should then be repeated with new scale factor specified from the previous formula (ASCE, 2006).

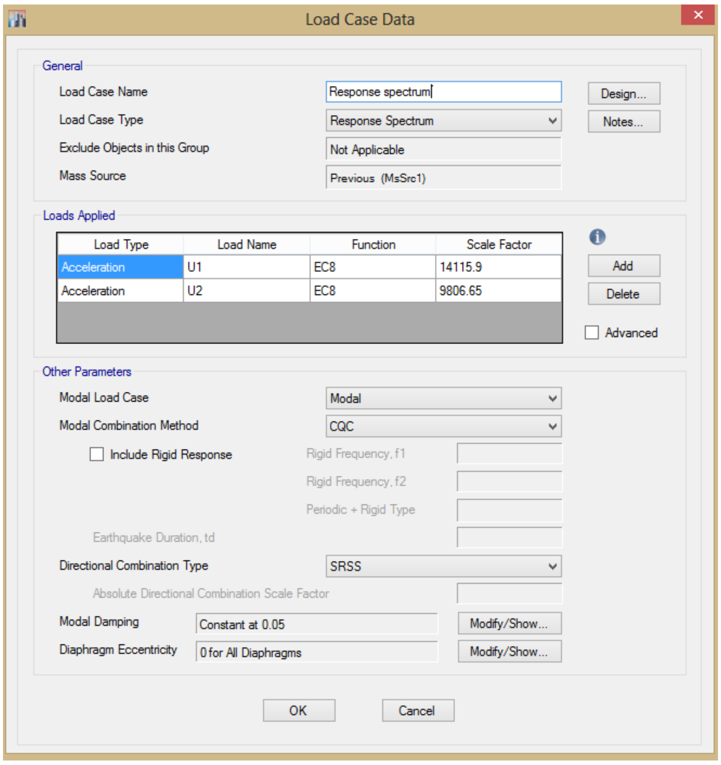

Figure 5.15: Scale factor

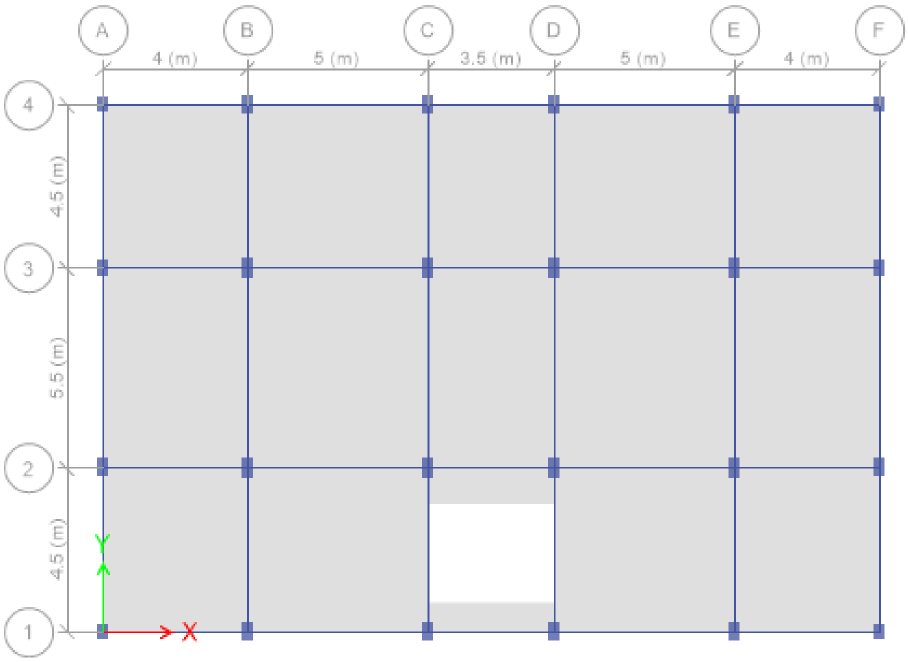

Figure 5.16: Floor plan for five story moment-resisting frame by ETABS

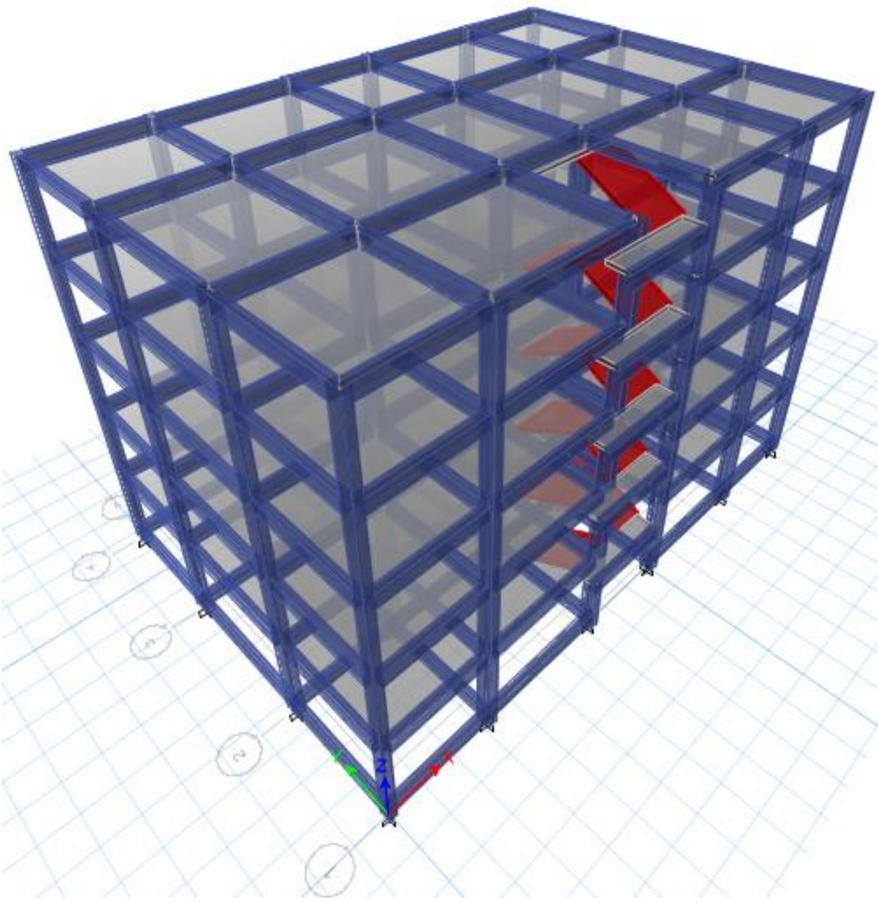

Figure 5.17: Three dimensional view for five story moment-resisting frame by ETABS

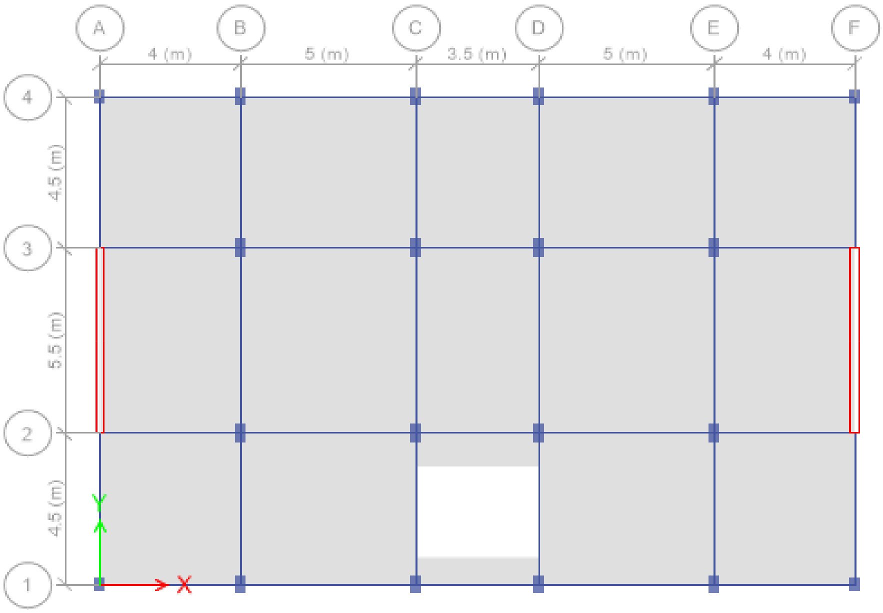

Figure 5.18: Floor plan for five story moment-resisting frame with shear wall by ETABS

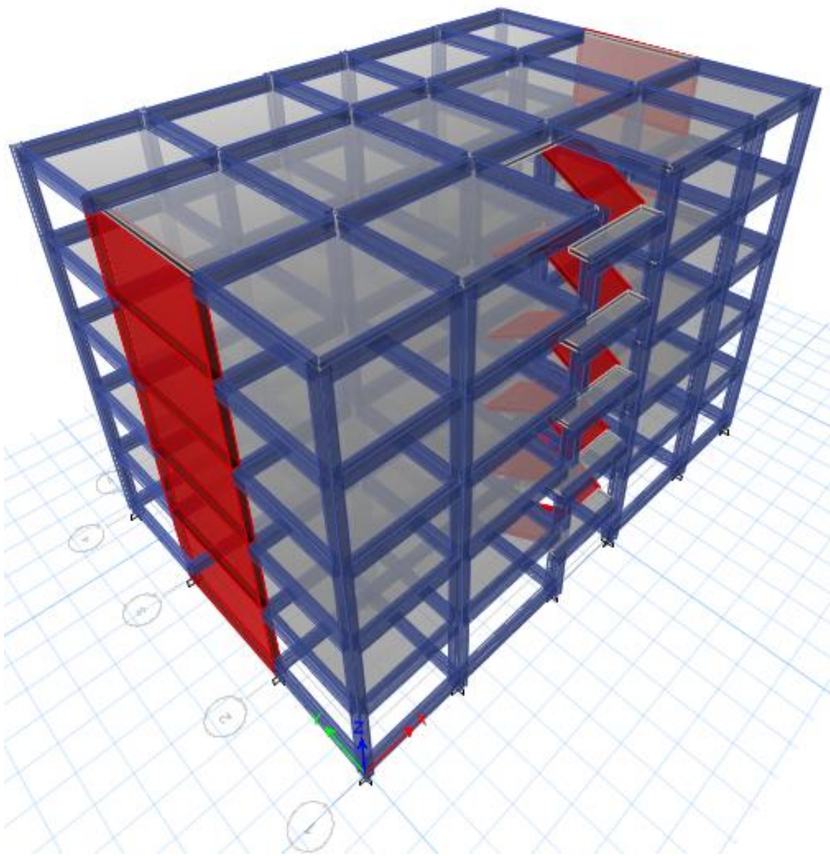

Figure 5.19: Three dimensional view for five story moment-resisting frame with shear wall by ETABS

CHAPTER 6

RESULTS AND DISCUSSIONS

6.1 Overview

In this chapter, results obtained from both static and dynamic analysis are presented in the form of base shear, story shear, displacement, axial force and bending moment for selected columns in the light of three different codes. Graphical representation has been shown in the following figures from 6.1 - 6.20.

6.2 Base Shear

The total base shear using ELFM and RSM for two types (moment-resisting frame and moment-resisting frame with shear wall) of RC framed structures has been adopted for different codes for the case region. Figures 6.1 - 6.4 shows the obtained base shear in the x and y directions, using both static and dynamic procedures.

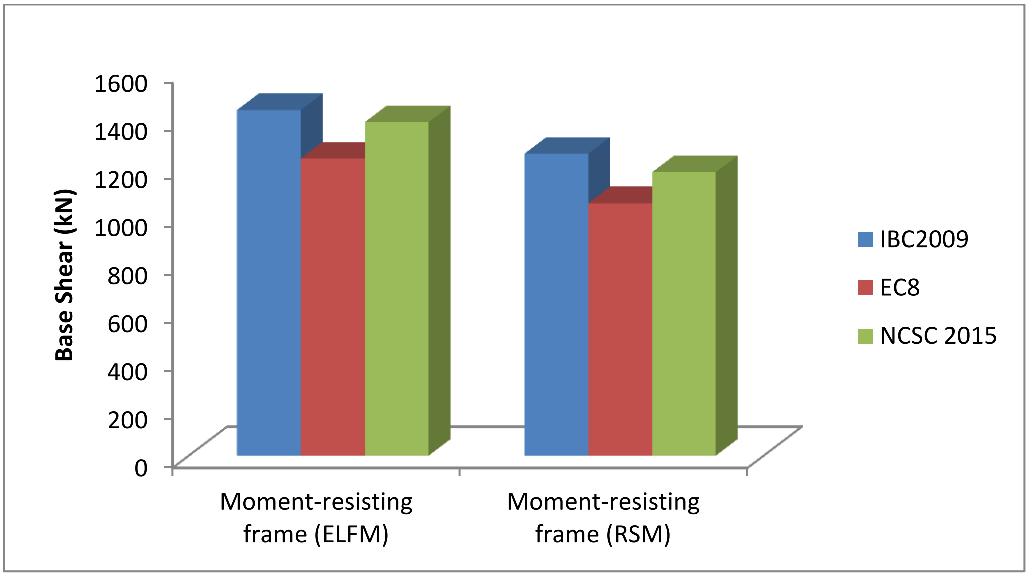

Figure 6.1: Total base shear MRF in x-direction

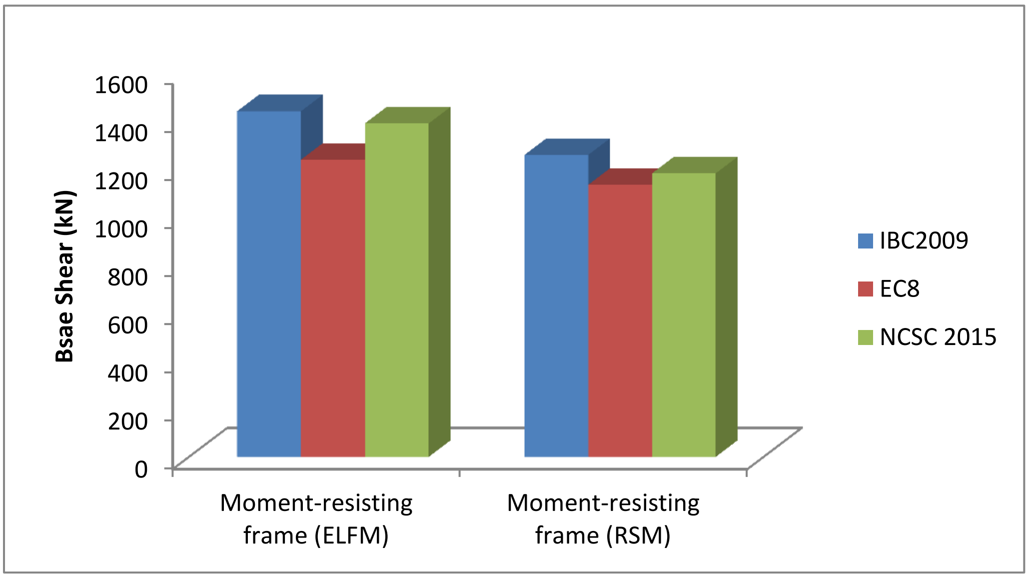

Figure 6.2: Total base shear MRF in y-direction

In the x-direction the total base shear obtained from EC 8 using ELFM (five stories MRF) are about 14% and 11% less than IBC 2009 and NCSC 2015, respectively. The same observation was made in the y-direction.

In the x-direction the total base shear obtained from EC 8 using RSM (five stories MRF) are about 14.4% and 10.9% less than IBC 2009 and NCSC 2015, respectively. In the y-direction the total base shear obtained from EC 8 using RSM (five stories MRF) are about 9.9% and 4% less than IBC 2009 and NCSC 2015 respectively.

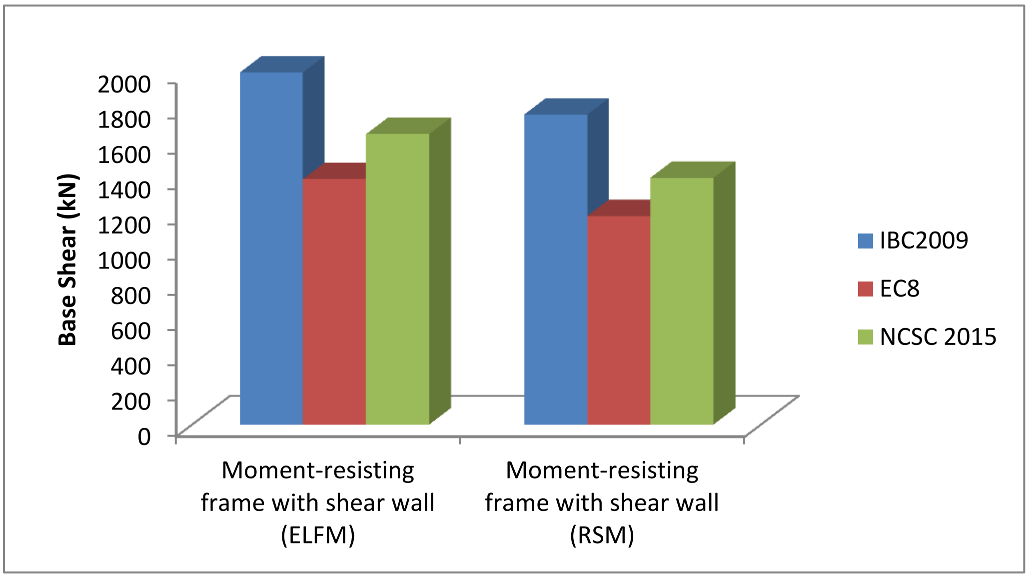

Figure 6.3: The base shear MRF+SW in x-direction

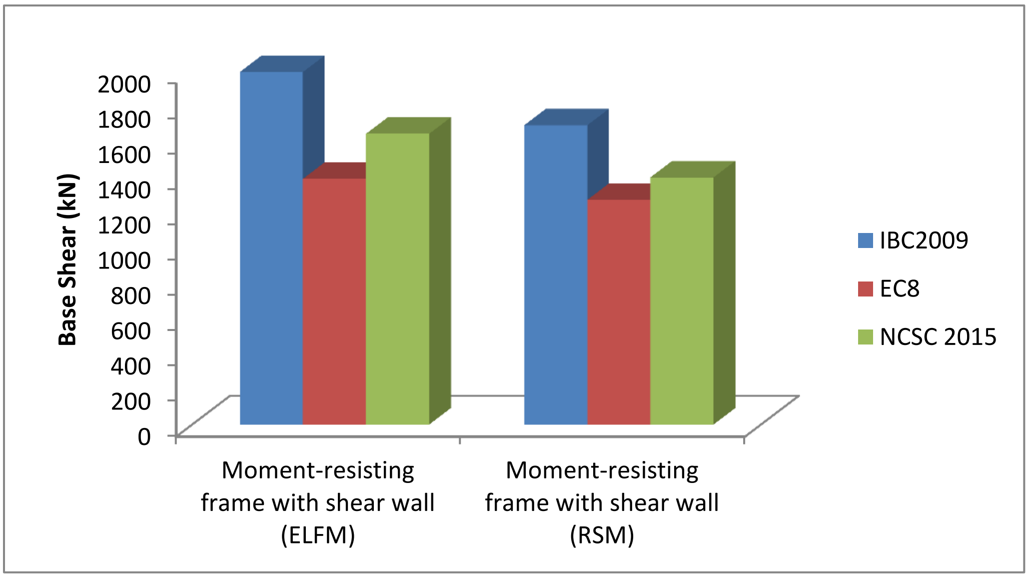

Figure 6.4: The base shear MRF+SW in y-direction

In the x-direction the total base shear obtained from EC 8 using ELFM (five stories MRF+SW) are about 30.3% and 15.5% less than IBC 2009 and NCSC 2015, respectively. The same observation was made in the y-direction.

In the x-direction the total base shear obtained from EC 8 using RSM (five stories MRF+SW) are about 32.7% and 15.5% less than IBC 2009 and NCSC 2015, respectively. In the y-direction the total base shear obtained from EC 8 using RSM (five stories MRF+SW) are about 24.9% and 9.1% less than IBC 2009 and NCSC 2015, respectively.

6.3 Story Shear

The graphs for story shear versus story height are made for three codes and for all RC frames (MRF and MRF+SW) using ELFM and RSM. The results are shown in the following Figures from 6.5 - 6.12.

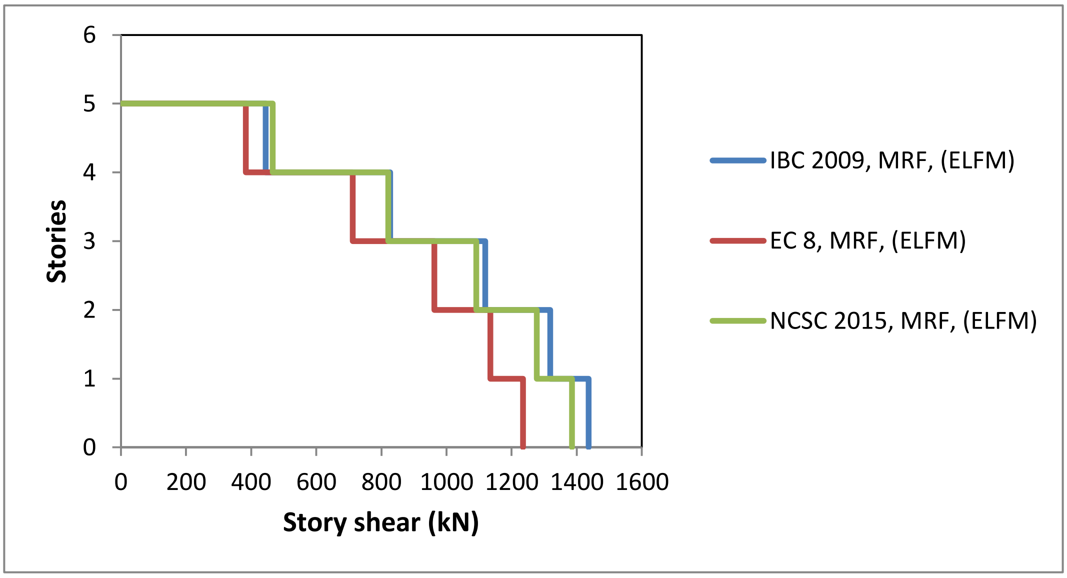

Figure 6.5: The story shear MRF using ELFM in x-direction

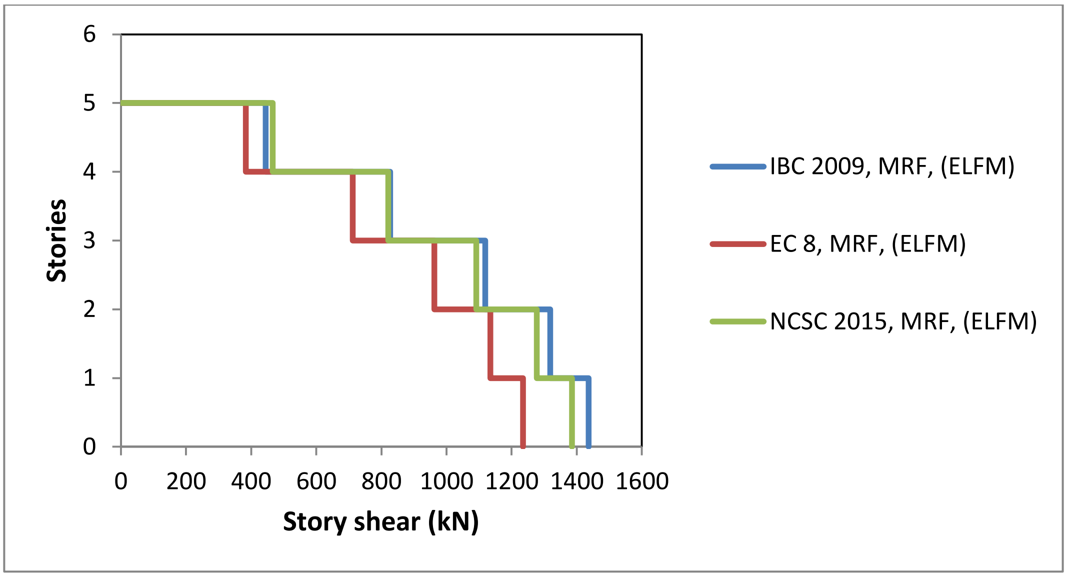

Figure 6.6: The story shear MRF using ELFM in y-direction

Comparing the story shear using ELFM (MRF) in x-direction for EC 8 and IBC 2009, the values are 14% from base to top. For EC 8 and NCSC2015, the values vary from 11% to 18% from base to top. The same observation was made in the y-direction.

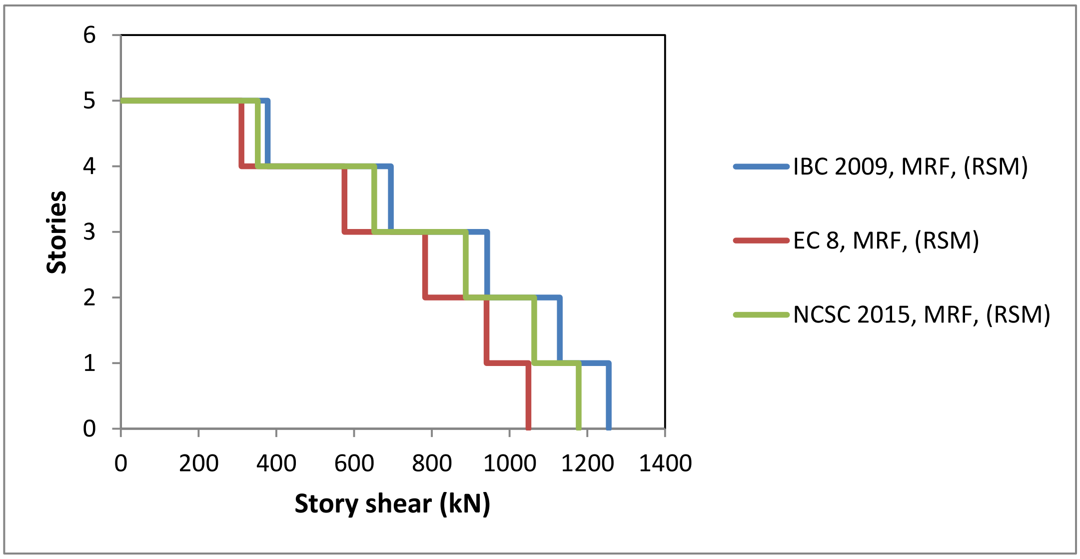

Figure 6.7: The story shear MRF using RSM in x-direction

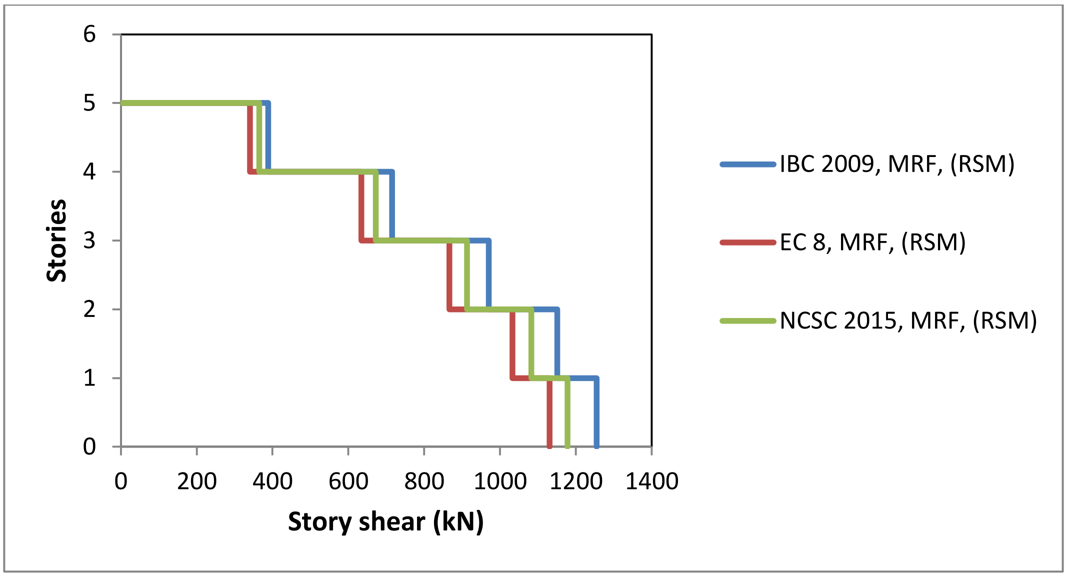

Figure 6.8: The story shear MRF using RSM in y-direction

Comparing the story shear using RSM (MRF) in x-direction for EC 8 and IBC 2009, the values vary from 16% to 18% from base to top. For EC 8 and NCSC 2015, the values vary from 11% to 12% from base to top. Comparing the story shear using RSM (MRF) in y-direction for EC 8 and IBC 2009, the values varies from 10% to 12% from base to top. For EC 8 and NCSC 2015, the values vary from 4% to 7% from base to top.

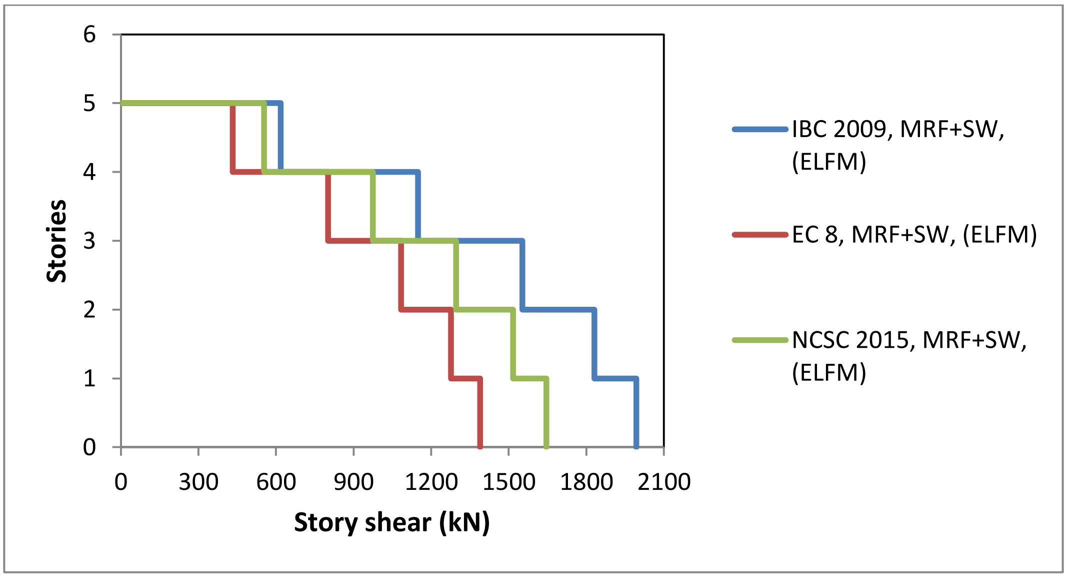

Figure 6.9: The story shear MRF+ SW using ELFM in x-direction

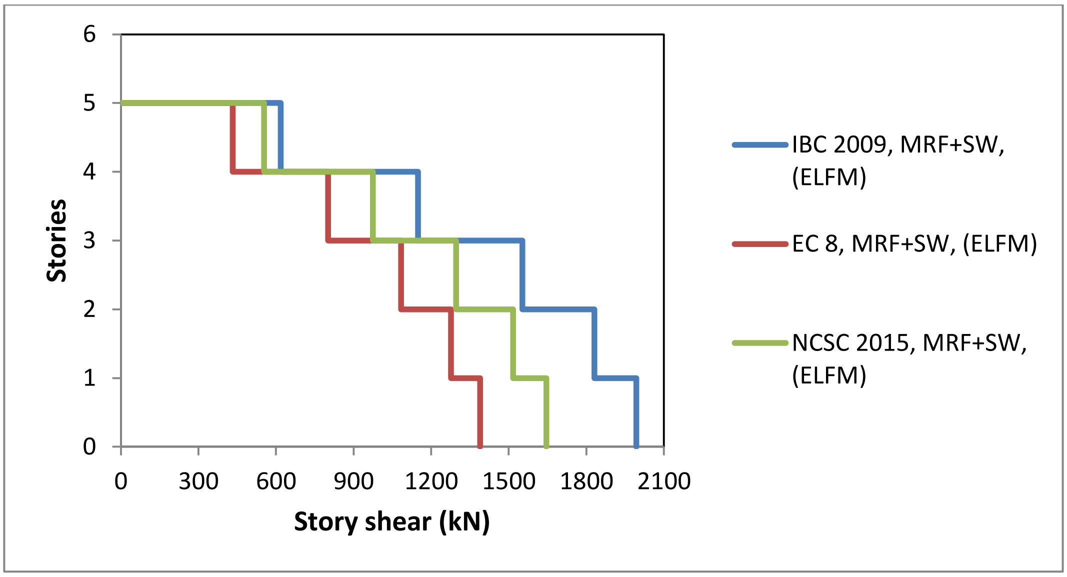

Figure 6.10: The story shear MRF+ SW using ELFM in y-direction

Comparing the story shear using ELFM (MRF+SW) in x-direction for EC 8 and IBC 2009, the values are 30% from base to top. For EC8 and NCSC 2015, the values vary from 16% to 22% from base to top. The same observation was made in the y-direction.

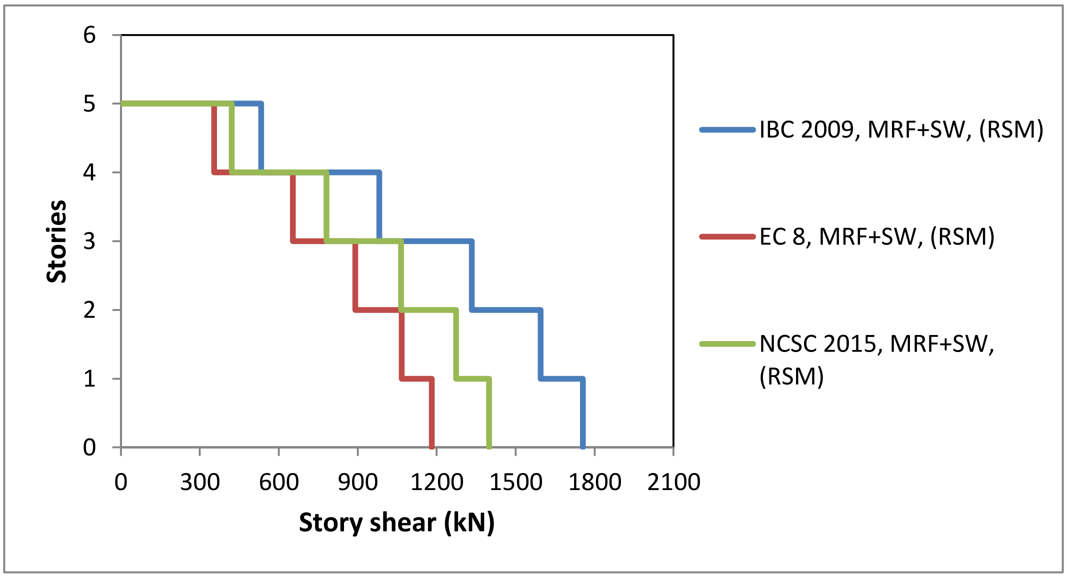

Figure 6.11: The story shear MRF+ SW using RSM in x-direction

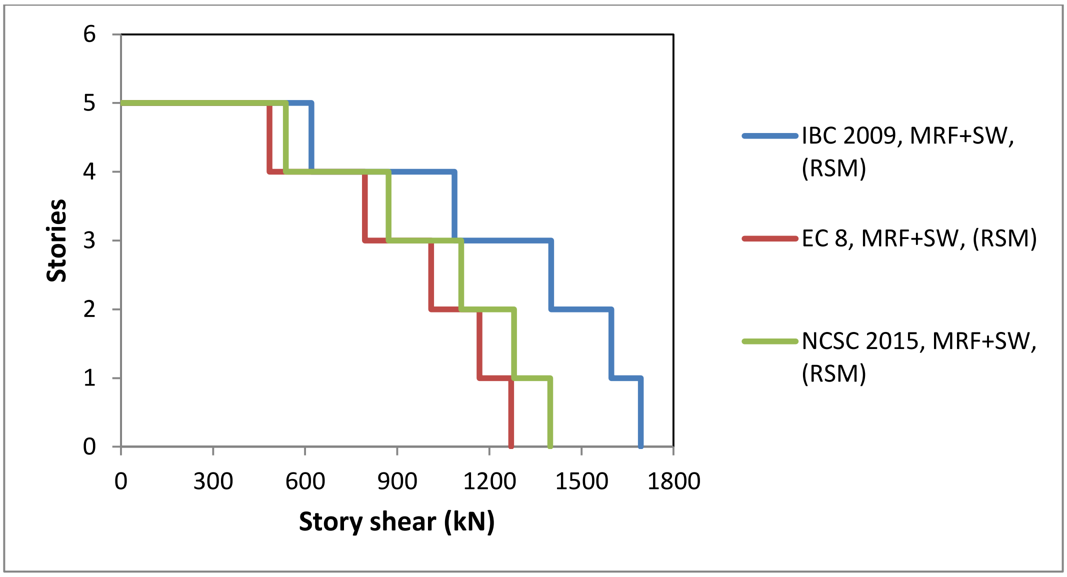

Figure 6.12: The story shear MRF+ SW using RSM in y-direction

Comparing the story shear using RSM (MRF+SW) in x-direction for EC 8 and IBC 2009, the values varies from 33% to 34% from base to top. For EC8 and NCSC 2015, the values are 16% from base to top. Comparing the story shear using RSM (MRF+SW) in y-direction for EC 8 and IBC 2009, the values vary from 25% to 22% from base to top. For EC 8 and NCSC 2015, the values vary from 9% to 10% from base to top.

6.4 Displacement

The graphs for displacement with story height for every codes are increased along the building height for all RC frames (MRF and MRF+SW) using ELFM and RSM. The results are shown in the following Figures from 6.13 - 6.16.

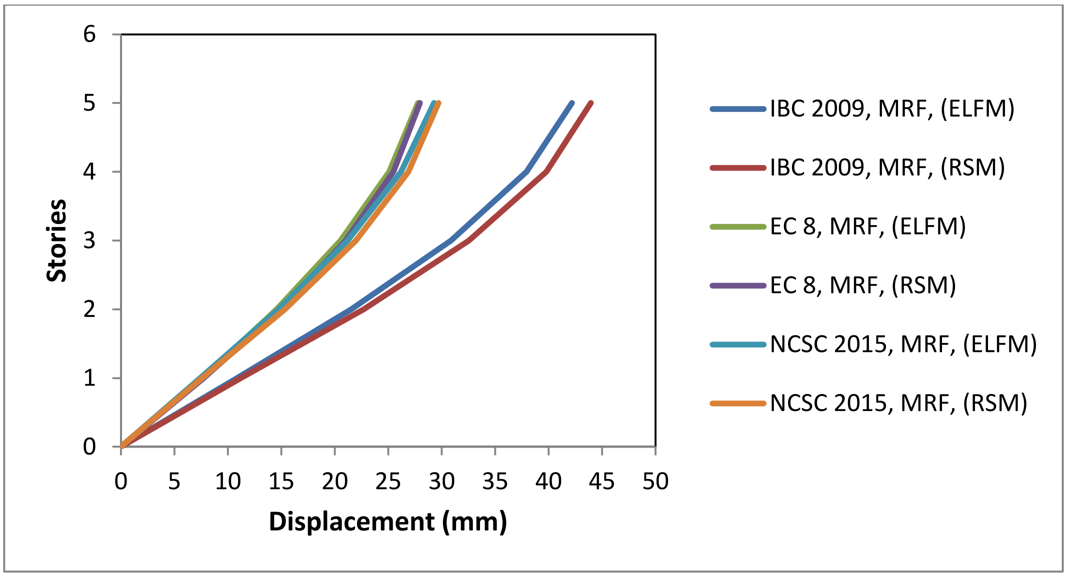

Figure 6.13: The displacement MRF in x-direction

In the x-direction the maximum displacement from EC 8 using ELFM (five stories MRF) is about 34.1% and 5.1% less than IBC 2009 and NCSC 2015, respectively. The maximum displacement from EC 8 using RSM (five stories MRF) is about 36.2% and 5.7% less than IBC 2009 and NCSC 2015, respectively.

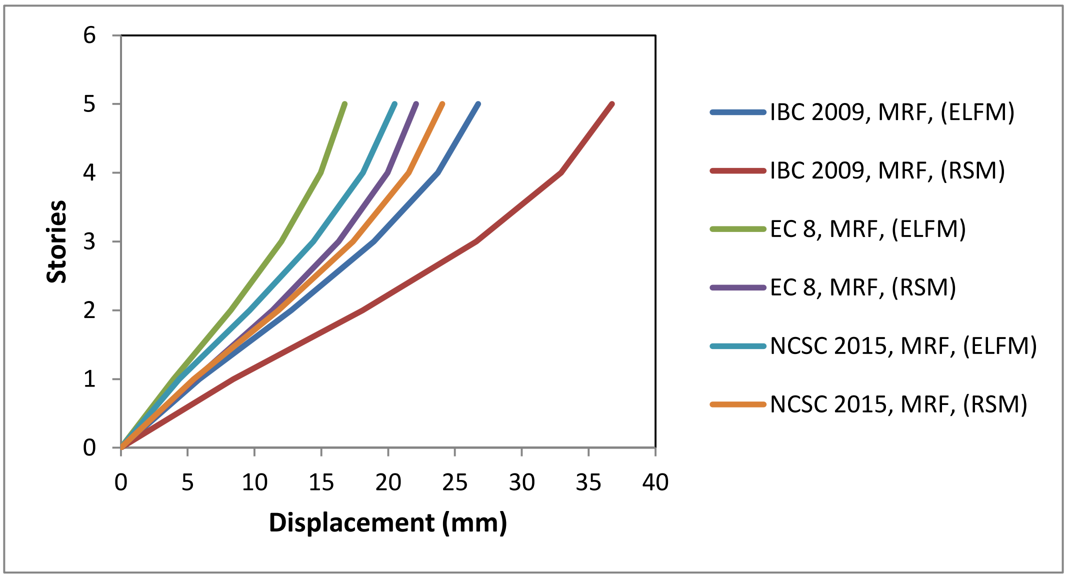

Figure 6.14: The displacement MRF in y-direction

In the y-direction the maximum displacement from EC 8 using ELFM (five stories MRF) is about 37.4% and 18.5% less than IBC 2009 and NCSC 2015, respectively. The maximum displacement from EC 8 using RSM (five stories MRF) is about 39.8% and 8.1% less than IBC 2009 and NCSC 2015, respectively.

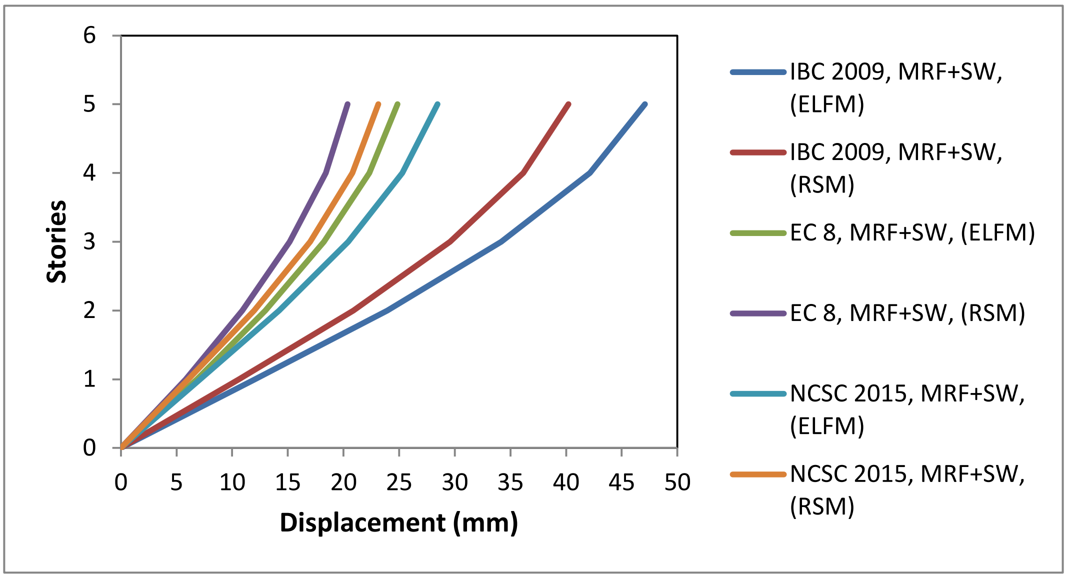

Figure 6.15: The displacement MRF+SW in x-direction

In the x-direction the maximum displacement from EC 8 using ELFM (five stories MRF+SW) is about 47.3% and 12.7% less than IBC 2009 and NCSC 2015, respectively. The maximum displacement from EC 8 using RSM (five stories MRF+SW) is about 49.2% and 12.1% less than IBC 2009 and NCSC 2015, respectively.

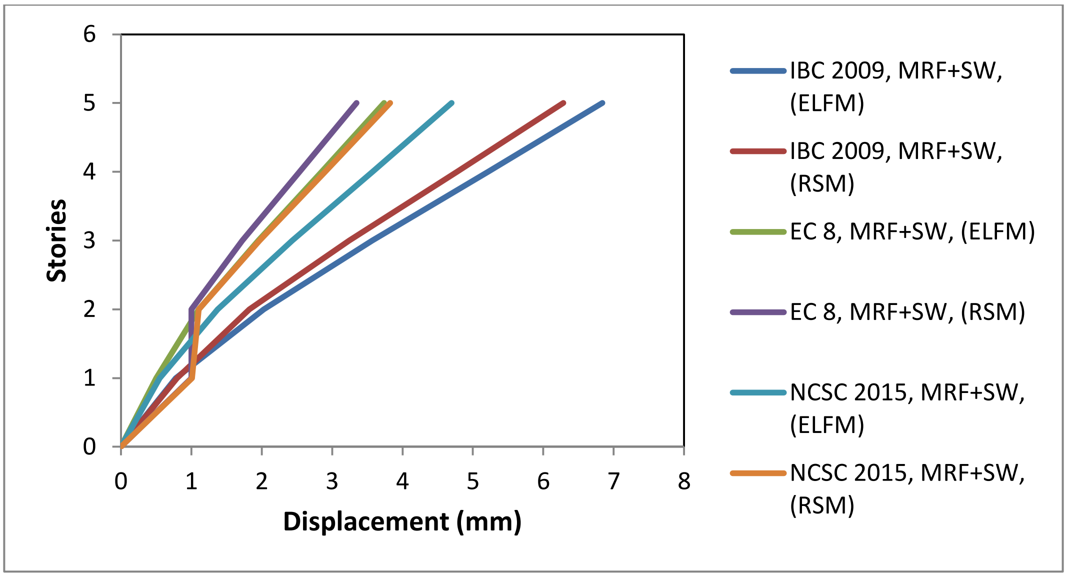

Figure 6.16: The displacement MRF+SW in y-direction

In the y-direction the maximum displacement from EC 8 using ELFM (five stories MRE+SW) is about 45.6% and 21.3% less than IBC 2009 and NCSC 2015, respectively. The maximum displacement from EC 8 using RSM (five stories MRF+SW) is about 47.6% and 13.2% less than IBC 2009 and NCSC 2015, respectively.

6.5 Axial Force in Columns

The column axial forces are analysed and chosen the columns C1(corner), C2(exterior), C3(interior) using ELFM and RSM for two types (MRF and MRF+SW) of RC framed structures has been adopted for different codes for the case region. Figures 6.17 - 6.19 shows the results of column axial forces for corner, external and interior column.

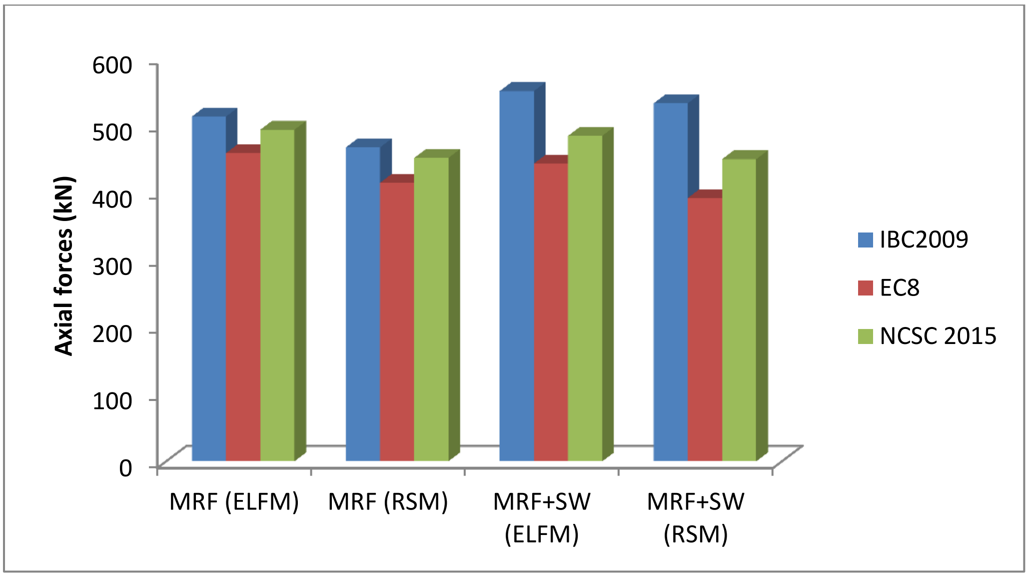

Figure 6.17: Axial forces for column C1 (corner)

The total axial force for C1, obtained from EC 8 (five story MRF) using ELFM is about 10.6% and 7.0% less than IBC 2009 and NCSC 2015, respectively. The total axial force for C1, obtained from EC 8 (five story MRF) using RSM is about 11.3% and 8.2% less than IBC 2009 and NCSC 2015, respectively. The total axial force for C1, obtained from EC 8 (five story MRF+SW) using ELFM is about 19.6% and 8.5% less than IBC 2009 and NCSC 2015, respectively. The total axial force for C1, obtained from EC 8 (five story MRF+SW) using RSM is about 26.5% and 12.8% less than IBC 2009 and NCSC 2015, respectively.

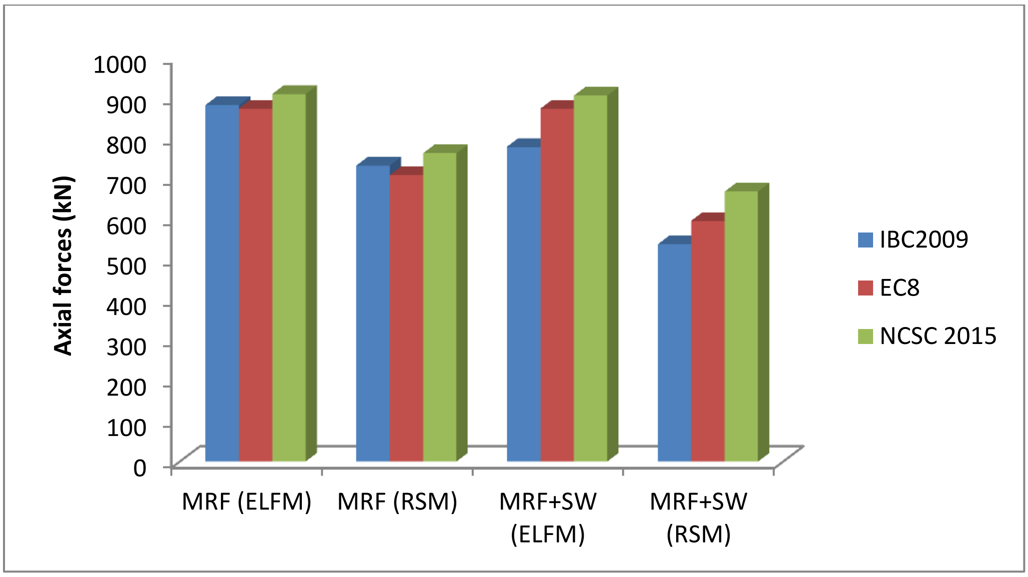

Figure 6.18: Axial forces for column C2 (exterior)

The total axial force for C2, obtained from EC 8 (five story MRF) using ELFM is about 1% and 4% less than IBC 2009 and NCSC 2015, respectively. The total axial force for C2, obtained from EC 8 (five story MRF) using RSM is about 3.2% and 7.1% less than IBC 2009 and NCSC 2015, respectively. The total axial force for C2, obtained from NCSC2015 (five story MRF+SW) using ELFM is about 3.5% and 14% higher than EC8 and IBC2009, respectively. The total axial force for C2, obtained from NCSC2015 (five story MRF+SW) using RSM is about 10.8% and 19.5% higher than EC8 and IBC2009, respectively.

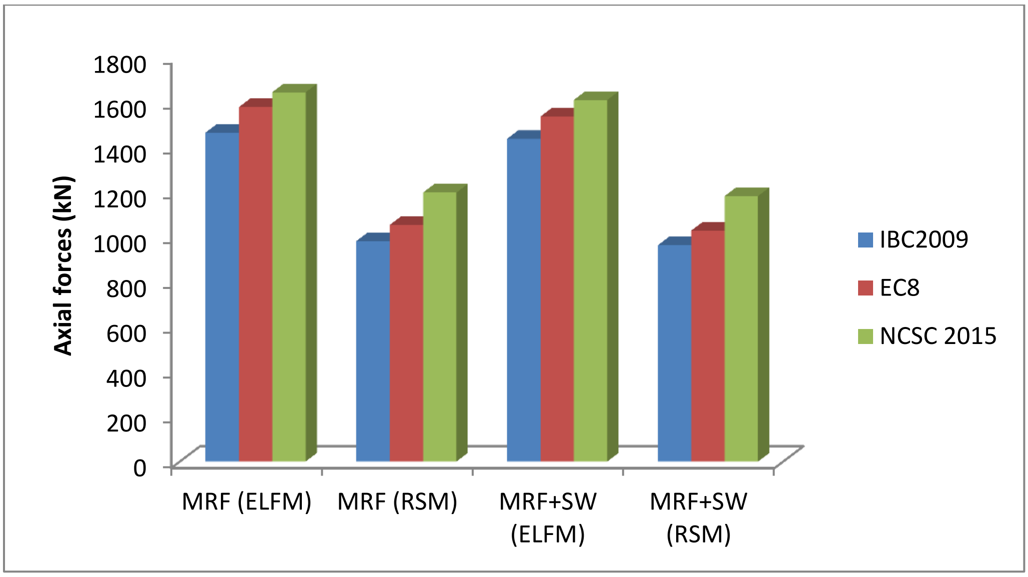

Figure 6.19: Axial forces for column C3 (interior)

The total axial force for C3, obtained from NCSC2015 (five story MRF) using ELFM is about 4% and 11% higher than EC8 and IBC2009, respectively. The total axial force for C3, obtained from NCSC2015 (five story MRF) using RSM is about 12% and 18.1% higher than EC8 and IBC2009, respectively. The total axial force for C3, obtained from NCSC2015 (five story MRF+SW) using ELFM is about 4.6% and 10.8% higher than EC8 and IBC2009, respectively. The total axial force for C3, obtained from NCSC2015 (five story MRF+SW) using RSM is about 13% and 18.6% higher than EC8 and IBC2009, respectively.

6.6 Bending Moments in Columns

The maximum column bending moments are analysed and chosen columns C1 (corner), C2 (exterior), C3 (interior) using ELFM and RSM for two types (MRF and MRF+SW) of RC framed structures has been adopted for different codes. Figures 6.20 - 6.22 shows the results of maximum column bending moments for corner, external and interior column.

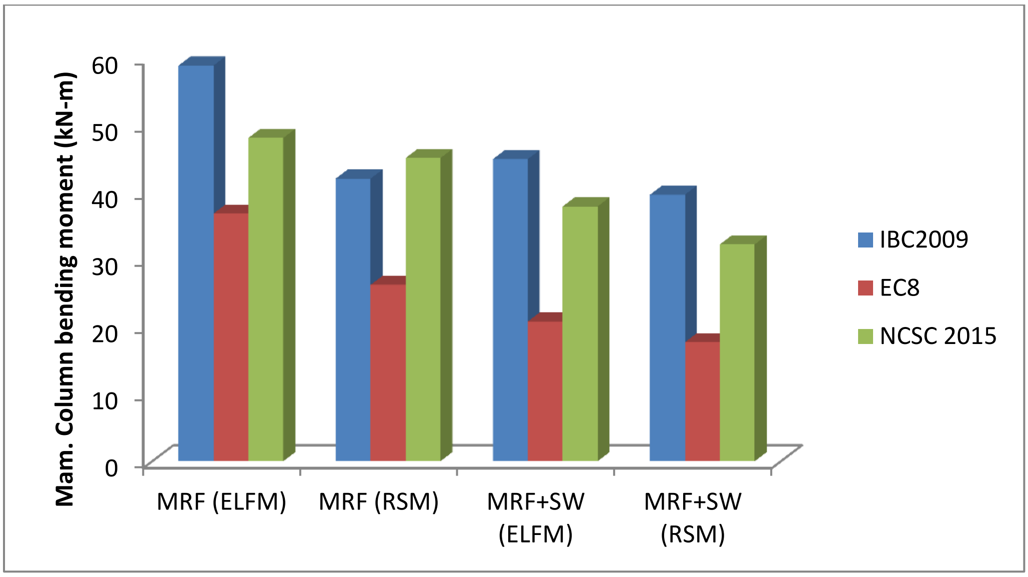

Figure 6.20: Maximum bending moments for column C1 (corner)

The maximum column bending moment for C1, obtained from EC 8 (five story MRF) using ELFM is about 37.4% and 23.4% less than IBC 2009 and NCSC 2015, respectively. The maximum column bending moment for C1, obtained from EC 8 (five story MRF) using RSM is about 37.6% and 41.8% less than IBC 2009 and NCSC 2015, respectively. The maximum column bending moment for C1, obtained from EC 8 (five story MRF+SW) using ELFM is about 53.7% and 45.1% less than IBC 2009 and NCSC 2015, respectively. The maximum column bending moment for C1, obtained from EC 8 (five story MRF+SW) using RSM is about 55.3% and 45.2% less than IBC 2009 and NCSC 2015, respectively.

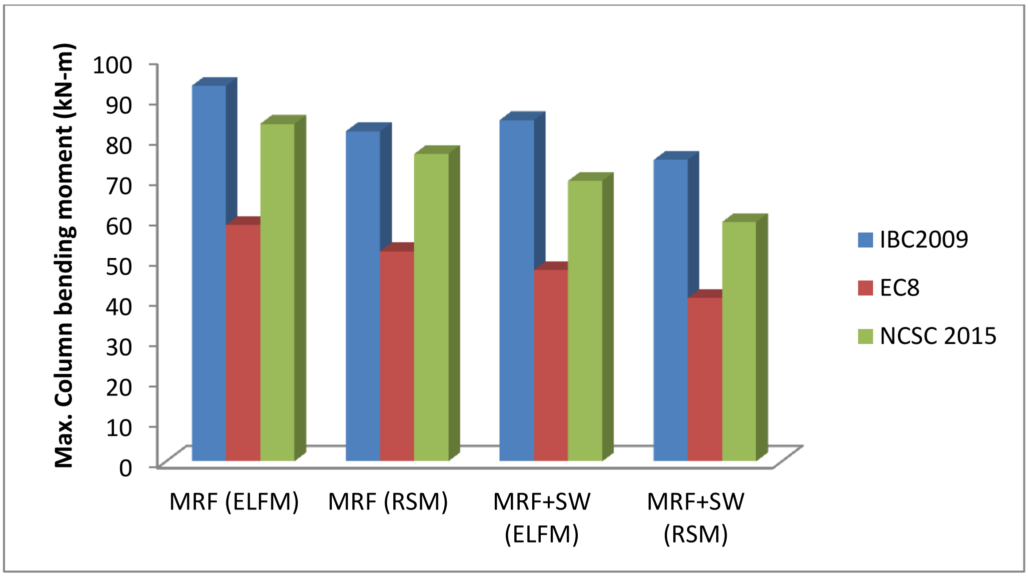

Figure 6.21: Maximum bending moments for column C2 (exterior)

The maximum column bending moment for C2, obtained from EC 8 (five story MRF) using ELFM is about 37.0% and 29.9% less than IBC 2009 and NCSC 2015, respectively. The maximum column bending moment for C2, obtained from EC 8 (five story MRF) using RSM is about 36.4% and 31.8% less than IBC 2009 and NCSC 2015, respectively. The maximum column bending moment for C2, obtained from EC 8 (five story MRF+SW) using ELFM is about 44.0% and 31.8% less than IBC 2009 and NCSC 2015, respectively. The maximum column bending moment for C2, obtained from EC 8 (five story MRF+SW) using RSM is about 45.8% and 31.7% less than IBC 2009 and NCSC 2015, respectively.

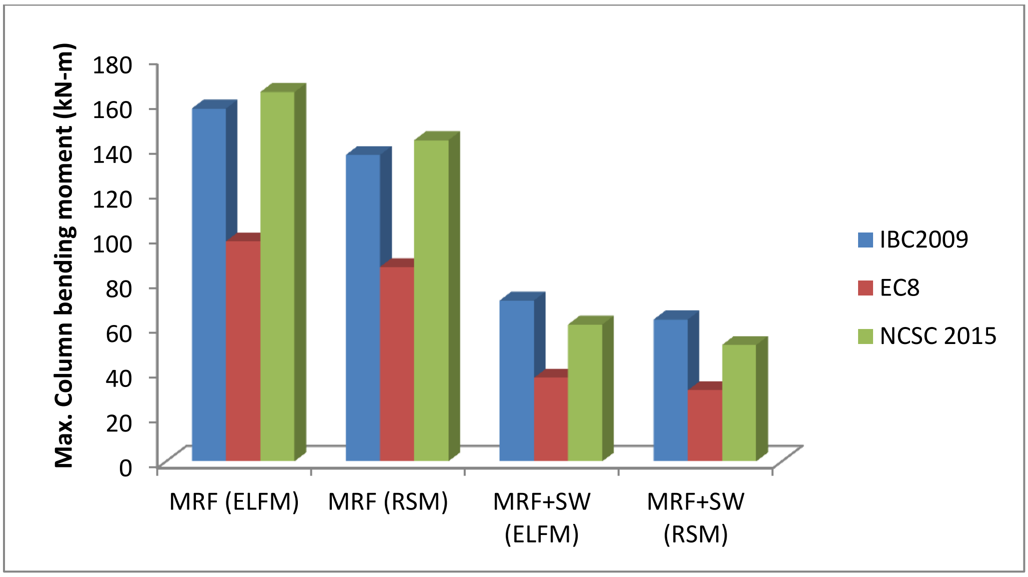

Figure 6.22: Maximum bending moments for column C3 (interior)

The maximum column bending moment for C3, obtained from EC 8 (five story MRF) using ELFM is about 37% and 40% less than IBC 2009 and NCSC 2015, respectively. The maximum column bending moment for C3, obtained from EC 8 (five story MRF) using RSM is about 36.5% and 39.5% less than IBC 2009 and NCSC 2015, respectively. The maximum column bending moment for C3, obtained from EC 8 (five story MRF+SW) using ELFM is about 48.2% and 39.0% less than IBC 2009 and NCSC 2015, respectively. The maximum column bending moment for C3, obtained from EC 8 (five story MRF+SW) using RSM is about 49.9% and 38.9% less than IBC 2009 and NCSC 2015, respectively.

CHAPTER 7

CONCLUSIONS

Based on the obtained results from the analysis of RC frame building in Lefkoşa city, it can be concluded that:

1. Base shear as per three codes;
    - Base shear as per NCSC 2015 for MRF using ELFM is remain in between compared to other codes.
    - Base shear as per NCSC 2015 for MRF using RSM is remain in between compared to other codes.
    - Base shear as per NCSC 2015 for MRF+SW using ELFM is remain in between compared to other codes.
    - Base shear as per NCSC 2015 for MRF+SW using RSM is remain in between compared to other codes.
2. Story shear as per three codes;
    - Story shear as per NCSC 2015 for MRF using ELFM is remain in between compared to other codes.
    - Story shear as per NCSC 2015 for MRF using RSM is remain in between compared to other codes.
    - Story shear as per NCSC 2015 for MRF+SW using ELFM is remain in between compared to other codes.
    - Story shear as per NCSC 2015 for MRF+SW using RSM is remain in between compared to other codes.
3. Displacements for top of the building as per three codes;
    - Displacement as per NCSC 2015 for MRF using ELFM is remain in between compared to other codes.
    - Displacement as per NCSC 2015 for MRF using RSM is remain in between compared to other codes.

- Displacement as per NCSC 2015 for MRF+SW using ELFM is remain in between compared to other codes.
- Displacement as per NCSC 2015 for MRF+SW using RSM is remain in between compared to other codes.

4. Axial forces for selected columns;
    - Axial forces as per NCSC 2015 for MRF using ELFM for corner column (C1) is remain in between compared to other codes. For exterior (C2) and interior (C3) columns are maximally compared to other codes.
    - Axial forces as per NCSC 2015 for MRF using RSM for corner column (C1) is remain in between compared to other codes. For exterior (C2) and interior (C3) columns are maximally compared to other codes.
    - Axial forces as per NCSC 2015 for MRF+SW using ELFM for corner column (C1) is remain in between compared to other codes. For exterior (C2) and interior (C3) columns are maximally compared to other codes.
    - Axial forces as per NCSC 2015 for MRF+SW using RSM for corner column (C1) is remain in between compared to other codes. For exterior (C2) and interior (C3) columns are maximally compared to other codes.
    - The different load combination factor in each seismic code has an effect on total axial load in columns.
5. Bending moments for selected columns;
    - Bending moment as per NCSC 2015 for MRF using ELFM is remain in between compared to other codes.
    - Bending moment as per NCSC 2015 for MRF using RSM is remain in between compared to other codes.
    - Bending moment as per NCSC 2015 for MRF+SW using ELFM is remain in between compared to other codes.
    - Bending moment as per NCSC 2015 for MRF+SW using RSM is remain in between compared to other codes.
6. The ELFM work well for low-rise to mid-rise buildings. However, the results of ELFM are approximately uneconomic. Because the design parameters such as base

shear, story shear, displacement, axial force and bending moment values are higher than RSM.

7. The results obtained from MRF and MRF+SW for both ELFM and RSM analysis are presented in the form of base shear, story shear, displacement, axial forces and bending moments for selected columns for three different codes. The first edition of northern Cyprus seismic code which is named as NCSC 2015 provides a generic level of safety that incorporate in well established code.
8. The current earthquake code NCSC 2015 used in northern part of the island is actually based on Turkish earthquake code 2007 (TEC2007). Also, harmonization with international structural design practice should be improved in north Cyprus where the National annexes exists for whole island already.
9. Moreover, to generalize the results obtained, an analysis on satisfactory number of buildings with different number of storeys and irregularities should be made.

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APPENDICES

APPENDIX 1

MINISTRY OF LABOUR AND SOCIAL SECURITY BUILDING, SOIL INVESTIGATION REPORT.

KKTC

Çevre ve Doğal Kaynaklar Bakanlığı

Jeoloji ve Maden Dairesi

'ÇALIŞMA VE SOSYAL GÜVENLİK BAKANLIĞI BİNASI' ZEMİN ETÜD RAPORU

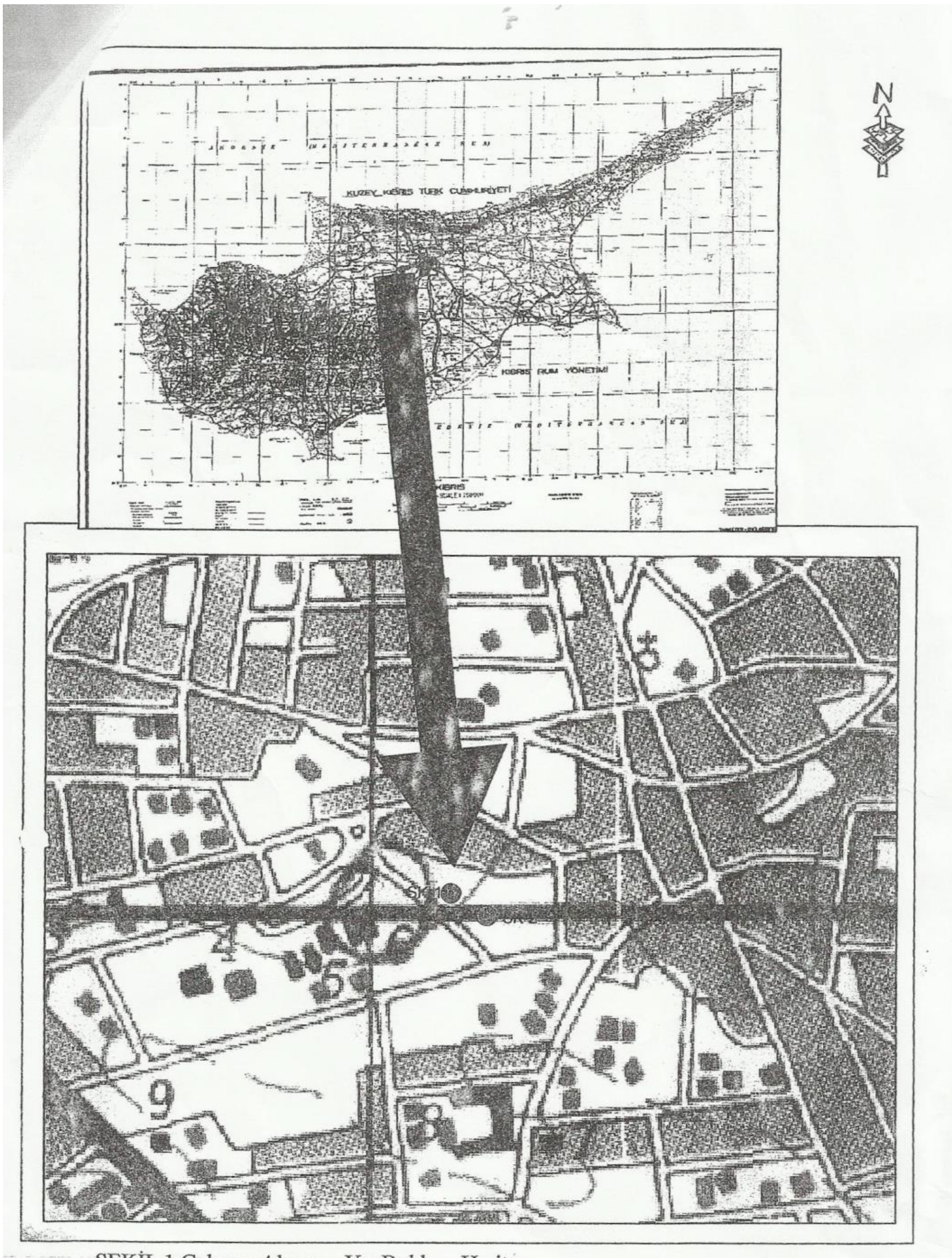
Hazırlayan

Hatice KAŞER

Jeoloji Yüksek Mühendisi

KASIM, 2007

Lefkoşa



ŞEKİL 1 Çalışma Alanının Yer Bulduru Haritası

Temel Zeminine Ait Mekanik Parametreler:

1. İnceleme alanı **2. Derece** deprem bölgesindedir.
2. Temel zemin grubu (**C**)
3. Yerel zemin sınıfı (Z_2)
4. Zeminin spekturum karakteristik periyotları $\mathbf{T_A=0.15sn}$,
   $\mathbf{T_B=0.40sn}$
5. Kayma dalgası hızı **200-400 m/s** alınabilir.
6. Deprem hesaplarında kullanılacak etkin yer ivmesi katsayısı $\mathbf{A_0 = 0.30}$ 'dur.
7. Yatak Katsayısı $\mathbf{K_0=2000\ ton/m^3}$
8. Bina önem katsayısı **I=1.4**

Hatice Kaşer

Jeoloji Yüksek Mühendisi

APPENDIX 2

ETABS RESULTS ACCORDING TO IBC 2009

1- ETABS results for moment-resisting frame

This calculation presents the automatically generated lateral seismic loads for load pattern Eq x according to IBC 2009, as calculated by ETABS.

Direction and Eccentricity

Direction = X

Structural Period

Period Calculation Method = User Specified

User Period $T = 0.5 \text{ sec}$

Long-Period Transition Period, T_L [ASCE 11.4.5] $T_L = 6 \text{ sec}$

Factors and Coefficients

Response Modification Factor, R [ASCE Table 12.2-1] $R = 8$

System Overstrength Factor, Ω_0 [ASCE Table 12.2-1] $\Omega_0 = 3$

Deflection Amplification Factor, C_d [ASCE Table 12.2-1] $C_d = 5.5$

Importance Factor, I [ASCE Table 11.5-1] $I = 1$

Ss and S1 Source = User Specified

Mapped MCE Spectral Response Acceleration, S_s [ASCE 11.4.1] $S_s = 1.24g$

Mapped MCE Spectral Response Acceleration, S_1 [ASCE 11.4.1] $S_1 = 0.56g$

Site Class [ASCE Table 20.3-1] = D - Stiff Soil

Site Coefficient, F_a [ASCE Table 11.4-1] $F_a = 1.004$

Site Coefficient, F_v [ASCE Table 11.4-2] $F_v = 1.5$

Seismic Response

MCE Spectral Response Acceleration, S_{MS} [ASCE 11.4.3, Eq. 11.4-1]	$S_{MS} = F_a S_S$	$S_{MS} = 1.24496g$
MCE Spectral Response Acceleration, S_{M1} [ASCE 11.4.3, Eq. 11.4-2]	$S_{M1} = F_v S_1$	$S_{M1} = 0.84g$
Design Spectral Response Acceleration, S_{DS} [ASCE 11.4.4, Eq. 11.4-3]	$S_{DS} = \frac{2}{3} S_{MS}$	$S_{DS} = 0.829973g$
Design Spectral Response Acceleration, S_{D1} [ASCE 11.4.4, Eq. 11.4-4]	$S_{D1} = \frac{2}{3} S_{M1}$	$S_{D1} = 0.56g$

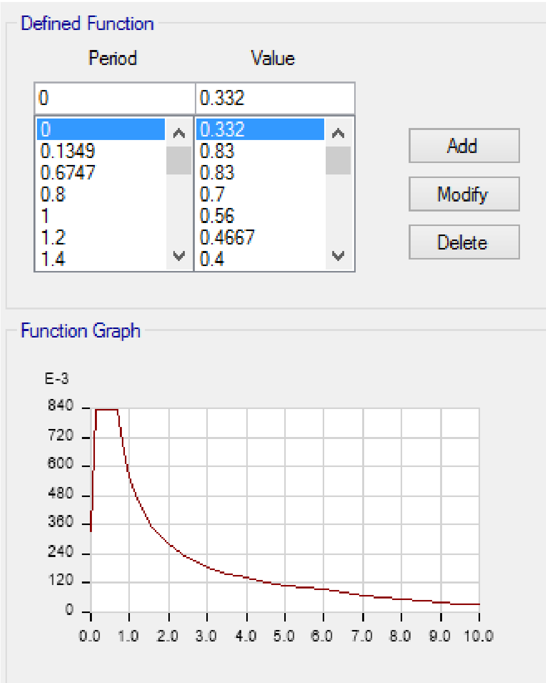


Figure 2.1: Design response spectrum curve according to IBC 2009

Equivalent Lateral Forces

Seismic Response Coefficient, C_S [ASCE 12.8.1.1, Eq. 12.8-2] $C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$

[ASCE 12.8.1.1, Eq. 12.8-3] $C_{S,max} = \frac{S_{D1}}{T\left(\frac{R}{I}\right)}$

[ASCE 12.8.1.1, Eq. 12.8-5] $C_{S,min} = 0.01$

[ASCE 12.8.1.1, Eq. 12.8-6] $C_{S,min} = 0.5 \frac{S_1}{\left(\frac{R}{I}\right)} \text{for} S_1 = 0.6g$

$$C_{S,min} \leq C_s \leq C_{S,max}$$

Calculated Base Shear

Direction	Period Used (sec)	C_s	W (kN)	V (kN)
X	0.5	0.103747	13841.5743	1436.0172

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story5	15.6	444.8919	0
Story4	12.6	382.3655	0
Story3	9.6	291.3261	0
Story2	6.6	200.2867	0
Story1	3.6	117.147	0
Base	0	0	0

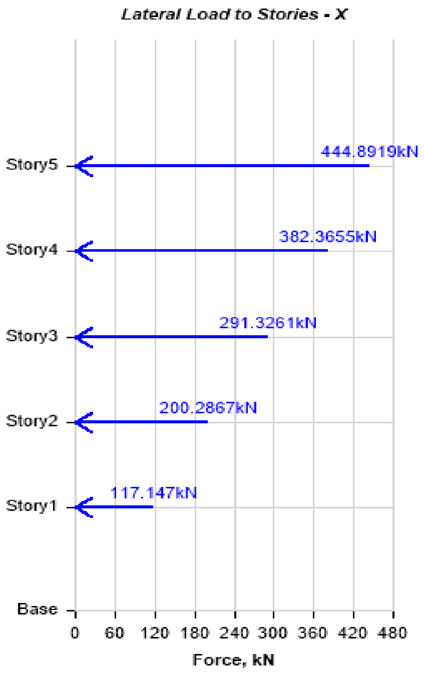


Figure 2.2: Lateral force acting in each stories in x-direction

This calculation presents the automatically generated lateral seismic loads for load pattern Eq y according to IBC 2009, as calculated by ETABS.

Direction and Eccentricity

Direction = Y

Structural Period

Period Calculation Method = User Specified

User Period $T = 0.5\ \mathrm{sec}$

Long-Period Transition Period, T_L [ASCE 11.4.5] $T_L = 6\ \mathrm{sec}$

Factors and Coefficients

Response Modification Factor, R [ASCE Table 12.2-1] $R = 8$

System Overstrength Factor, Ω_0 [ASCE Table 12.2-1] $\Omega_0 = 3$

Deflection Amplification Factor, C_d [ASCE Table 12.2-1] $C_d = 5.5$

Importance Factor, I [ASCE Table 11.5-1] $I = 1$

Ss and S1 Source = User Specified

Mapped MCE Spectral Response Acceleration, S_s [ASCE 11.4.1] $S_s = 1.24g$

Mapped MCE Spectral Response Acceleration, S_1 [ASCE 11.4.1] $S_1 = 0.56g$

Site Class [ASCE Table 20.3-1] = D - Stiff Soil

Site Coefficient, F_a [ASCE Table 11.4-1] $F_a = 1.004$

Site Coefficient, F_v [ASCE Table 11.4-2] $F_v = 1.5$

Seismic Response

MCE Spectral Response Acceleration, S_{MS} [ASCE 11.4.3, Eq. 11.4-1]	$S_{MS} = F_a S_S$	$S_{MS} = 1.24496g$
MCE Spectral Response Acceleration, S_{M1} [ASCE 11.4.3, Eq. 11.4-2]	$S_{M1} = F_v S_1$	$S_{M1} = 0.84g$
Design Spectral Response Acceleration, S_{DS} [ASCE 11.4.4, Eq. 11.4-3]	$S_{DS} = \frac{2}{3} S_{MS}$	$S_{DS} = 0.829973g$
Design Spectral Response Acceleration, S_{D1} [ASCE 11.4.4, Eq. 11.4-4]	$S_{D1} = \frac{2}{3} S_{M1}$	$S_{D1} = 0.56g$

Equivalent Lateral Forces

Seismic Response Coefficient, C_S [ASCE 12.8.1.1, Eq. 12.8-2] $C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$

[ASCE 12.8.1.1, Eq. 12.8-3] $$C_{S,max} = \frac{S_{D1}}{T(\frac{R}{I})}$$

[ASCE 12.8.1.1, Eq. 12.8-5] $$C_{S,min} = 0.01$$

[ASCE 12.8.1.1, Eq. 12.8-6] $$C_{S,min} = 0.5 \frac{S_1}{(\frac{R}{I})} \text{for} S_1 = 0.6g$$

$$C_{S,min} \le C_s \le C_{S,max}$$

Calculated Base Shear

Direction	Period Used (sec)	C_s	W (kN)	V (kN)
Y	0.5	0.103747	13841.5743	1436.0172

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story5	15.6	0	444.8919
Story4	12.6	0	382.3655
Story3	9.6	0	291.3261
Story2	6.6	0	200.2867
Story1	3.6	0	117.147
Base	0	0	0

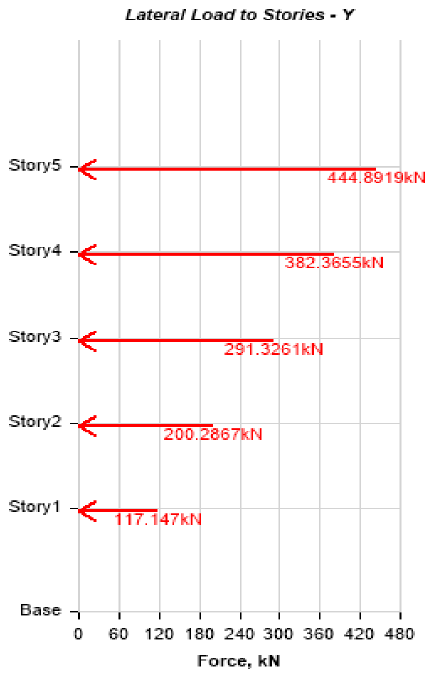


Figure 2.3: Lateral force acting in each stories in y-direction

2- ETABS results for moment-resisting frame with shear wall

This calculation presents the automatically generated lateral seismic loads for load pattern Eq x according to IBC 2009, as calculated by ETABS.

Direction and Eccentricity

Direction = X

Structural Period

Period Calculation Method = User Specified

User Period	$T = 0.5\ sec$
Long-Period Transition Period, T_L [ASCE 11.4.5]	$T_L = 6\ sec$

Factors and Coefficients

Response Modification Factor, R [ASCE Table 12.2-1]	$R = 6$
System Overstrength Factor, Ω_0 [ASCE Table 12.2-1]	$\Omega_0 = 2.5$
Deflection Amplification Factor, C_d [ASCE Table 12.2-1]	$C_d = 5$
Importance Factor, I [ASCE Table 11.5-1]	$I = 1$

Ss and S1 Source = User Specified

Mapped MCE Spectral Response Acceleration, S_s [ASCE 11.4.1]	$S_s = 1.24g$
Mapped MCE Spectral Response Acceleration, S_1 [ASCE 11.4.1]	$S_1 = 0.56g$

Site Class [ASCE Table 20.3-1] = D - Stiff Soil

Site Coefficient, F_a [ASCE Table 11.4-1]	$F_a = 1.004$
Site Coefficient, F_v [ASCE Table 11.4-2]	$F_v = 1.5$

Seismic Response

MCE Spectral Response Acceleration, S_{MS} [ASCE 11.4.3, Eq. 11.4-1]	$S_{MS} = F_a S_S$	$S_{MS} = 1.24496g$
MCE Spectral Response Acceleration, S_{M1} [ASCE 11.4.3, Eq. 11.4-2]	$S_{M1} = F_v S_1$	$S_{M1} = 0.84g$
Design Spectral Response Acceleration, S_{DS} [ASCE 11.4.4, Eq. 11.4-3]	$S_{DS} = \frac{2}{3} S_{MS}$	$S_{DS} = 0.829973g$

Design Spectral Response Acceleration, S_{D1} [ASCE 11.4.4, Eq. 11.4-4] $S_{D1} = \frac{2}{3} S_{M1}$ $S_{D1} = 0.56g$

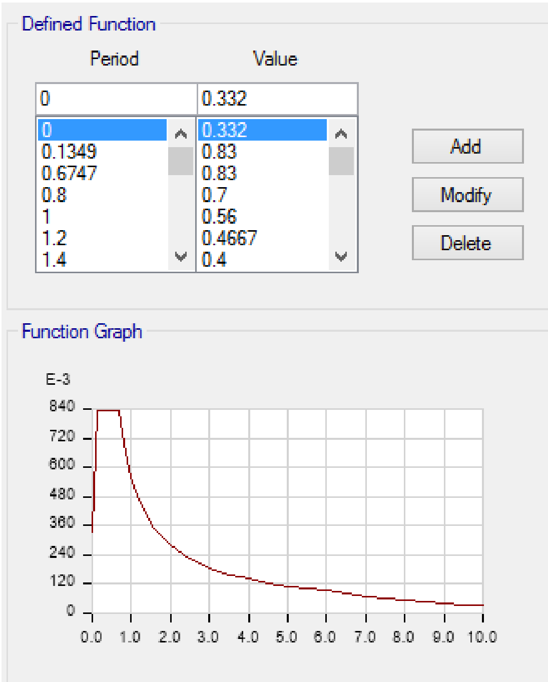


Figure 2.4: Design response spectrum curve according to IBC 2009

Equivalent Lateral Forces

Seismic Response Coefficient, C_S [ASCE 12.8.1.1, Eq. 12.8-2] $$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$$

[ASCE 12.8.1.1, Eq. 12.8-3] $$C_{S,max} = \frac{S_{D1}}{T\left(\frac{R}{I}\right)}$$

[ASCE 12.8.1.1, Eq. 12.8-5] $$C_{S,min} = 0.01$$

[ASCE 12.8.1.1, Eq. 12.8-6] $$C_{S,min} = 0.5\frac{S_1}{\left(\frac{R}{I}\right)} \text{for} S_1 = 0.6g$$

$$C_{S,min} \le C_s \le C_{S,max}$$

Calculated Base Shear

Direction	Period Used (sec)	C_s	W (kN)	V (kN)
X	0.5	0.138329	14415.1785	1994.0356

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story5	15.6	618.4173	0
Story4	12.6	530.6408	0
Story3	9.6	404.2978	0
Story2	6.6	277.9547	0
Story1	3.6	162.725	0
Base	0	0	0

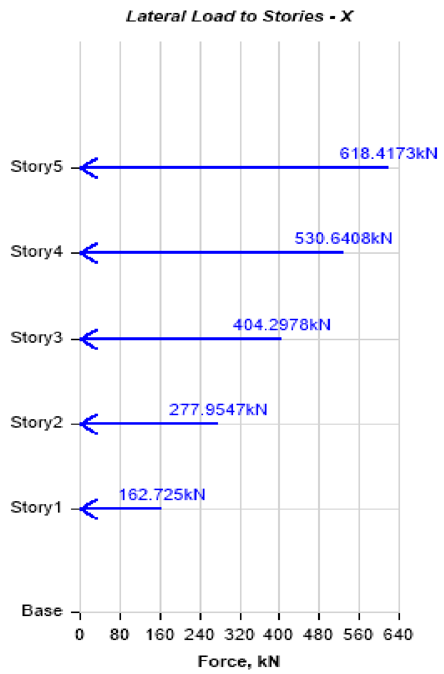


Figure 2.5: Lateral force acting in each stories in x-direction

This calculation presents the automatically generated lateral seismic loads for load pattern Eq y according to IBC 2009, as calculated by ETABS.

Direction and Eccentricity

Direction = Y

Structural Period

Period Calculation Method = User Specified

User Period $T = 0.5\ sec$

Long-Period Transition Period, T_L [ASCE 11.4.5] $T_L = 6\ sec$

Factors and Coefficients

Response Modification Factor, R [ASCE Table 12.2-1] $R = 6$

System Overstrength Factor, Ω_0 [ASCE Table 12.2-1] $\Omega_0 = 2.5$

Deflection Amplification Factor, C_d [ASCE Table 12.2-1] $C_d = 5$

Importance Factor, I [ASCE Table 11.5-1] $I = 1$

Ss and S1 Source = User Specified

Mapped MCE Spectral Response Acceleration, S_s [ASCE 11.4.1] $S_s = 1.24g$

Mapped MCE Spectral Response Acceleration, S_1 [ASCE 11.4.1] $S_1 = 0.56g$

Site Class [ASCE Table 20.3-1] = D - Stiff Soil

Site Coefficient, F_a [ASCE Table 11.4-1] $F_a = 1.004$

Site Coefficient, F_v [ASCE Table 11.4-2] $F_v = 1.5$

Seismic Response

MCE Spectral Response Acceleration, S_{MS} [ASCE 11.4.3, Eq. 11.4-1]	$S_{MS} = F_a S_S$	$S_{MS} = 1.24496g$
MCE Spectral Response Acceleration, S_{M1} [ASCE 11.4.3, Eq. 11.4-2]	$S_{M1} = F_v S_1$	$S_{M1} = 0.84g$
Design Spectral Response Acceleration, S_{DS} [ASCE 11.4.4, Eq. 11.4-3]	$S_{DS} = \frac{2}{3} S_{MS}$	$S_{DS} = 0.829973g$
Design Spectral Response Acceleration, S_{D1} [ASCE 11.4.4, Eq. 11.4-4]	$S_{D1} = \frac{2}{3} S_{M1}$	$S_{D1} = 0.56g$

Equivalent Lateral Forces

Seismic Response Coefficient, C_S [ASCE 12.8.1.1, Eq. 12.8-2] $C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$

[ASCE 12.8.1.1, Eq. 12.8-3] $C_{S,max} = \frac{S_{D1}}{T\left(\frac{R}{I}\right)}$

[ASCE 12.8.1.1, Eq. 12.8-5] $C_{S,min} = 0.01$

[ASCE 12.8.1.1, Eq. 12.8-6] $C_{S,min} = 0.5 \frac{S_1}{(\frac{R}{I})} \text{for} S_1 = 0.6g$

$C_{S,min} \leq C_s \leq C_{S,max}$

Calculated Base Shear

Direction	**Period Used (sec)**	**C_s**	**W (kN)**	**V (kN)**
Y	0.5	0.138329	14415.1785	1994.0356

Story	**Elevation**	**X-Dir**	**Y-Dir**
	m	**kN**	**kN**
Story5	15.6	0	618.4173
Story4	12.6	0	530.6408
Story3	9.6	0	404.2978
Story2	6.6	0	277.9547
Story1	3.6	0	162.725
Base	0	0	0

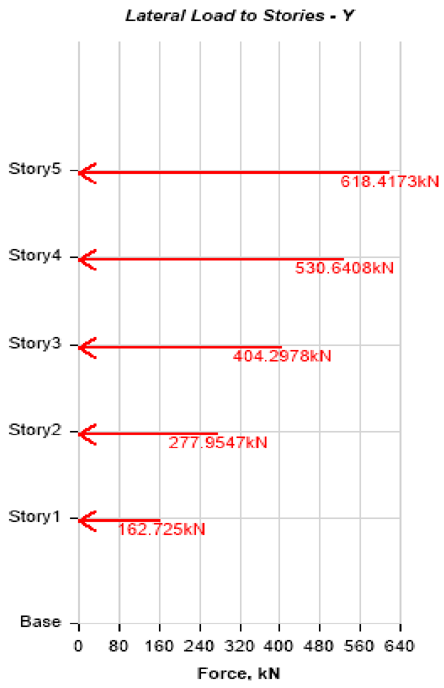


Figure 2.6: Lateral force acting in each stories in y-direction

APPENDIX 3

ETABS RESULTS ACCORDING TO EC 8

1- ETABS results for moment-resisting frame

This calculation presents the automatically generated lateral seismic loads for load pattern Eq x according to EUROCODE8, as calculated by ETABS.

Direction and Eccentricity

Direction = X

Structural Period

Period Calculation Method = Approximate

Coefficient, C_t [EC 4.3.3.2.2]		$C_t = 0.075\mathrm{m}$
Structure Height Above Base, H		$H = 15.6\ \mathrm{m}$
Approximate Fundamental Period, T_1 [EC 4.3.3.2.2(3) Eq. 4.6]	$T_1 = C_t H^{\frac{3}{4}}$	$T_1 = 0.589\ \mathrm{sec}$

Factors and Coefficients

Country = CEN Default

Design Ground Acceleration, a_g	$a_g = 0.2g$

Ground Type [EC Table 3.1] = C

Soil Factor, S [EC Table 3.2]	$S = 1.15$
Constant Acceleration Period Limit, T_B [EC Table 3.2]	$T_B = 0.2\ \mathrm{sec}$
Constant Acceleration Period Limit, T_C [EC Table 3.2]	$T_C = 0.6\ \mathrm{sec}$
Constant Displacement Period Limit, T_D [EC Table 3.2]	$T_D = 2\ \mathrm{sec}$
Lower Bound Factor, β [EC 3.2.2.5(4)]	$\beta_0 = 0.2$
Behavior Factor, q [EC 3.2.2.5(3)]	$q = 5.85$

Seismic Response

Spectral Response Acceleration, Sd($T_{1)}$ [EC 3.2.2.5(4) Eq. 3.13]

$$S_d(T_1) = a_g S[\frac{2}{3} + \frac{T}{T_B}(\frac{2.5}{q} - \frac{2}{3})]\ \mathrm{for}\ T \leq T_B$$

$$= a_g S \frac{2.5}{q}\ \mathrm{for} T_B \leq T \leq T_C$$

$$= a_g S \frac{2.5}{q} [\frac{T_C}{T}] \geq \beta a_g \text{ for} T_C \leq T \leq T_D$$
$$= a_g S \frac{2.5}{q} [\frac{T_C T_D}{T^2}] \geq \beta a_g \text{ for} T_D \leq T$$

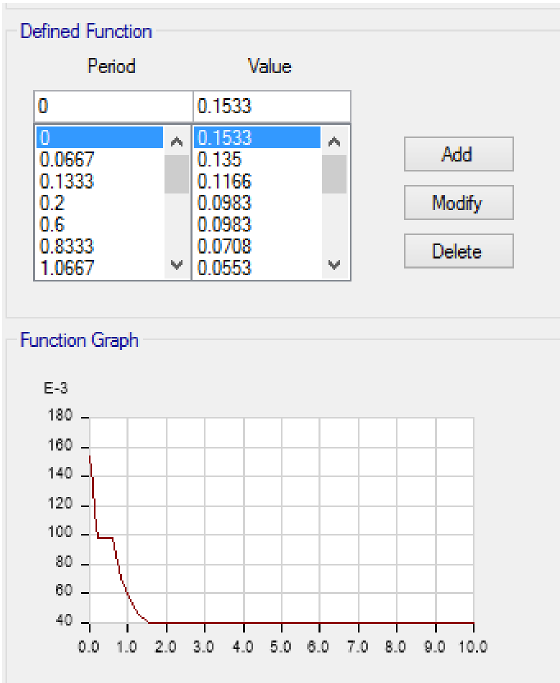


Figure 3.1: Design response spectrum curve according to EC 8

Equivalent Lateral Forces

Seismic Base Shear Coefficient $\quad V_{coeff} = S_d(T_1)\lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
X	0.589	14781.7806	1234.9735

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story5	15.6	383.5947	0
Story4	12.6	328.5838	0
Story3	9.6	250.3496	0
Story2	6.6	172.1153	0
Story1	3.6	100.3301	0
Base	0	0	0

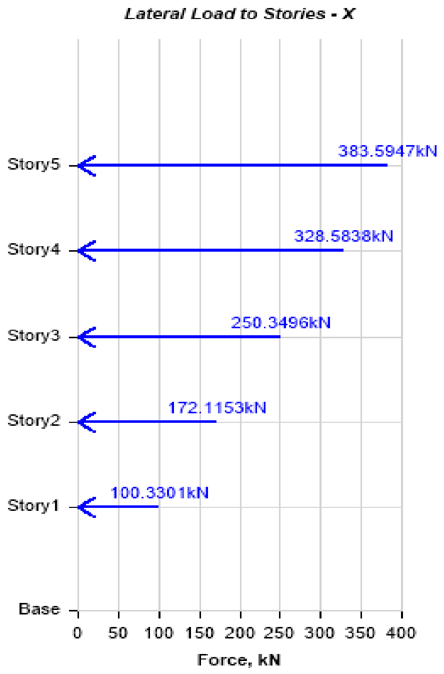


Figure 3.2: Lateral force acting in each stories in x-direction

This calculation presents the automatically generated lateral seismic loads for load pattern Eq y according to EUROCODE8, as calculated by ETABS.

Direction and Eccentricity

Direction = Y

Structural Period

Period Calculation Method = Approximate

Coefficient, C_t [EC 4.3.3.2.2]		$C_t = 0.075m$
Structure Height Above Base, H		$H = 15.6$ m
Approximate Fundamental Period, T_1 [EC 4.3.3.2.2(3) Eq. 4.6]	$T_1 = C_t H^{\frac{3}{4}}$	$T_1 = 0.589$ sec

Factors and Coefficients

Country = CEN Default

Design Ground Acceleration, a_g $a_g = 0.2g$

Ground Type [EC Table 3.1] = C

Soil Factor, S [EC Table 3.2] $S = 1.15$

Constant Acceleration Period Limit, T_B [EC Table 3.2] $T_B = 0.2$ sec

Constant Acceleration Period Limit, T_C [EC Table 3.2] $T_C = 0.6$ sec

Constant Displacement Period Limit, T_D [EC Table 3.2] $T_D = 2$ sec

Lower Bound Factor, β [EC 3.2.2.5(4)] $\beta_0 = 0.2$

Behavior Factor, q [EC 3.2.2.5(3)] $q = 5.85$

Seismic Response

Spectral Response Acceleration, Sd(T_1) [EC 3.2.2.5(4) Eq. 3.13]

$$S_d(T_1) = a_g S[\frac{2}{3} + \frac{T}{T_B}(\frac{2.5}{q} - \frac{2}{3})] \text{ for } T \leq T_B$$

$$= a_g S \frac{2.5}{q} \text{ for} T_B \leq T \leq T_C$$

$$= a_g S \frac{2.5}{q}[\frac{T_C}{T}] \geq \beta a_g \text{ for} T_C \leq T \leq T_D$$

$$= a_g S \frac{2.5}{q}[\frac{T_C T_D}{T^2}] \geq \beta a_g \text{ for} T_D \leq T$$

Equivalent Lateral Forces

Seismic Base Shear Coefficient $V_{coeff} = S_d(T_1)\lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
Y	0.589	14781.7806	1234.9735

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story5	15.6	0	383.5947
Story4	12.6	0	328.5838
Story3	9.6	0	250.3496
Story2	6.6	0	172.1153
Story1	3.6	0	100.3301
Base	0	0	0

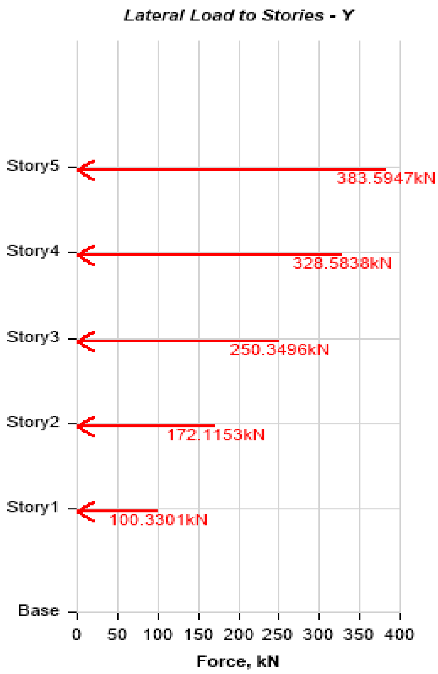


Figure 3.3: Lateral force acting in each stories in y-direction

2- ETABS results for moment-resisting frame with shear wall

This calculation presents the automatically generated lateral seismic loads for load pattern Eq x according to EUROCODE8, as calculated by ETABS.

Direction and Eccentricity

Direction = X

Structural Period

Period Calculation Method = Approximate

Coefficient, C_t [EC 4.3.3.2.2]		$C_t = 0.075m$
Structure Height Above Base, H		$H = 15.6$ m
Approximate Fundamental Period, T_1 [EC 4.3.3.2.2(3) Eq. 4.6]	$T_1 = C_t H^{\frac{3}{4}}$	$T_1 = 0.589$ sec

Factors and Coefficients

Country = CEN Default

Design Ground Acceleration, a_g $a_g = 0.2g$

Ground Type [EC Table 3.1] = C

Soil Factor, S [EC Table 3.2] $S = 1.15$

Constant Acceleration Period Limit, T_B [EC Table 3.2] $T_B = 0.2\ \text{sec}$

Constant Acceleration Period Limit, T_C [EC Table 3.2] $T_C = 0.6\ \text{sec}$

Constant Displacement Period Limit, T_D [EC Table 3.2] $T_D = 2\ \text{sec}$

Lower Bound Factor, β [EC 3.2.2.5(4)] $\beta_0 = 0.2$

Behavior Factor, q [EC 3.2.2.5(3)] $q = 5.4$

Seismic Response

Spectral Response Acceleration, $Sd(T_{1)}$ [EC 3.2.2.5(4) Eq. 3.13] $S_d(T_1) = a_g S[\frac{2}{3} + \frac{T}{T_B}(\frac{2.5}{q} - \frac{2}{3})]\ \text{for}\ T \leq T_B$

$$= a_g S \frac{2.5}{q}\ \text{for} T_B \leq T \leq T_C$$

$$= a_g S \frac{2.5}{q}[\frac{T_C}{T}] \geq \beta a_g\ \text{for} T_C \leq T \leq T_D$$

$$= a_g S \frac{2.5}{q}[\frac{T_C T_D}{T^2}] \geq \beta a_g\ \text{for} T_D \leq T$$

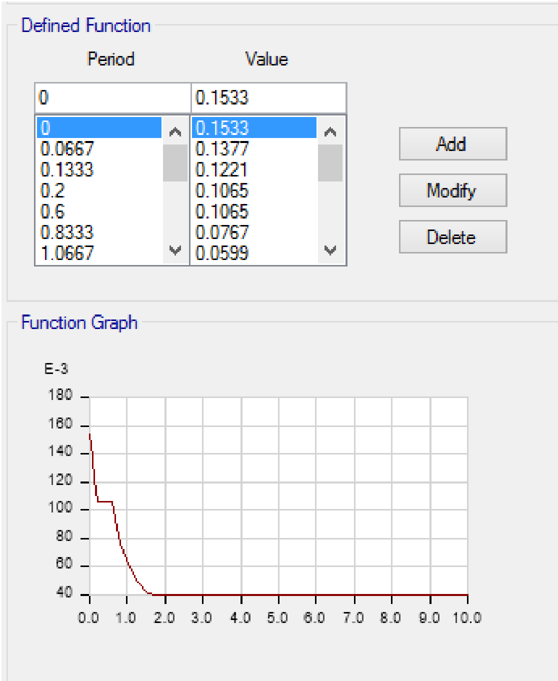


Figure 3.4: Design response spectrum curve according to EC 8

Equivalent Lateral Forces

Seismic Base Shear Coefficient $V_{coeff} = S_d(T_1)\lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
X	0.589	15355.3847	1389.8045

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story5	15.6	432.1903	0
Story4	12.6	369.5557	0
Story3	9.6	281.5662	0
Story2	6.6	193.5768	0
Story1	3.6	112.9156	0
Base	0	0	0

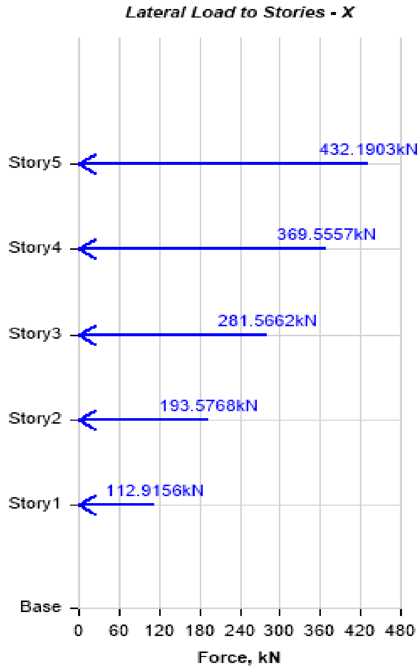


Figure 3.5: Lateral force acting in each stories in x-direction

This calculation presents the automatically generated lateral seismic loads for load pattern Eq y according to EUROCODE8, as calculated by ETABS.

Direction and Eccentricity

Direction = Y

Structural Period

Period Calculation Method = Approximate

Coefficient, C_t [EC 4.3.3.2.2]		$C_t = 0.075m$
Structure Height Above Base, H		$H = 15.6\ m$
Approximate Fundamental Period, T_1 [EC 4.3.3.2.2(3) Eq. 4.6]	$T_1 = C_t H^{\frac{3}{4}}$	$T_1 = 0.589\ sec$

Factors and Coefficients

Country = CEN Default

Design Ground Acceleration, a_g	$a_g = 0.2g$

Ground Type [EC Table 3.1] = C

Soil Factor, S [EC Table 3.2] $S = 1.15$

Constant Acceleration Period Limit, T_B [EC Table 3.2] $T_B = 0.2\ \text{sec}$

Constant Acceleration Period Limit, T_C [EC Table 3.2] $T_C = 0.6\ \text{sec}$

Constant Displacement Period Limit, T_D [EC Table 3.2] $T_D = 2\ \text{sec}$

Lower Bound Factor, β [EC 3.2.2.5(4)] $\beta_0 = 0.2$

Behavior Factor, q [EC 3.2.2.5(3)] $q = 5.4$

Seismic Response

Spectral Response Acceleration, Sd($T_{1)}$ [EC 3.2.2.5(4) Eq. 3.13] $S_d(T_1) = a_g S[\frac{2}{3} + \frac{T}{T_B}(\frac{2.5}{q} - \frac{2}{3})]\ \text{for}\ T \le T_B$

$$= a_g S \frac{2.5}{q}\ \text{for} T_B \le T \le T_C$$

$$= a_g S \frac{2.5}{q}[\frac{T_C}{T}] \ge \beta a_g\ \text{for} T_C \le T \le T_D$$

$$= a_g S \frac{2.5}{q}[\frac{T_C T_D}{T^2}] \ge \beta a_g\ \text{for} T_D \le T$$

Equivalent Lateral Forces

Seismic Base Shear Coefficient $V_{coeff} = S_d(T_1)\lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
Y	0.589	15355.3847	1389.8045

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story5	15.6	0	432.1903
Story4	12.6	0	369.5557
Story3	9.6	0	281.5662
Story2	6.6	0	193.5768
Story1	3.6	0	112.9156
Base	0	0	0

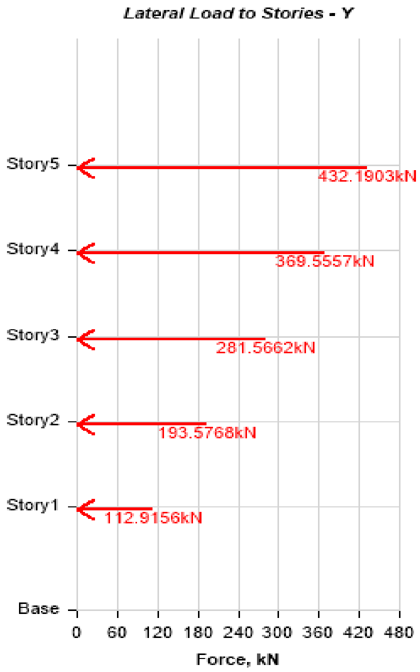


Figure 3.6: Lateral force acting in each stories in y-direction

APPENDIX 4

ETABS RESULTS ACCORDING TO NCSC 2015

1- ETABS results for moment-resisting frame

This calculation presents the automatically generated lateral seismic loads for load pattern Eq x according to NCSC 2015, as calculated by ETABS.

Direction and Eccentricity

Direction = X

Structural Period

Period Calculation Method = Approximate

Approximate Fundamental Period, T_a [TSC 2.7.4.2]	$T_a = 0.1N$	$T = 0.5\ \text{sec}$

Factors and Coefficients

Seismic Zone = Zone 2

Effective Ground Acceleration, A_0	$A_0 = 0.3$
Importance Factor, I [TSC Table 2.3]	$I = 1$
Characteristic Period, T_A [TSC Table 2.4]	$T_A = 0.15\ \text{sec}$
Characteristic Period, T_B [TSC Table 2.4]	$T_B = 0.6\ \text{sec}$
Factor, R [TSC Table 2.5]	$R = 8$

Seismic Response

Spectral Coefficient, $S(T_1)$ [TSC Eq. 2.2]

$$S(T_1) = 1 + 1.5\frac{T_1}{T_A}\ \text{for}\ 0 \le T_1 \le T_A$$

$$= 2.5\ \text{for}T_A \le T_1 \le T_B$$

$$= 2.5(^{\frac{T_B}{T_1}}\ \text{for}T_B T_1$$

Seismic Load Reduction Factor, $R_a(T_1)$ [TSC Eq. 2.3]

$$R_a(T_1) = 1.5 + (R - 1.5)\frac{T}{T_A}\ \text{for}\ 0 \le T_1 \le T_A$$

$$= R\ \text{for}T_A T_1$$

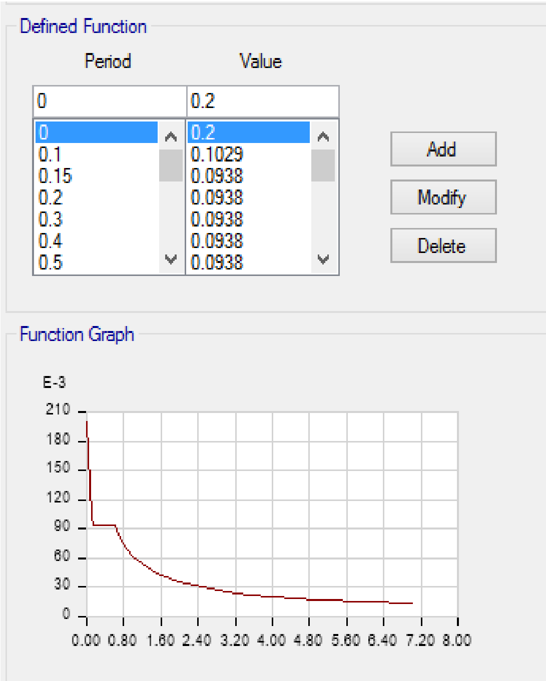


Figure 4.1: Design response spectrum curve according to NCSC 201

Equivalent Lateral Forces

Base Shear Coefficient [TSC Eq. 2.4] $= \frac{A(T_1)}{R_a(T_1)}$

minimum [TSC Eq. 2.4] $= 0.10 A_0 IW$

$\min \leq V_{coeff}$

Calculated Base Shear

Direction	Period Used (sec)	$S(T_1)$	W (kN)	V (kN)	F_N (kN)
X	0.5	2.5	14781.7806	1385.7919	51.9672

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story5	15.6	466.266	0
Story4	12.6	354.8847	0
Story3	9.6	270.3884	0
Story2	6.6	185.892	0
Story1	3.6	108.3609	0
Base	0	0	0

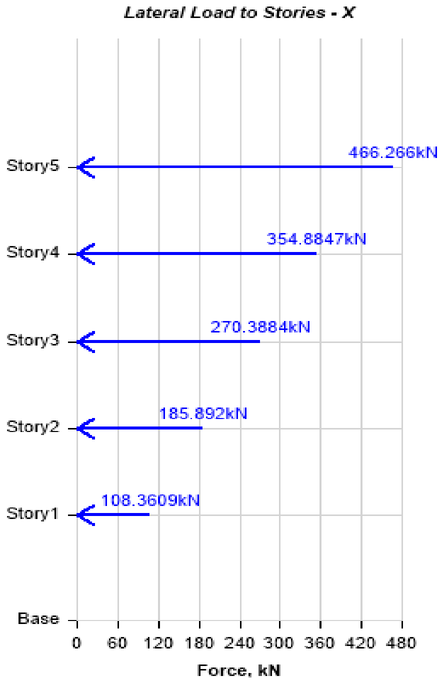


Figure 4.2: Lateral force acting in each stories in x-direction

This calculation presents the automatically generated lateral seismic loads for load pattern Eq y according to NCSC 2015, as calculated by ETABS.

Direction and Eccentricity

Direction = Y

Structural Period

Period Calculation Method = Approximate

Approximate Fundamental Period, T_a [TSC 2.7.4.2] $T_a = 0.1N$ $T = 0.5\ \text{sec}$

Factors and Coefficients

Seismic Zone = Zone 2

Effective Ground Acceleration, A_0 $A_0 = 0.3$

Importance Factor, I [TSC Table 2.3] $I = 1$

Characteristic Period, T_A [TSC Table 2.4] $T_A = 0.15\ \text{sec}$

Characteristic Period, T_B [TSC Table 2.4] $T_B = 0.6\ \text{sec}$

Factor, R [TSC Table 2.5] $R = 8$

Seismic Response

Spectral Coefficient, $S(T_1)$ [TSC Eq. 2.2]

$S(T_1) = 1 + 1.5\frac{T_1}{T_A}$ for $0 \leq T_1 \leq T_A$

$= 2.5$ for $T_A \leq T_1 \leq T_B$

$= 2.5(\frac{T_B}{T_1}$ for $T_B T_1$

Seismic Load Reduction Factor, $R_a(T_1)$ [TSC Eq. 2.3]

$R_a(T_1) = 1.5 + (R - 1.5)\frac{T}{T_A}$ for $0 \leq T_1 \leq T_A$

$= R$ for $T_A T_1$

Equivalent Lateral Forces

Base Shear Coefficient [TSC Eq. 2.4]

$= \frac{A(T_1)}{R_a(T_1)}$

minimum [TSC Eq. 2.4]

$= 0.10 A_0 I W$

$\text{min} \leq V_{coeff}$

Calculated Base Shear

Direction	Period Used (sec)	$S(T_1)$	W (kN)	V (kN)	F_N (kN)
Y	0.5	2.5	14781.7806	1385.7919	51.9672

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story5	15.6	0	466.266
Story4	12.6	0	354.8847
Story3	9.6	0	270.3884
Story2	6.6	0	185.892
Story1	3.6	0	108.3609
Base	0	0	0

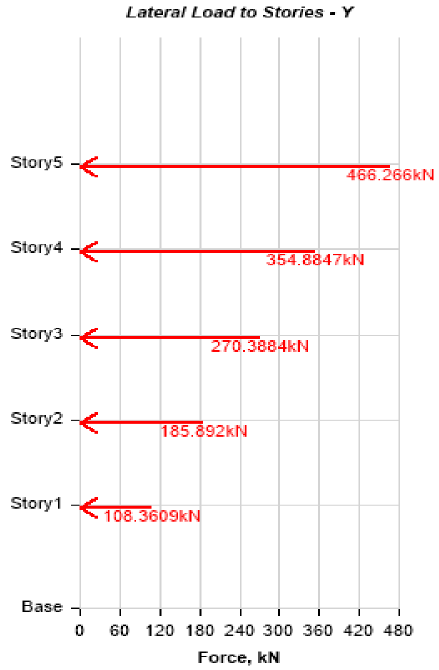


Figure 4.3: Lateral force acting in each stories in y-direction

2- ETABS results for moment-resisting frame with shear wall

This calculation presents the automatically generated lateral seismic loads for load pattern Eq x according to NCSC 2015, as calculated by ETABS.

Direction and Eccentricity

Direction = X

Structural Period

Period Calculation Method = Approximate

Approximate Fundamental Period, T_a [TSC 2.7.4.2]	$T_a = 0.1N$	$T = 0.5$ sec

Factors and Coefficients

Seismic Zone = Zone 2

Effective Ground Acceleration, A_0	$A_0 = 0.3$
Importance Factor, I [TSC Table 2.3]	$I = 1$
Characteristic Period, T_A [TSC Table 2.4]	$T_A = 0.15$ sec
Characteristic Period, T_B [TSC Table 2.4]	$T_B = 0.6$ sec

Factor, R [TSC Table 2.5] $R = 7$

Seismic Response

Spectral Coefficient, $S(T_1)$ [TSC Eq. 2.2]

$$S(T_1) = 1 + 1.5\frac{T_1}{T_A} \text{ for } 0 \leq T_1 \leq T_A$$

$$= 2.5 \text{ for} T_A \leq T_1 \leq T_B$$

$$= 2.5(\frac{T_B}{T_1} \text{ for} T_B T_1$$

Seismic Load Reduction Factor, $R_a(T_1)$ [TSC Eq. 2.3]

$$R_a(T_1) = 1.5 + (R - 1.5)\frac{T}{T_A} \text{ for } 0 \leq T_1 \leq T_A$$

$$= R \text{ for} T_A T_1$$

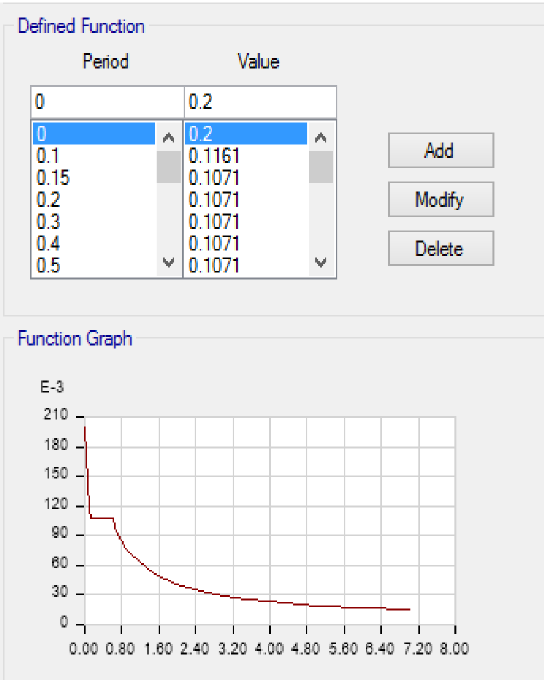


Figure 4.4: Design response spectrum curve according to NCSC 2015

Equivalent Lateral Forces

Base Shear Coefficient [TSC Eq. 2.4]

$$= \frac{A(T_1)}{R_a(T_1)}$$

minimum [TSC Eq. 2.4]

$$= 0.10A_0IW$$

$$\min \leq V_{coeff}$$

Calculated Base Shear

Direction	Period Used (sec)	S(T_1)	W (kN)	V (kN)	F_N (kN)
X	0.5	2.5	15355.3847	1645.2198	61.6957

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story5	15.6	554.1274	0
Story4	12.6	421.0666	0
Story3	9.6	320.8127	0
Story2	6.6	220.5587	0
Story1	3.6	128.6544	0
Base	0	0	0

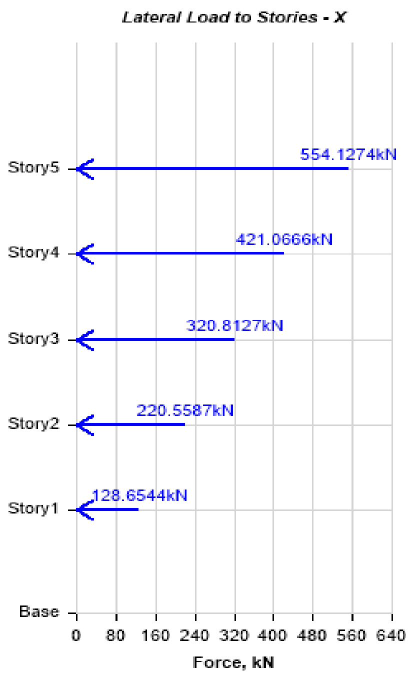


Figure 4.5: Lateral force acting in each stories in x-direction

This calculation presents the automatically generated lateral seismic loads for load pattern Eq y according to NCSC 2015, as calculated by ETABS.

Direction and Eccentricity

Direction = Y

Structural Period

Period Calculation Method = Approximate

Approximate Fundamental Period, T_a [TSC 2.7.4.2]	$T_a = 0.1N$	$T = 0.5$ sec

Factors and Coefficients

Seismic Zone = Zone 2

Effective Ground Acceleration, A_0	$A_0 = 0.3$
Importance Factor, I [TSC Table 2.3]	$I = 1$
Characteristic Period, T_A [TSC Table 2.4]	$T_A = 0.15$ sec
Characteristic Period, T_B [TSC Table 2.4]	$T_B = 0.6$ sec
Factor, R [TSC Table 2.5]	$R = 7$

Seismic Response

Spectral Coefficient, $S(T_1)$ [TSC Eq. 2.2] $\quad S(T_1) = 1 + 1.5\frac{T_1}{T_A}$ for $0 \le T_1 \le T_A$

$= 2.5$ for$T_A \le T_1 \le T_B$

$= 2.5(^{\frac{T_B}{T_1}}$ for$T_B T_1$

Seismic Load Reduction Factor, $R_a(T_1)$ [TSC Eq. 2.3] $\quad R_a(T_1) = 1.5 + (R - 1.5)\frac{T}{T_A}$ for $0 \le T_1 \le T_A$

$= R$ for$T_A T_1$

Equivalent Lateral Forces

Base Shear Coefficient [TSC Eq. 2.4] $\quad = \frac{A(T_1)}{R_a(T_1)}$

minimum [TSC Eq. 2.4] $\quad = 0.10A_0IW$

$\min \le V_{coeff}$

Calculated Base Shear

Direction	Period Used (sec)	$S(T_1)$	W (kN)	V (kN)	F_N (kN)
Y	0.5	2.5	15355.3847	1645.2198	61.6957

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story5	15.6	0	554.1274
Story4	12.6	0	421.0666
Story3	9.6	0	320.8127
Story2	6.6	0	220.5587
Story1	3.6	0	128.6544
Base	0	0	0

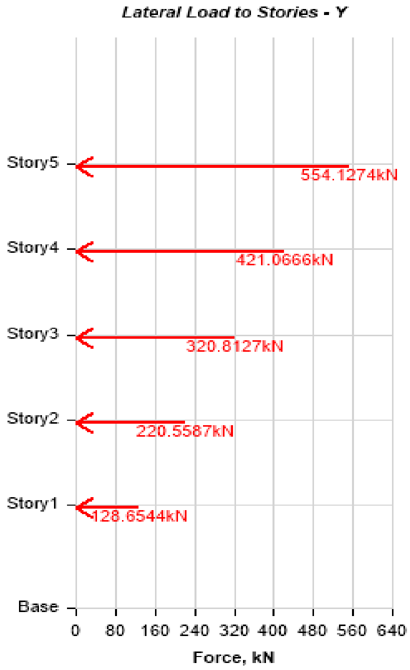


Figure 4.6: Lateral force acting in each stories in y-direction

APPENDIX 5

MODE SHAPES

The mode shapes for RC building systems, MRF and MRF+SW are given from Figure 5.1 to 5.6.

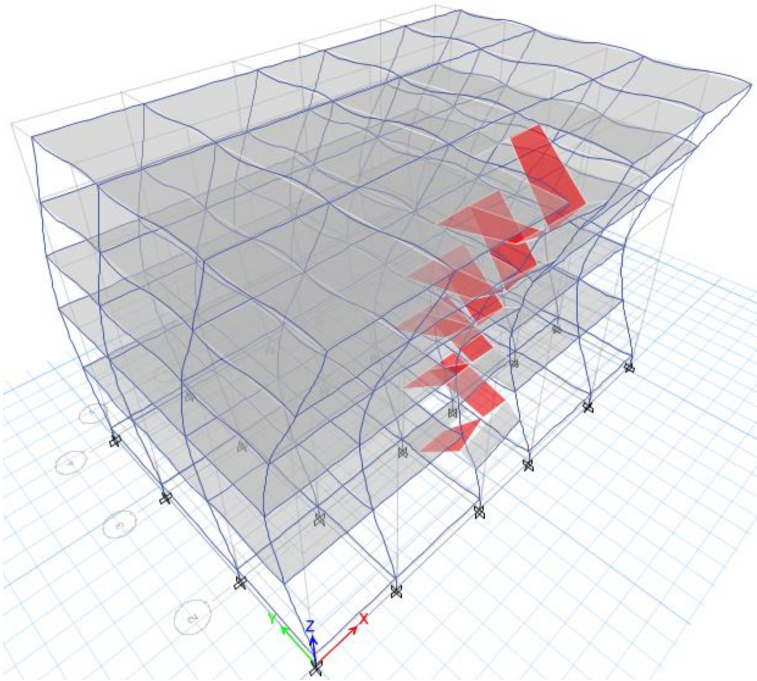


Figure 5.1: Mode shape 1 MRF

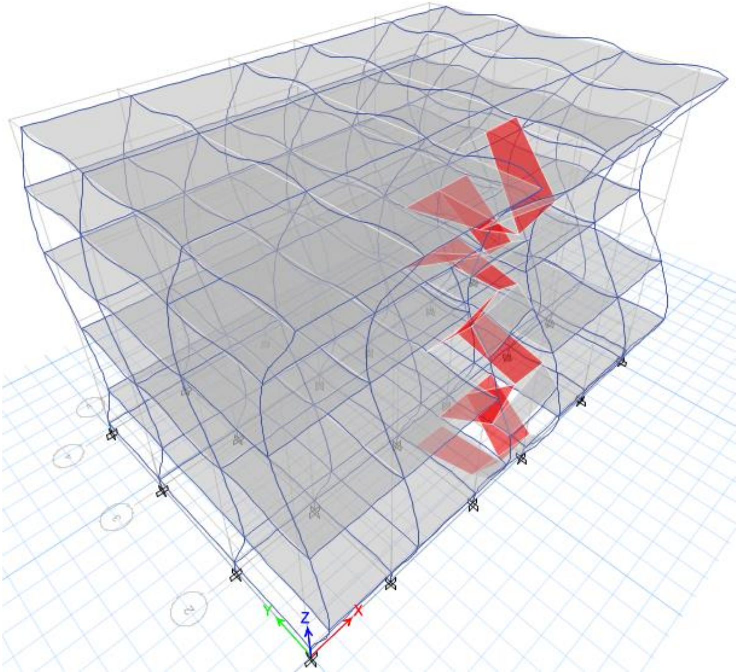


Figure 5.2: Mode shape 2 MRF

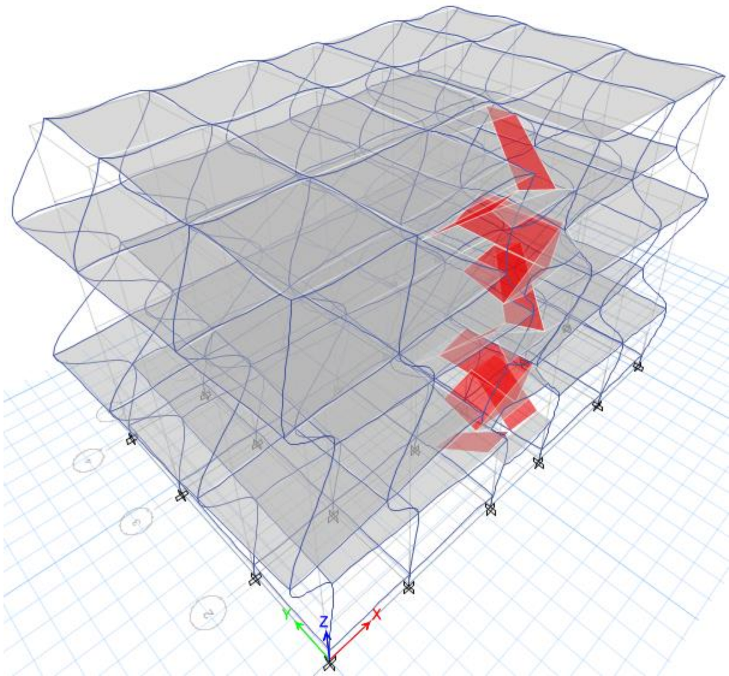


Figure 5.3: Mode shape 3 MRF

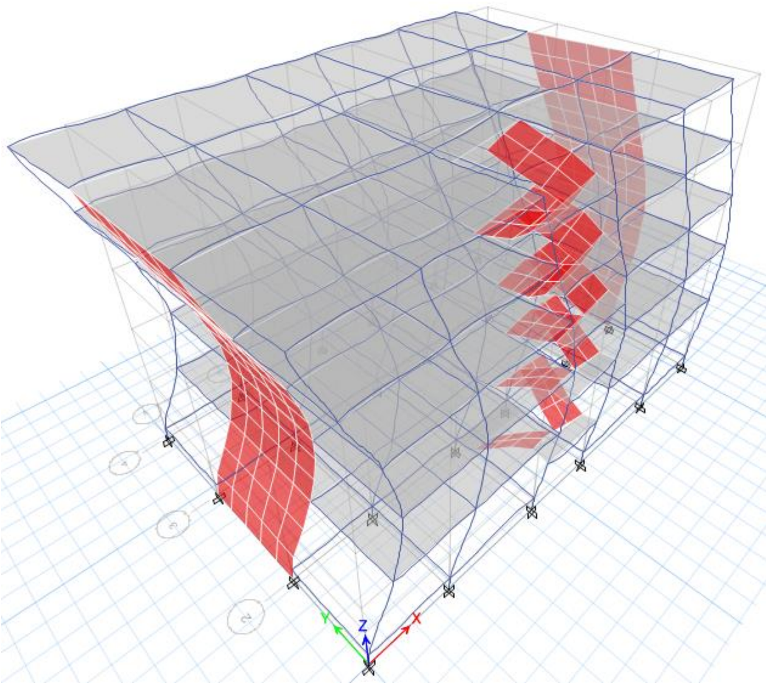


Figure 5.4: Mode shape 1 MRF+SW

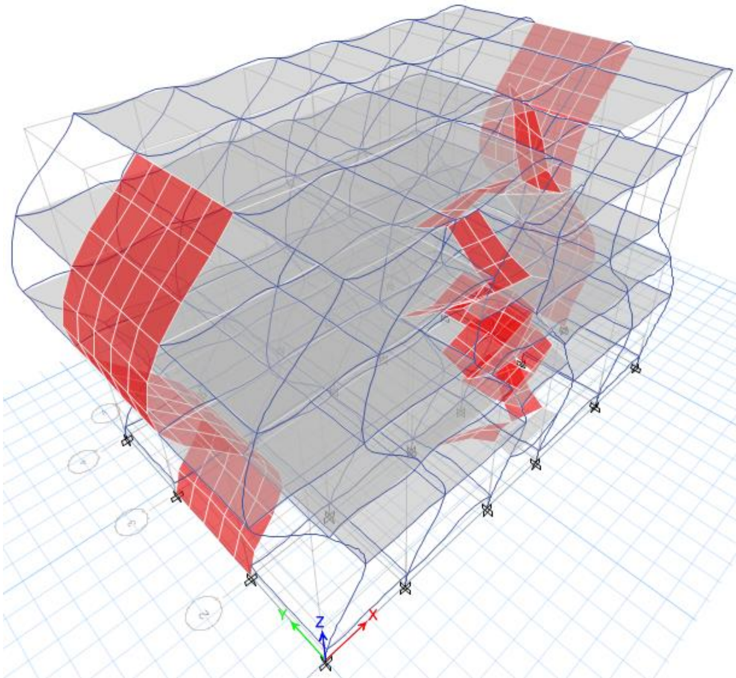


Figure 5.5: Mode shape 2 MRF+SW

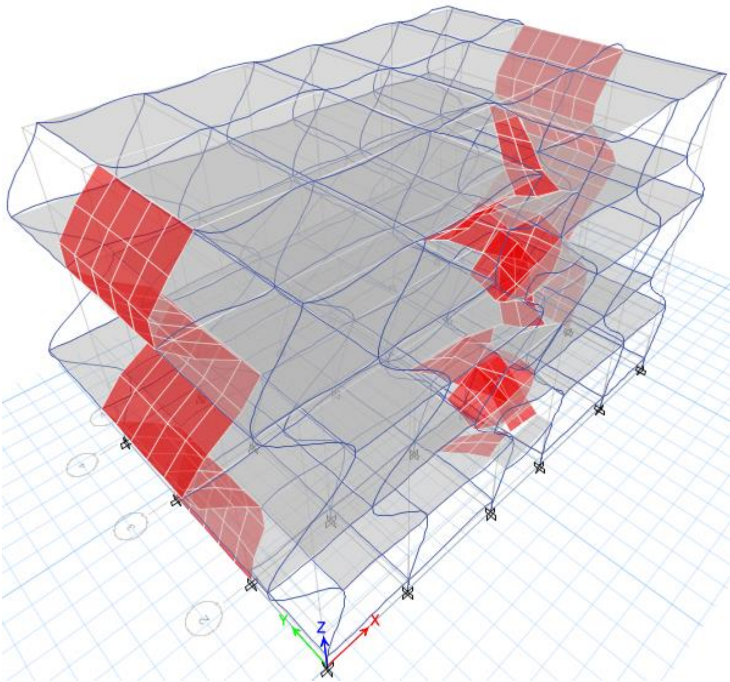


Figure 5.6: Mode shape 3 MRF+SW

ABSTRACT

This study presents a comparative evaluation between three seismic design codes, the International Building code (IBC 2009) and Eurocode 8 (EC 8) which are well known and the seismic design code for northern Cyprus which was established in 2015. In order to make possible comparison among the codes, a particular location and the most common residential frame model has been chosen. In this research, a building of moment-resisting frame and moment-resisting frame with shear wall plan of reinforced concrete (RC) frames were analysed for low-rise to mid-rise structures. Response spectrum method (RSM) and equivalent lateral force method (ELFM) were performed using extended three dimensional analysis of building system (ETABS) software package. The main objective of this study is to examine the seismic provisions of the first edition of the northern Cyprus seismic code to determine whether it provides a generic level of safety that incorporate in well established code. The results obtained from both static and dynamic analysis are presented in the form of base shear, story shear, displacement, axial forces and bending moments for selected columns for three different codes.

Keywords: Seismic design code; equivalent lateral force method; response spectrum method; moment-resisting frame; moment-resisting frame with shear wall; north Cyprus

ÖZET

Bu çalışmada, üç farklı deprem yönetmeliği için karşılaştırmalı değerlendirmeler yapılmıştır. Kuzey Kıbrıs'ta 2015 yılında hazırlanmış deprem bölgelerinde yapılacak binalar hakkındaki yönetmelik, iyi bilinen ve yaygın olarak kullanılan IBC2009 ve EC 8 yönetmelikleri ile karşılaştırılmış ve değerlendirmeler yapılmıştır. Yönetmelikler arasında olası karşılaştırmaların yapılabilmesi için, belirli bir yer ve en yaygın konut çerçeve modeli seçilmiştir. Bu araştırmada, az ve orta yükseklikteki yapılar için, moment dayanımlı çerçeveve perde duvarlı moment dayanımlı betonarme çerçevelerin yapısal analizleri yapılmıştır.Bunun için ETABS yazılım paketi yardımı ile , tepki spektrumu yöntemi ve eşdeğer yanal kuvvet yöntemi kullanılarak üç boyutlu analiz gerçekleştirilmiştir.Bu çalışmanın temel amacı kuzey Kıbrıs'ta kullanılmaya başlanan sismik tasarım yönetmeliğinin ilk baskısının sismik hükümlerini inceleyip, iyi hazırlanmış yönetmeliklerin dahil edildiği kapsamlı bir güvenlik seviyesi sağlayıp sağlamadığını tespit etmektir. Üç farklı yönetmelik için statik ve dinamik analizden elde edilen sonuçlar, taban kesme kuvveti, kat kesme kuvveti, yerdeğiştirme, ve bazı seçilmiş kolonlarda,eksenel kuvvetler ve eğilme momentleri şeklinde sunulmuştur.

Anahtar Kelimeler: Sismik tasarım yönetmeliği; eşdeğer yanal kuvvet yöntemi; tepki spektrum yöntemi; momente dayanımlı çerçeve; perde duvarlı moment dayanımlı çerçeve; Kuzey Kıbrıs