

**ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
INSTITUTE OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING**

**A COMPARATIVE STUDY OF THE SEISMIC PROVISIONS OF
EBCS-8 AND CURRENT MAJOR BUILDING CODES ON THE
EQUIVALENT LATERAL FORCE ANALYSIS AND DYNAMIC
RESPONSE SPECTRUM ANALYSIS**

**A Thesis Submitted to the Graduate School of the Addis Ababa University in
Partial Fulfillment of the Requirements for the Degree of
Master of Science in Civil Engineering**

By

Yiha Wassie

Advisor: Dr.-Ing. Girma Zerayohannes

September 2011

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Declaration

I declare that this thesis is my original work. This thesis has not been presented for any other university and is not concurrently submitted in candidature of any other degree, and that all sources of material used for the thesis have been duly acknowledged.

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Abstract

This thesis work provides a comparison of seismic provisions having relevance to equivalent lateral force (ELF) and response spectrum analysis (RSA) according to some current seismic codes. The codes reviewed and compared are the uniform building code (UBC, 1997), the International building code (IBC, 2009), the Eurocode 8 (EC8, 2004) and the Ethiopia building code of standard (EBCS-8, 1995). Selection of analysis procedure, regularity criteria, base shear coefficient, distribution of base shear, consideration of torsion, story drift determination and limitation and P-delta effect consideration have been compared. In addition, the design spectra for all soil types and the combination rules of horizontal seismic action are considered for comparison. Furthermore, the ELF and RSA base shear of the four codes using different sample buildings with different heights are compared. Each building is located at all different soil types considered in the codes. Also, the analysis results of the two methods of analysis using different sample buildings compared with each other. Finally, combination rules for horizontal seismic forces are compared and evaluated. In this work, the differences caused by the use of different codes in the ELF and RSA analysis are investigated. The results prove the need to review the EBCS-8 and develop more appropriate relations towards achieving economy and safety.

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LIST OF SYMBOLS AND NOTATIONS

Symbols and Notations of UBC 97

A_c	combined effective area, in m^2 , of the shear walls in the first story of the structure
A_x	amplification of accidental torsion
A_e	the minimum cross sectional area in any horizontal plan
C_a	seismic coefficient
C_t	numerical coefficient
C_v	seismic coefficient
D_e	first story, in m^2 of the shear wall
f_i	lateral force at level i
F_t	the portion of the base shear concentrated at the top of the structure in addition to f_n
g	acceleration due to gravity
h_i, h_n	height in meter above the base to level i and n
I	important factor
N_v	near source factor
P_x	total vertical design load at and above level X in KN
R	numerical coefficient representative of the inherent over strength and global ductility capacity of lateral force resisting system
T	elastic fundamental period of vibration, in second , of the structure in the direction under consideration
V	total design latera force or shear at the base
V_x	design story shear in story x
W	total seismic dead load
W_p	weight of an element or component
Z	seismic zone factor
δ_{avg}	the average of the displacements of the extreme points of the structure at level x
δ_i	horizontal displacement at level i related to the base due to the applied lateral forces, f
Δ_m	maximum inelastic response displacement, which is the total drift or lateral story drift.
δ_{max}	maximum displacement at level x
Δ_s	Design level response displacement, which is the total drift or total story drift

Symbol and Notation of IBC 2009

C_s	seismic response coefficient (dimensionless)
A_x	amplification of accidental torsion
C_d	deflection amplification factor
C_t	building period coefficient
C_{vx}	vertical distribution factor
D_i	effect of dead load
F_a	short-period site coefficient (at 0.2 s-period)
F_v	long-period site coefficient (at 1.0 s-period)
F_x	portion of the seismic base shear, V , induced at Level i , n , or x , respectively
h	average roof height of structure with respect to the base
h_n	height above the base to Level n
h_{sx}	story height below Level $x = (h_x - h_{x-1})$ in mm.
I	importance factor
L_i	floor-dimension perpendicular to the direction of the seismic action..
N	number of stories
N_{SPT}	average field standard penetration resistance for the top 100 ft (30 m)
P_x	total vertical design load at and above level x in kN
R	response modification coefficient
S_1	mapped MCE, 5 percent damped, spectral response acceleration parameter at a period of 1 s
S_{D1}	design, 5 percent damped, spectral response acceleration parameter at a period of 1 s
S_{DS}	design, 5 percent damped, spectral response acceleration parameter at short periods
S_{M1}	the MCE, 5 percent damped, spectral response acceleration at a period of 1 s adjusted for site class effects
S_{MS}	the MCE, 5 percent damped, spectral response acceleration at short periods adjusted for site class effects
S_S	mapped MCE, 5 percent damped, spectral response acceleration parameter at short periods
S_U	average undrained shear strength in top 30 m
T	the fundamental period of the building
T_a	approximate fundamental period of the building

T_L	long-period transition period
T_S	S_{SD1}/S_{DS}
V	total design lateral force or shear at the base
V_t	design value of the seismic base shear as determined
V_x	seismic design shear force acting between level x and $x-1$ in kN.
v_s	average shear wave velocity at small shear strains in top 30 m
W	effective seismic weight of the building
W_x	portion of W that is located at or assigned to Level x
x	level under consideration, 1 designates the first level above the base
β	ratio of shear demand to shear capacity for the story between Level x and $x - 1$. This ratio is permitted to be conservatively taken as 1.0.
δ_x	deflection of Level x at the center of the mass at and above Level x
δ_{xe}	deflection of Level x at the center of the mass at and above Level x
θ	stability coefficient for P-delta effects

Symbol and Notation of EUROCODE 8

A_c	total effective area of the shear walls in the first storey of the building, in m ²
A_x	amplification of accidental torsion
C_u	average undrained shear strength in top 30 m
d	lateral elastic displacement of the top of the building, in m, due to the gravity loads applied in the horizontal direction
d_e	displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum
d_r	design inter-storey drift
d_r	design inter-storey drift, evaluated as the difference of the average lateral displacements, d_s at the top and bottom of the storey
d_s	displacement of a point of the structural system induced by the design seismic action
d_{SB}	design inter-storey drift at the bottom of the story
d_{st}	design inter-storey drift at the top of the story
e_{ai}	accidental eccentricity of storey mass i from its nominal location, applied in the same direction at all floors

E_E	seismic action effect under consideration (force, displacement, etc.)
E_{Ei}	value of this seismic action effect due to the vibration mode i .
e_o	distance between the centre of stiffness and the centre of mass which is normal to the direction of analysis considered
F_b	base shear force
H	building height from the foundation or from the top of a rigid basement
h	inter-storey height.
k	number of modes taken into account
L_e	distance between the two outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action considered
L_i	floor-dimension perpendicular to the direction of the seismic action.
L_S	radius of gyration of the floor mass in plan
L_{wi}	length of the shear wall i in the first storey in the direction parallel to the applied forces, in m, with the restriction that l_{wi}/H should not exceed 0,9
m_i, m_j	mass of storey i and j
n	number of storey above the foundation or the top of a rigid basement
P_{tot}	total gravity load at and above the storey considered in the seismic design situation
q	behaviour factor
q_d	displacement behaviour factor, assumed equal to q unless otherwise specified;
r	torsional radius
r_x	square root of the ratio of the torsional stiffness to the lateral stiffness in the y direction (“torsional radius”)
S	soil factor
S_d	design spectrum (for elastic analysis). At $T=0$, the spectral acceleration given by this spectrum equals the design ground acceleration on type A ground multiplied by the soil factor S
S_i, S_j	displacements of masses m_i, m_j in the fundamental mode shape
T	vibration period of a linear single-degree-of-freedom system
T_1	fundamental period of vibration of a building
T_1	fundamental vibration period of the building in the relevant direction
T_a	fundamental vibration period of the non-structural element
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	beginning of the constant displacement response range of the spectrum
T_k	period of vibration of mode k
v	reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement.
V_{tot}	total seismic storey shear
$V_{S,30}$	average shear wave velocity at small shear strains in top 30 m
x	distance of the element under consideration from the centre of mass of the building in plan, measured perpendicularly to the direction of the seismic action considered
Z_i, Z_j	height of mass m_i and m_j above the level of application of the seismic action
α	ratio of the design ground acceleration to the acceleration of gravity
α_g	design ground acceleration on type A ground ($ag = \gamma I. agR$)
α_{gR}	reference peak ground acceleration on type A ground, agR ,
β	lower bound factor for the horizontal design spectrum
θ	interstorey drift sensitivity coefficient
λ	correction factor

Symbols and Notations of EBCS-8

A_c	total effective area of the shear walls in the first storey of the building, in m^2
A_i	cross sectional area of shear wall in the first story of the building, in m^2
A_x	amplification factor
d	lateral displacement of the top of the building, in m, due to the gravity loads applied horizontally
d_e	displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum.
d_r	design inter story drift, evaluated as the difference of the average lateral displacement at the top and bottom of story under consideration
d_s	displacement of a point of the structural system induced by the design seismic action
F_b	base shear force
F_i	horizontal forces at the level i
F_t	concentrated force at the top, in addition f_n

h	height of the story
H	height of the building above the base in meter
L_e	distance between the two outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action considered
L_{wi}	length of the shear wall i in the first storey in the direction parallel to the applied forces, in m, with the restriction that l_{wi}/h should not exceed 0,9
P_{tot}	total gravity load at and above the story considered
S	site coefficient for the site
$S_d(T_1)$	for linear analysis, the design spectra normalized by the acceleration of gravity
T_1	fundamental period of vibration of the structure for translation motion in the direction considered
V_x	story shear
V_{tot}	total seismic story shear
W	seismic dead load computed
x	distance of the element under consideration from the centre of mass of the building in plan, measured perpendicularly to the direction of the seismic action considered
	seismic story drift
α	the ratio of design bedrock acceleration to the acceleration of gravity g .
α_0	bed rock acceleration ratio for the site and depend on the seismic zone
β	design response factor for the site
γ	behavioural factor
δ_{avg}	average of the displacements of the extreme points of the structure at level x
δ_{max}	maximum displacement at level x
θ	inter story drift sensitivity coefficient

CHAPTER ONE

INTRODUCTION

1.1 Background

Seismic building codes are guidelines to design and construct the buildings and civil engineering works in seismic regions. Their purposes are to protect human lives, to limit damage, and to sustain operations of important structures for civil protection. Seismic design has progressed significantly over the year due to the contribution of practicing engineers, as well as academic and governmental researchers. The progress depends on the improvement of the representation of ground motion, soil type and structure [1, 2].

Approval and requirement to apply the first edition of the Ethiopian standard code of practice for loading (ESCP1) was practically enacted in 1983. The current second edition (EBCS-8) approved, officially announced and impacted for design, control and inspection for the building in 1995.

Before 2000, three regional model Codes prevailed in the United States; the uniform building code (UBC) in west, the BOCA Code in north and the Standard Building Code (SBC) was prevalent in the south of that country. The International Council of Codes was established in 1994 to develop the unique comprehensive code not bound by regional limitations; and it ultimately formulated the International Building Code (IBC 2000) as the first publication. IBC 2003 was the next version, which was developed based on the Federal Emergency Management Agency (FEMA) instructions in the framework of National Earthquake Hazards Reduction Program (NEHRP) recommending certain precautions to improve seismic regulations for new buildings. IBC code supplement has been continuing every 18 month, with new full edition every three years

Seismic code provisions stipulate the standard procedure for the analysis of forces and displacements in structures subjected to ground motion. The two most analysis procedures are Equivalent lateral force (ELF) analysis and Dynamic response spectrum analysis (RSA). The ELF analysis is a simplified technique derived from structural dynamics. It is based on the dynamics of a single degree of freedom oscillator or multi degree of freedom system vibrating in accordance with the specific shape. In contrast the response spectrum analysis is

a more rigorous method directly based on the structural dynamics theory. It presupposes a structure modeled by a multi degree of freedom (MDOF) oscillator subjected to an earthquake ground motion represented by a design response spectrum [3]. The general guidance seen in the codes is that structural engineers conduct the seismic analysis according to the simplified procedure required or a more rigorous analysis may be used to more accurately determine the seismic force acting on the structure.

Since the EBCS-8 is driven from UBC and EC8, which have undergone major change over the year, this study aims to compare specifically the ELF analysis procedure and dynamic response spectrum analysis procedure covered in UBC 97, IBC 2009, EC8 and EBCS-8. This comparison is used to evaluate EBCS-8 with respect to major current codes, earthquake Engineering and structural dynamics.

1.2 Statement of the Problem

Earthquake codes have been revised and updated depending on the improvements in the representation of ground motions, soils and structures. EBCS-8 is derived from U.S. codes and European standards, which have undergone major changes over the year. Due to this there may be some drawbacks with relative to other major current codes. Some provisions may be prohibitive and the other may be unsafe. This comparison is used to evaluate EBCS-8 with respect to major current codes. It believed that this study will play a great role to initiate updating and it can be used as the base for updating the current code.

1.3 Objectives

1.3.1 General Objective

The general objective of this research work is to compare and evaluate the Equivalent lateral force analysis and dynamic response analysis procedures of EBCS 8 with respect to UBC 97, IBC 2009 and EUROCODE 8.

1.3.2 Specific Objectives

The specific objectives of this research are:

- To evaluate and compare analysis techniques.
- To evaluate and compare combination rules of the horizontal component of seismic action with respect to the critical response.

1.4 Methodology

The tasks used to accomplish the objectives and come up with relevant conclusions and recommendations are outlined as follows.

1.4.1 Literature Review and Assessment of Seismic Specifications

Detail literature review and assessment of selected seismic code provisions from the perspective of the theory of structural dynamics and principle of earthquake engineering have been done.

1.4.2 Compare Some Section of Seismic Provision of EBCS-8 with Major Current Codes

Selection of analysis procedure, regularity criteria, base shear coefficient, distribution of base shear, consideration of torsional, story drift determination and limitation and P-delta effect consideration have been compared. In addition the design spectra for all soil type, the combination rules of horizontal seismic action are considered for comparison.

1.4.3 Identification of Case Study Buildings

To accomplish these study twenty two case study buildings were chosen. All buildings are intermediate reinforced concrete moment frame and standard occupancies. Five buildings are use for the comparison of the base shear of selected codes. Additional fifteen buildings are chosen for the comparison of the two analysis techniques (ELF analysis and RSA). The other two simple framed buildings are used for evaluation of combination rules of horizontal seismic forces.

1.4.4 Comparison of Base Shears of Selected Seismic Codes

A Sample of five buildings with the same layouts and different heights was used for this comparison. Each of these buildings assumed to be located on different soil types. For these buildings the base shear calculated using the codes specified response spectra and static base shear are compared.

1.4.5 Comparison of ELF Analysis and Response Spectral Analysis

For this comparison a sample of twenty buildings with different layout and different height were selected. All buildings are assumed located on soil type B (EBCS-8). For these buildings response spectrum analysis and static analysis was performed. In addition the member forces due to the two methods are compared and the accuracy and relative merits of the analysis approach for these structures are discussed.

1.4.6 Evaluation and Comparison of Combination Rules of Horizontal Seismic Action for Peak Response Calculation.

For this task two simple framed building which are located on soil type B was chosen. Using the response spectra defined by EBCS-8 the result of the combination rules are compared with the critical response. The commercial computer software package ETABS non-linear version 9.7.2 was used for all analysis done in this thesis.

1.5 Scope of the Study

Comparing and assessing all seismic provisions are a very wide and time taking task and also may be difficult to cover in one thesis. In this thesis, the most common types of seismic code analysis procedures which are ELF analysis and RSA procedures are covered. Under the two procedures: Selection of analysis procedure, regularity criteria, base shear coefficient, distribution of base shear, consideration of torsional, story drift determination and limitation, P-delta effect consideration, soil classification, effect of local soil condition, design spectra and the combination rules of horizontal seismic action are covered. Most building codes permit dynamic response history analysis and non-linear static analysis. The codes versions of these procedures are not included in this thesis. Furthermore the design provisions and detailing rules are not part of this thesis.

1.6 Application of the Result

Since the last version of EBCS-8 is revised before fifteen years, it is the time to amend. This study can be used as an impute for updating our current code (EBCS-8).

CHAPTER TWO

COMPARISON OF EQUIVALENT LATERAL FORCE ANALYSIS PROCEDURE

2.1 Introduction

The ELF analysis is a simplified technique derived from structural dynamics. It is based on the dynamics of a single degree of freedom oscillator or multi degree of freedom system vibrating in accordance with the specific shape.

The preliminary design based on an ELF analysis is a natural place to start. The ELF analysis is useful, even if the final design is based on a more sophisticated analysis. For instance, forces from an ELF analysis are used to apply accidental torsion, to scale the results from the more advanced analysis and is useful as a check on a modal response spectrum or time-history analysis. Equilibrium can be checked and various modeling approximations can be verified with simple static load patterns [4].

The simplifications inherent in the ELF procedure result in approximations that are likely to be inadequate if the lateral motions in two orthogonal directions and the torsional motion are strongly coupled. Moreover, ELF analysis is inadequate if story strengths are distributed irregularly over height. In such cases, a more rigorous procedure that considers the dynamic behavior of the structure should be employed. It would be adequate, in general, to use the ELF procedure for structures whose floor masses, cross-sectional areas and moments of inertia of structural members do not differ by more than 30 percent in adjacent floors [5]. In addition, this type of analysis may be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction [6].

2.2 Permitted ELF Analysis Procedure

Table 2-1 Permitted ELF Analysis Procedure in the UBC, IBC, EC8 and EBCS-8

Code	Applicability of ELF
UBC [7]	<ul style="list-style-type: none"> All structures regular or irregular, in seismic zone 1 and in occupancy categories 4 and 5 in seismic zone 2. Regular structures under 73.152m in height. Irregular structures not more than five story or 19.812m.
IBC [8]	<ul style="list-style-type: none"> For all structures located in seismic category B and C. For seismic category D, E and F. <ul style="list-style-type: none"> Risk Category I or II buildings not exceeding 2 stories above the base Structures of light frame construction. Structures with no structural irregularities and not exceeding 48.7m in structural height. Structures exceeding 48.7m in structural height with no structural irregularities and with $T < 3.5T_S$. Structures not exceeding 48.7m in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 2-3 or vertical irregularities of Type 4, 5a, or 5b in Table 2-2.
EC8 [2]	<ul style="list-style-type: none"> structures having fundamental periods of vibration, T_I, in the two main directions which are smaller than $4T_C$ and 2s and meet the criteria for regularity in elevation Regular structures under 40m in height.
EBCS-8 [9]	<ul style="list-style-type: none"> structures having fundamental periods of vibration, T_I, in the two main directions less than 2 sec. and meet the criteria for regularity in plan and/or in elevation Regular structures under 80m in height.

2.2.1 Criteria of Regularity

2.2.1.1 Regular Structures

Those have no significant physical discontinuity in plan or vertical configuration or in their lateral force system such as irregular features. more rational definition is that a “regular structure” is one in which there is a minimum coupling between the lateral displacements and the torsional rotations for the mode shapes associated with the lower frequencies of the system [7, 10].

2.2.1.2 Irregular Structures

Those have significant physical discontinuities in configuration or in their lateral force resisting systems. Irregularity can be vertical or plan in irregularity.

2.2.1.3 Structural Irregularity of UBC and IBC

Table 2-2 Vertical Structural Irregularities in UBC and IBC

Irregularity Type and Description	
1a.	Stiffness soft story irregularity: A soft story is one in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of three stories above.
1b.	Stiffness Extreme soft story irregularity: A soft story is one in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of three stories above.
2.	Weight (mass) irregularity: Mass irregularity shall be considered to exist where the effective mass of any story is more than 150% of the effective mass of adjacent story. A roof that is lighter than the roof below need not be considered.
3.	Vertical geometric irregularity: Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force resisting system in any story is more than 130% of that in adjacent story. One-story penthouses need not be considered.
4.	In-plane discontinuity in vertical lateral-force resisting elements: An in-plane offset of the lateral-load resisting elements greater than the length of those elements.
5a.	Discontinuity in capacity weak story: A weak story is one in which the story strength is less than 80% of that in the story above. The story strength is the total strength of all seismic-resisting elements shearing the story shear for the direction under consideration.
5b.*	Discontinuity in capacity weak story: A weak story is one in which the story strength is less than 80% of that in the story above. The story strength is the total strength of all seismic-resisting elements shearing the story shear for the direction under consideration.

Table 2-3 Horizontal Structural Irregularities in UBC and IBC

Irregularity Type and Description	
1a.	Torsion irregularity: Torsional irregularity shall be considered to exist when the maximum story drift, computing including accidental torsion, at one end of the structural transverse to an axis is more than 1.2 times the average of the story drift of the two ends of the structure in which the diaphragms are rigid or semi-rigid.
1b.*	Extreme Torsion irregularity: Torsional irregularity shall be considered to exist when the maximum story drift, computing including accidental torsion, at one end of the structural transverse to an axis is more than 1.4 times the average of the story drift of the two ends of the structure in which the diaphragms are rigid or semi-rigid.
2.	Re-entrant corner: Plane configuration of a structure and its lateral force resisting system contain re-entrant corners, where both projection of the structure beyond a re-entrant corner are greater than 15% of the plan dimension of the structure in the given direction.
3.	Diaphragm discontinuity: Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open area greater than 50% of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50% from one story to the next.
4.	Out of plane offsets: Discontinuity in the lateral force path, such as out of plane offsets of the vertical elements.
5.	Nonparallel system: The vertical lateral load resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force resisting system.

* Not in UBC 97.

2.2.1.4 Structural Irregularity of EC8 and EBCS-8

Table 2-4 Plan Structural Regularities in EC8 and EBCS-8

Criteria for Regularity in Plan	
1.	„Approximately“ symmetrical distribution of mass and stiffness in plan.
2.	Compact plane configuration: reentrant length in one direction < 25% (EBCS-8) or re-entrant area < 5% (EC8) of the total plan area.
3.	In-plane stiffness of floors sufficiently large compared to stiffness of vertical elements.
4.*	The ratio of longer side to shorter sides in plan does not exceed four.
5.	For each direction the torsional radius, r , must exceed 3.33 times the structural eccentricity, e_o , and the radius of gyration of the floor mass in plane, I_s .(EC8). Under the equivalent static seismic force, maximum displacement in the direction of seismic force does not exceed average displacement by 20% (EBCS-8).

Table 2-5 Elevation Structural Regularities in EC8 and EBCS-8

Criteria for Regularity in Elevation	
1.	All lateral load resisting systems run without interruption from foundation to top.
2.	Both lateral stiffness & mass of storey remain constant or reduce gradually without abrupt change.
3.	Ratio of actual storey resistance to required resistance should not vary disproportionately between adjacent stories.
4	Buildings with setbacks should be less than the limits in the code. The limits broadly speaking are a total reduction in width from top to bottom on any face not exceeding 30%, with not more than 10% at any level compared to the level below. However, an overall reduction in width of up to half is permissible within the lowest 15% of the height of the building.

* Not in EBCS8.

For EC8 and EBCS8 vertical regularity number 2 and 3 in table 2-5 quantification are not provided. However in UBC 97 and IBC qualifications are provided.

2.4 Base Shear

In static analysis procedure, equivalent static force applied at the story levels replace the time varying inertia forces. The relative magnitudes of these equivalent static forces are based on simplifying assumption for mode shapes and mode participation. A comparison of base shear is the simplest way of comparing the final result. The lateral force provisions of various codes are presented in table 2-6 for comparison. As shown in table 2-6 that the base shear of an elastic structure, designed following code provisions and subjected to a seismic ground motion, can be characterized by zone factor, important factor, structural system factor, soil profile dependent numerical coefficient and time period. These factors have been compared in the following sections.

Table 2-6 Design Base Shear Formulas of UBC, IBC, EC8, and EBCS-8

Code	Base shear
UBC	$V = \frac{C_v I}{R T} W$ <p>V, is not exceed the following: $V = \frac{2.5 C_a I}{R} W$ <p>V, shall not be less than the following: $V = 0.11 C_a I W$ <p>In addition, for seismic zone 4, the total base shear shall also not be less than the following: $V = \frac{0.8 Z N v I}{R} W$</p> </p></p>
IBC	$V = C_s W$ $C_s = \frac{S_{DS}}{\left(\frac{R}{T}\right)} \quad \text{for } 0.0 < T < T_S$ $C_s = \frac{S_{D1}}{T \left(\frac{R}{T}\right)} \quad \text{for } T_S < T \leq T_L$ $C_s = \frac{S_{D1}}{T^2 \left(\frac{R}{T}\right)} \quad \text{for } T > T_L$ <p>C_s shall not be less than $C_s = 0.044 S_{DS} > 0.01$, it provide a strength approximately 3% of the weight And, The structure located where $S_1 \geq 0.6g$, C_s shall not be less than $C_s = \frac{0.5 S_1}{\left(\frac{R}{T}\right)}$</p>
EC8	$F_b = S_d(T_1) m \lambda$ <p>For, $0 \leq T \leq T_B$: $S_d(T) = \alpha_g S \left[\frac{2}{3} + \frac{T}{T_B} \left(\frac{2.5}{q} - \frac{2}{3} \right) \right]$ For, $T_B \leq T \leq T_C$: $S_d(T) = \alpha_g S \frac{2.5}{q}$ For, $T_C \leq T \leq T_D$: $S_d(T) = \begin{cases} = \alpha_g S \frac{2.5}{q} \left[\frac{T_C}{T} \right] \\ \geq \beta \alpha_g \end{cases}$ For, $T_D \leq T$: $S_d = \begin{cases} = \alpha_g S \frac{2.5}{q} \left[\frac{T_C T_D}{T^2} \right] \\ \geq \beta \alpha_g \end{cases}$</p>
EBCS-8	$F_b = S_d(T_1) W$ $S_d(T_1) = \alpha \beta \gamma$ $\alpha = \alpha_0 I$ $\beta = \frac{1.2 S}{T^{2/3}} \leq 2.5$

2.4.1 Zone Factor

Zone factor expresses the zone seismicity, generally in terms of effective peak ground acceleration. Most codes the value of zone factor described in terms of a single parameter, i.e. the value of the reference peak ground acceleration on rock soil, others include site spectral

amplification. The direct comparison between zone factor values recommended by different codes is difficult since the seismicity of an area for zoning purpose is determined primarily by the historical records of an earthquakes, location, length, and estimated activity of earthquake faults in the region concerned [11].

Ordinarily any seismic code that employs a zone factor is used in conjunction with a map showing the geographical extent of each zone of a particular seismic intensity. In most cases countries are divided geographically in to different seismic zones, often with significant change in value as a boundary between zones is crossed. However, if a hazard map is produced with a high enough spatial resolutions, then changes in hazards over small distances are always relatively subtle. Because of this, some current codes adopt earthquake ground motions based on a set of rules that depend on the seismicity of an individual region [12].

It must be stressed that seismic zone of a country is not a static endeavor. The philosophy of changing seismic zone is necessary. This should be guided by the changing idea on the methodology of analysis, available data and improving knowledge of the tectonic of the country. The changes are related in the refinement in the knowledge of the location and the area extent of tectonic features and their correlation with particular seismic events [11].

The biggest difference of IBC from other three codes is in the design ground motion parameters. UBC, EC8 and EBCS-8 use seismic zone map giving seismic zone values. However, in IBC the seismic zone maps has been replaced by contour maps giving mapped values of the spectral response acceleration at short periods, S_S , and at 1 second, S_I , for rock Sites. From which S_{DS} and S_{DI} are derived. The design earthquake of the IBC is two-thirds of the maximum considered earthquake (MCE), has a return period of 2500-years (two-percent probability of exceedance in 50 years), whereas the design earthquake of UBC and EC8 have an approximate return period of 475 years (ten-percent probability of exceedance in 50 years). Contrary, EBCS-8 has return period of 100 years only.

S_{DS} and S_{DI} are two-thirds of S_{MS} and S_{MI} which are the soil modified MCE spectral response accelerations at short period and 1-second period, respectively. S_{MS} is obtained by multiplying the mapped MCE spectral response acceleration, S_S , by F_a (table 2-13), the acceleration related soil factor. S_{MI} is similarly obtained by multiplying the mapped MCE spectral response acceleration, S_I , by F_V (table 2-14) [13].

The 1997 UBC introduced two near- source factors for the first time: acceleration-related, N_a , and velocity related, N_v . The purpose of which is to increase the soil modified ground motion parameters C_a and C_v when there are active faults capable of generating large-magnitude earthquakes within 15 km of a Seismic Zone 4 sites. These factors became necessary in view of the artificial truncation of Z -values to 0.4 in UBC Seismic Zone 4. These near-source factors are not found in the IBC because the artificial truncation of ground motion is not a feature of this code. Both S_S and S_I attain high values in the vicinity of seismic sources that are judged capable of generating large earthquakes. Near source effects are not also considered in EC8 and EBCS-8.

2.4.1.1 Seismic Design Category (SDS)

The IBC uses Seismic Design Categories (SDC) to determine permissible structural systems, limitations on height and irregularity, the type of lateral force analysis that must be performed, quality assurance, specialized requirements, the level of detailing for structural members and joints that are part of the lateral-force-resisting system and for the components that are not. The 1997 UBC and other two codes, EC8 and EBCS-8 utilize the Seismic Zone in which a structure is located for some these purposes. The Seismic Design Category is a function of occupancy and of soil modified seismic risk at the site of the structure. For the purposes of detailing as well as the other restrictions, the UBC Seismic Zones and the corresponding IBC Seismic Design Categories shown in Table 2-7 may be considered to be approximately equivalent. Table 2-8 to 2-11 show the seismic zone factor and the seismic design category defined by UBC, IBC and EBCS-8.

Table 2-7 Approximate Equivalency between UBC Seismic Zones and IBC SDC

1997 UBC Seismic Zone	0,1	2A, 2B	3, 4
2000 IBC Seismic Design Category	A, B	C	D, E, F

Table 2-8 Seismic Zone Factor of Uniform Building Code, UBC

Zone	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

Table 2-9 Bed Rock Acceleration Ratio, α_o , EBCS-8

Zone	1	2	3	4
α_o	0.03	0.05	0.07	0.1

Table 2-10 Seismic Design Category Based on Short-Period Response S_{DS} , IBC

	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g < S_{DS} < 0.33g$	B	B	C
$0.33g < S_{DS} < 0.5g$	C	C	D
$0.5g \leq S_{DS}$	D	D	D
mapped $S_1 \geq 0.75g$	E	E	F

Many near-fault sites have $S_1 \geq 0.75g$

Table 2-11 Seismic Design Category Based on 1-Second Response S_{D1} , IBC

	Occupancy Category		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g < S_{D1} < 0.133g$	B	B	C
$0.133g < S_{D1} < 0.2g$	C	C	D
$0.2g \leq S_{D1}$	D	D	D
mapped $S_1 \geq 0.75g$	E	E	F

Many near-fault sites have $S_1 \geq 0.75g$

2.4.2 Soil Classification and Site Amplification Factor

Local geology and soil characteristics influence the ground motions at a site. Contemporary seismic codes have largely accepted the significant role of site effect and attempt to incorporate their influence either by means of a constant amplification factor exclusively depend on the soil class or include additional parameters like shaking intensity, near field condition effect.

2.4.2.1 Soil Classification

Even though concerning site classification different approaches exist, the basic idea of the mean value of shear wave velocity, $V_{s,30}$, over the last few decades of meter (30m) is considered to be ground parameter for site classification [1]. However soil classification exclusively based in terms of $V_{s,30}$ assumption is a rather simplified hypothesis, especially in case of deep soil formations or abrupt stiffness change between the soil layer at 30m and bed rock layer depth. Table 2-10 compares soil classification schemes in modern seismic codes worldwide applying the $V_{s,30}$ criteria.

2.4.2.2 Site Amplification Factor

In UBC 97 a soil profile type accompanied with seismic zone factor gives seismic coefficient,

Table 2-12 Ground types defined in the UBC, IBC, EC8 and EBCS-8

UBC, IBC		EC8		EBCS8	
Ground types	Description	Ground types	Description	Ground types	Description
S _A	Hard rock V > 1500 m/s	A	Rock or rock-like geological formation including most 5 m weaker material at the surface V _{s,30} > 800 m/s	A	Rock V _s ≥ 800 m/s in the top 5m and stiff clay deposits V _s ≥ 400 m/s at 10m depth.
S _B	Rock V _s ≈ 760~1500				
S _C	Very dense soil or soft rock V _s ≈ 360~760 N _{spt} > 50 Cu > 100	B	Deposit of very dense sand, gravel or very stiff clay, at least several tens of m in thicknesses, characterized by a gradual increase of mechanical properties with depth. V _{s,30} ≈ 360~800 N _{spt} > 50, Cu > 250		
S _D	Stiff soil V _s ≈ 180~360 N _{spt} = 15-50 Cu = 50-100	C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of m V _{s,30} ≈ 180~360 N _{spt} =15-50, Su=70-250	B	medium dense sand, gravel or medium stiff clays V _s ≥ 200 m/s at 10m depth.
S _E	Soft Soil V _s < 180 N _{spt} < 15 Su < 50	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,30} < 180	C	Loose cohesionless soil deposits with or without some soft cohesive layers V _s < 200 m/s in the uppermost 20m.
S _F	Soil requiring site specific evaluation	E	A soil profile consisting of a surface alluvium layer with V _{s,30} values of class C or D and thickness varying between about 5m and 20m, underlain by stiffer material with V > 800 m/s		
		S ₁	Deposits consisting or containing a layer at least 10m thick of soft clays/silts with high plasticity index (PI>40) and high water content, V _{s,30} < 100 m/s Cu = 10 – 20		
		S ₂	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A-E or S1		

S_A, S_B, S_C, S_D, S_E and S_F given in the UBC are symbolized with A, B, C, D, E, and F, respectively, the IBC. N_{SPT} (blows/30cm), and Cu (kPa)

C_a and C_v . In IBC 2009 the effect of the site express in terms of site coefficients, F_a and F_v , which are a function of mapped spectral response acceleration and site class. However If S_{DS} of IBC is equal to $2.5C_a$ of UBC 97 and S_{DI} of IBC is equal to C_v of UBC 97 for a particular location, then the soil-modified seismicity for that site has not changed from the 1997 UBC to IBC. It will be seen that the EBCS-8 adopt a constant amplification factor exclusively depend on soil class. However, EC8 has two amplification factors for each soil type depends on the seismicity of the area. Table 2-13 and 2-14 are seismic response coefficient of UBC, table 2-15 and 2-16 lists the site coefficients of IBC and table 2-17 show soil factors of EC8 and EBCS-8.

Table 2-13 Acceleration Related Seismic Coefficient C_a

Soil profile type	Seismic zone factor, Z				
	$z = 0.075$	$z = 0.15$	$z = 0.2$	$z = 0.3$	$z = 0.4$
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	Site-specific geotechnical investigation and dynamic site response shall be performed				

Table 2-14 Velocity Related Seismic Coefficient C_v

Soil profile type	Seismic Zone Factor, Z				
	$z = 0.075$	$z = 0.15$	$z = 0.2$	$z = 0.3$	$z = 0.4$
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	Site-specific geotechnical investigation and dynamic site response shall be performed				

Table 2-15 Value of Acceleration Related Site Coefficient F_a

site class	Mapped Spectral Response Acceleration at Short Period				
	$S_s \leq 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
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	Site specific analysis needed				

Note: Use straight-line interpolation for intermediate values.

Table 2-16 Velocity Related Site Coefficient, F_v

site class	mapped spectral response acceleration at 1-second period				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
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	Site specific analysis needed				

Site class F: A site response analysis is performed and the soil is vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils.

Table 2-17 Soil Factor (Site Coefficient) of EC8 and EBCS-8

EC8			EBCS-8	
Ground types	S For type 1 spectra (for areas of high seismicity) ($M_s > 5.5$)	S For Type 2 spectra (for areas of moderate seismicity) ($M_s \leq 5.5$).	Ground types	S
	A	1.0		
B	1.2	1.35	C	1.5
C	1.15	1.5		
D	1.35	1.8		
E	1.4	1.6		

2.4.3 Importance Factor

The importance factor is applied in most codes to reflect the need to protect essential facilities that must operate after earthquakes, such as hospital, fire stations, civil defense headquarters, and it also applied to buildings whose collapse could cause unusual hazard to the public, such as facilities storing toxic chemicals. The implication is to reduce the acceptable probability of occurrence of the design earthquake [12]. However, it must be recognized that added strength, by itself, is not adequate to provide for superior seismic performance in buildings with critical occupancies. Good connections and construction details, quality assurance procedures, and limitations on building deformation or drift are also important to significantly improve the capability for maintenance of function and safety in critical facilities and those with high density occupancy [5]. Table 2-18 compares the range of important factor relevant to UBC 97, IBC 2009, EUROCODE 8, and EBCS-8codes.

In all codes considered here, except EC8, practice the importance factors are applied to the-

Table 2-18 Importance Factor Range of UBC, IBC, EC 8, and EBCS-8 Codes

Code	Range of Importance Factor
UBC	1 – 1.25
IBC	1 – 1.5
EC8	0.8 – 1.4
EBCS-8	0.8 – 1.4

-seismic loads. However, in EC8 they are applied to the input motions. This makes an important difference when non-linear analysis is employed, since increasing the ground motions by X percent may cause an increase of less than X percent in forces, due to yielding of elements, but (possibly) more than X percent in deflections, due to plastic strains and P-delta effects [1].

2.4.4 Force Reduction Factor

It would, in most cases, be economically prohibitive to design a building so that it remains elastic at all levels of earthquake ground motions. A fundamental tenet of seismic design is that yielding is allowed to accommodate seismic loading as long as such yielding does not impair the vertical load capacity of the structure. In other words, damage is allowed in the maximum expected earthquake loading case only if it does not pose a significant probability of the structure's collapse. Designing structures taking into account of non-linear seismic response, a variety of analysis options are available. The simplest and most widely used approach is to use the linear analysis methods with the design forces reduced on the basis of a single, global behaviour factor. Codes recommended force reduction values for common structural system. This approach is most suitable for regular structures, where inelasticity can be expected to be reasonably uniformly distributed [14, 5]. In table 2-19 the range of structural system factor of different codes have been listed.

Table 2-19 Force Reduction Factors of UBC, IBC, EC8 and EBCS-8 Codes

Code	Coefficient	Most Ductile	Usual Structure
UBC	1/R	R = 8.5	R= 2.2
IBC	1/R	R = 8	R = 1.25
EC8	1/q	q = 5	q = 1.5
EBCS-8	γ	$\gamma = 0.17$	$\gamma = 0.7$

2.4.5 Time Period

The equivalent static methods adopt seismic coefficient, which is depends on the natural period of their vibration, for earthquake resistance design of the structures to calculate the

base shear. Codes of practice proposed approximate formula for the determination of period of structures. The empirical formulas of UBC, IBC, EC8, and EBCS-8 are tabulated in table 2-20.

Evidence from table 2-20, the empirical period determinations of all codes are similar with some exceptions. First, IBC gives an alternative formula for the determination of approximate fundamental period of concrete and steel moment resisting frame buildings not exceeding twelve stories in height and having a minimum story height of 10ft. This formula is not included in the other codes. Second, an alternative estimation of the period ($T_1 = 2\sqrt{d}$) is give by EC8 and EBCS-8 not in UBC and IBC.

All equations in table 2-20 except method B of UBC are empirical relationship determined through statistical analysis of the measured response of buildings. Since this empirical expression is based on the lower bound of the data, it produces a lower bound for the period of a building of given height [14]. This lower bound period provides a conservative estimate of base shear. Due to this rational period provided in the UBC and IBC. However it is not the feature of EC8 and EBCS-8.

Rational period computation is also the same in the IBC and the UBC (Method B) except that the UBC specifically includes one particular rational computation procedure, as given in table 2-20 method B, which is not included in the IBC. The limits on rationally computed period are different in the two codes. The IBC limits the rationally computed period to be no larger than C_u times the approximate period. The 1997 UBC limits rational period to be no larger than 1.4 times the approximate period in Seismic Zones 1, 2, and 3, and no larger than 1.3 times the approximate period in Seismic Zone 4.

2.4.6 Base Shear Distribution

The way the base shear is distributed over the height of the building is a function of the fundamental mode shape. The fundamental mode shapes in the horizontal directions approximated by horizontal displacements increasing linearly along the height of the building.

As shown in table 2-21 IBC prescribes a linear distribution and a parabolic distribution, for structures with $T \leq 0.5s$ and $T \geq 2.5s$, respectively. The UBC provide a linear distribution, with zero value at the base, for structures with $T \leq 0.7s$.

Table 2-20 period estimates of UBC, IBC, EC8, and EBCS-8

Code	Formula suggested																											
UBC	<p>Method A $T = C_t(h_n)^{3/4}$</p> <p>Method B $T = 2\pi \sqrt{(\sum_{i=1}^n w_i \delta_i^2) \div (g \sum_{i=0}^n f_i \delta_i)}$</p> <p>C_t = 0.0853 for steel moment-resisting frames C_t = 0.0731 for reinforced concrete MRF and essentially braced frames C_t = 0.0488 for an other buildings</p> <p>Alternatively, $C_t = 0.0743/\sqrt{A_c}$ for structures with concrete or masonry shear wall $A_c = \sum A_e [0.2 + (D_e/h_n)^2]$</p>																											
IBC	<p>$T_a = C_t h_n^x$</p> <table border="1" style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th style="text-align: center;">Structure type</th> <th style="text-align: center;">C_t</th> <th style="text-align: center;">x</th> </tr> </thead> <tbody> <tr> <td>Steel moment resisting frames</td> <td style="text-align: center;">0.0724</td> <td style="text-align: center;">0.8</td> </tr> <tr> <td>Concrete moment resisting frames</td> <td style="text-align: center;">0.0466</td> <td style="text-align: center;">0.9</td> </tr> <tr> <td>Eccentrically braced steel frames</td> <td style="text-align: center;">0.0731</td> <td style="text-align: center;">0.75</td> </tr> <tr> <td>All other structural systems</td> <td style="text-align: center;">0.0488</td> <td style="text-align: center;">0.75</td> </tr> </tbody> </table> <p>$T_a = 0.1N$, for entire concrete or steel MRF and the average story height is at least 3 m For masonry or concrete shear wall</p> $T_a = \frac{0.0019}{\sqrt{C_w}} h_n$ $C_w = \frac{100}{A_B} \sum_{i=1}^x \left(\frac{h_n}{h_i}\right)^2 \left[\frac{A_i}{1 + 0.83 \left(\frac{h_i}{D_i}\right)^2} \right]$ <p>Coefficients for upper limit on calculated period</p> <table border="1" style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th style="text-align: center;">Design spectral response acceleration parameter at 1s, S_{D1}</th> <th style="text-align: center;">Coefficient C_U</th> </tr> </thead> <tbody> <tr> <td style="text-align: center;">>0.4</td> <td style="text-align: center;">1.4</td> </tr> <tr> <td style="text-align: center;">0.3</td> <td style="text-align: center;">1.4</td> </tr> <tr> <td style="text-align: center;">0.2</td> <td style="text-align: center;">1.5</td> </tr> <tr> <td style="text-align: center;">0.15</td> <td style="text-align: center;">1.6</td> </tr> <tr> <td style="text-align: center;">≤ 0.1</td> <td style="text-align: center;">1.7</td> </tr> </tbody> </table> <p>The fundamental period, T, shall not exceed the product of C_U and T_a</p>	Structure type	C _t	x	Steel moment resisting frames	0.0724	0.8	Concrete moment resisting frames	0.0466	0.9	Eccentrically braced steel frames	0.0731	0.75	All other structural systems	0.0488	0.75	Design spectral response acceleration parameter at 1s, S _{D1}	Coefficient C _U	>0.4	1.4	0.3	1.4	0.2	1.5	0.15	1.6	≤ 0.1	1.7
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0.3	1.4																											
0.2	1.5																											
0.15	1.6																											
≤ 0.1	1.7																											
EC8 and EBCS-8	<p>$T = C_t(h_n)^{3/4}$</p> <p>C_t = 0.085 for moment-resistant space steel frames C_t = 0.075 for moment-resistant concrete space frames C_t = 0.050 for an other structures</p> <p>For the structures with concrete or masonry shear wall $C_t = 0.075/\sqrt{A_c}$ $A_c = \sum [A_i(0.2 + (L_{wi}/H))^2]$ $L_{wi}/H \leq 0.9$ $T = 2\sqrt{d}$</p>																											

For longer-period structures, a portion of the design base shear ($0.07TV \leq 0.25V$) is concentrated at the top, with the remainder of the design base shear being distributed linearly as for short-period structures. In EBCS-8, somewhat similar to UBC, a portion of the design base shear ($0.07TV$) is concentrated at the top regardless of the period of the structure. The concentrated force at the top is an allowance made for higher mode effects. The parabolic distribution of the IBC also shifts more forces towards the top, thereby increasing overturning effects. The EC8 uses a linear distribution for all time periods which is non-conservative. However, EC8 permit equivalent static analysis up to a structural height of 40m only.

Table 2-21 Vertical Distribution of the Base Shear of UBC, IBC, EC8 and EBCS-8

Code	Base Shear and Its Distribution								
UBC	$V = F_t + \sum_{i=1}^n F_i$ $F_t = 0.07TV \leq 0.25V$ $F_t = 0, \text{ if } T \leq 0.7s$ $F_x = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^n w_i h_i}$								
IBC	$F_x = C_{vx} V$ $C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k}$ <table border="1" style="margin-left: auto; margin-right: auto;"> <thead> <tr> <th style="text-align: center;">T</th> <th style="text-align: center;">K</th> </tr> </thead> <tbody> <tr> <td style="text-align: center;">$\leq 0.5s$</td> <td style="text-align: center;">1</td> </tr> <tr> <td style="text-align: center;">$\geq 2.5s$</td> <td style="text-align: center;">2</td> </tr> <tr> <td style="text-align: center;">$0.5s < T < 2.5s$</td> <td style="text-align: center;">2 or $0.5 + 0.75T$</td> </tr> </tbody> </table>	T	K	$\leq 0.5s$	1	$\geq 2.5s$	2	$0.5s < T < 2.5s$	2 or $0.5 + 0.75T$
T	K								
$\leq 0.5s$	1								
$\geq 2.5s$	2								
$0.5s < T < 2.5s$	2 or $0.5 + 0.75T$								
EC8	$F_x = F_b \frac{s_i m_i}{\sum_{j=1}^n s_j m_j}$ <p>Or when the fundamental mode shape is approximated by horizontal displacements increasing linearly along the length</p> $F_i = F_b \frac{z_i m_i}{\sum_{j=1}^n z_j m_j}$								
EBCS-8	$F_b = F_t + \sum_{i=1}^n F_i$ $F_t = 0.07T_1 F_b$ $F_x = \frac{(F_b - F_t)w_i h_i}{\sum_{i=1}^n w_i h_i}$								

2.4 Torsion

Possible torsional ground motion, the unpredictable distribution of live load mass, the variations of structural properties are three reasons why both regular and irregular structures

must be designed for accidental torsional loads [10]. All four codes, included in this study, have the same torsion design provisions for non-flexible diaphragm situations. The torsion be included in design is the actual torsion (inherent torsion) plus accidental torsion.

Table 2-22 Accidental Torsional Effects by UBC, IBC, EC8 and EBCS-8

Code	Accidental Torsional effect
UBC and IBC	Accidental torsion is considered by applied earthquake forces at displacement of e_{Li} from the center of mass) $e_{Li} = \pm 0.05L_i$ If torsional irregularity exist, amplifying the accidental torsion by $A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}} \right]^2 \leq 3.0$
EC8 and EBCS-8	Accidental torsion is considered by applied earthquake forces at displacement of e_{Li} from the center of mass. $e_{Li} = \pm 0.05L_i$ If lateral stiffness and mass are symmetrically distributed in plan and unless the accidental eccentricity is taken into account by more exact method, the accidental torsional effect may be accounted by multiplying the action effects in the individual load resisting element by a factor δ given by $\delta = 1 + 0.6 \frac{x}{L_e}$ If torsional irregularity exist, amplifying the accidental torsion by $A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}} \right]^2 \leq 3.0$

For all codes considered here the amount of accidental torsion included and the amplification of accidental torsion for torsionally irregular buildings are the same. However, the IBC requires the amplification for structures assigned to Seismic Design Category C, D, E or F only. In addition, if the lateral stiffness and mass are symmetrically distributed in plan, EC8 and EBCS-8 allow an alternative approach to account accidental torsional, as shown above in table 2-22.

2.5 Story Drift Determination and Limitation

Story drift is the maximum lateral displacement within a story (i.e., the displacement of one floor relative to the floor below caused by the effects of seismic loads). There are many reasons for controlling drift; it is use to control member inelastic strain, to control flexibility by providing the upper limit of P-delta effect and to restrict damage [5]. The design story-

Table 2-23 Story Drift Limitations by UBC 97, IBC 2009, EUROCODE 8 and EBCS-8

Code	Story Drift																							
UBC	<p>The Maximum Inelastic Response Displacement, Δ_m</p> $\Delta_m = 0.7R\Delta_s$ $\Delta_m \leq 0.025H \text{ for } T < 0.7s$ $\Delta_m \leq 0.020H \text{ for } T \geq 0.7s$ <p>Δ_m, The maximum inelastic response displacement Δ_s, the design level response displacement</p>																							
IBC	<p>The deflection at level x, δ_x</p> $\delta_x = \frac{C_d \delta_{xe}}{I}$ $\delta_x \leq \Delta_a^{a,b}$ <p>C_d = the deflection amplification factor δ_{xe} = the deflections determined by an elastic analysis.</p> <p>Allowable story drift, $\Delta_a^{a,b}$</p> <table border="1" data-bbox="344 792 1366 1106"> <thead> <tr> <th data-bbox="344 792 900 869" rowspan="2">Structure</th> <th colspan="3" data-bbox="900 792 1366 831">Occupancy category</th> </tr> <tr> <th data-bbox="900 831 1070 869">I or II</th> <th data-bbox="1070 831 1203 869">III</th> <th data-bbox="1203 831 1366 869">IV</th> </tr> </thead> <tbody> <tr> <td data-bbox="344 869 900 981">Structures, other than masonry shear wall , 4 story or less with interior and exterior wall</td> <td data-bbox="900 869 1070 981">0.025h_{sx}</td> <td data-bbox="1070 869 1203 981">0.020h_{sx}</td> <td data-bbox="1203 869 1366 981">0.015h_{sx}</td> </tr> <tr> <td data-bbox="344 981 900 1021">Masonry cantilever shear wall structure</td> <td data-bbox="900 981 1070 1021">0.010h_{sx}</td> <td data-bbox="1070 981 1203 1021">0.010h_{sx}</td> <td data-bbox="1203 981 1366 1021">0.010h_{sx}</td> </tr> <tr> <td data-bbox="344 1021 900 1061">Other masonry shear wall structures</td> <td data-bbox="900 1021 1070 1061">0.007h_{sx}</td> <td data-bbox="1070 1021 1203 1061">0.007h_{sx}</td> <td data-bbox="1203 1021 1366 1061">0.007h_{sx}</td> </tr> <tr> <td data-bbox="344 1061 900 1106">All other structures</td> <td data-bbox="900 1061 1070 1106">0.020h_{sx}</td> <td data-bbox="1070 1061 1203 1106">0.015h_{sx}</td> <td data-bbox="1203 1061 1366 1106">0.010h_{sx}</td> </tr> </tbody> </table>	Structure	Occupancy category			I or II	III	IV	Structures, other than masonry shear wall , 4 story or less with interior and exterior wall	0.025 h_{sx}	0.020 h_{sx}	0.015 h_{sx}	Masonry cantilever shear wall structure	0.010 h_{sx}	0.010 h_{sx}	0.010 h_{sx}	Other masonry shear wall structures	0.007 h_{sx}	0.007 h_{sx}	0.007 h_{sx}	All other structures	0.020 h_{sx}	0.015 h_{sx}	0.010 h_{sx}
Structure	Occupancy category																							
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All other structures	0.020 h_{sx}	0.015 h_{sx}	0.010 h_{sx}																					
EC8	<p>For the buildings having non structural element of brittle material attached to the structure $d_r v \leq 0.005h$</p> <p>For buildings having ductile non structural element $d_r v \leq 0.0075h$</p> <p>For building having non structural elements fixed in away so as not to interfere with structural deformations, or without non structural element: $d_r v \leq 0.010h$</p> <p>v is the reduction factor that take in to account the lower return period of seismic events. The recommended values of v are 0,4 for importance classes III and IV and $v = 0,5$ for importance classes I and II.</p> $d_r = \Delta d_s$ $d_s = q_d * d_e$ <p>d_r is the design inter storey drift, Δd_s is the difference of the average lateral displacements d_s at the top and bottom of the storey. q_d, the displacement behaviour factor d_e, the displacement determined by an linear analysis h, the story height</p>																							
EBCS-8	<p>For the buildings having non structural elements of brittle material attaches to the structure. $d_r \leq 0.010h$</p> <p>For buildings having non structural elements fixed in away as not to interface with structural deformation. $d_r \leq 0.015h$</p> $d_r = \Delta d_s, \quad d_s = \frac{d_e}{\gamma_d}$ <p>The definition of the terms are the same ae EC8</p>																							

-drift limits in different codes of Table 2-23 reflect consensus judgment taking into account the goals of drift control outlined above.

The IBC limits on story drift depend upon the seismic use group, and become tighter for higher use groups. The limits also depend on the type of structure. In the UBC 97, the drift limits are not dependent on occupancy, which is already effectively considered because Δ , is higher for structures in higher occupancy categories with an I of 1.25 assigned to them, and because the multiplier of $0.7R$ is independent of I . The UBC drift limits are dependent only on the period of the structure, drift limits being tighter for structures having periods greater than or equal to 0.7 seconds, although some exceptions to general rules are made. The EBCS-8 limit on story drift is equivalent to EC8 when the value of the reduction factor, ν , equal to 0.5 employed in EC8. However, EBCS-8 is more conservative for buildings having ductile non structural element than EC8. Generally for standard occupancy buildings, the EC8 and EBCS-8 UBC requirement is more stringent than UBC and IBC requirement. If we compare UBC and IBC, the IBC requirement is likely to be more stringent for periods less than or equal to 0.7 seconds; the requirements are likely to be comparable for longer-period buildings. For essential facilities, the IBC requirements are likely to be more stringent irrespective of the structural period.

2.6 Second Order Effect (P-Delta effect)

All four codes, included in this study allow P-delta effects to be disregarded as long as the secondary to primary moment ratio does not exceed 10 percent. The IBC, EC8 and EBCS-8 give a specific procedure to consider P-delta effects, the UBC does not. UBC has an additional drift-based criterion to neglect P-delta effects that is not included in the rest of the three codes.

Table 2-24 Second Order Effect of UBC 97, IBC 2009, EUROCODE 8, and EBCS-8

Code	Second Order Effect (P-Delata Effect)
UBC	P Δ need not be consider When the ratio, θ , of second moment to primary moment is less than or equal to 0.1 $\theta = \frac{M_S}{M_P} = \frac{(P_X \Delta)}{(V_X h)} \leq 0.1$ Also in seismic zones 3 and 4, P Δ need not be considered when story drift ratio $\leq \frac{0.02}{R}$
IBC	P Δ need not be consider, when $\theta = \frac{(P_X \Delta)}{(V_X h_{sx} C_d)} \leq 0.1$

	$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25$ <p>When $0.1 < \theta \leq \theta_{max}$ The incremental factor related to P-delta effect on displacement and member forces shall be determined by rational analysis or it is permitted to multiply displacement and member force by $1.0/(1-\theta)$ When $\theta > \theta_{max}$, the structure is potentially unstable and shall be redesigned.</p>
EC8	<p>Second order effect(P-Δ effect) It is not taken in to account if the following condition is fulfilled in all storys</p> $\theta = \frac{(P_{tot} d_r)}{(V_{tot} \cdot h)} \leq 0.1$ $\theta \leq 0.3$ <p>If $0 < \theta \leq 0.2$, the second order effects may be approximately be taken into accounted by multiplying the relevant seismic action effects by a factor equal to $1/(1-\theta)$</p>
EBCS-8	<p>Second order effect(P-Δ effect) It is not taken in to account if the following condition is fulfilled in all storys</p> $\theta = \frac{(P_{tot} d_r)}{(V_{tot} \cdot h)} \leq 0.1$ $\theta \leq 2.5$ <p>If $0 < \theta \leq 0.2$, the second order effects may be approximately be taken into accounted by multiplying the relevant seismic action effects by a factor equal to $1/(1-\theta)$</p>

CHAPTER THREE

COMPARISON OF DYNAMIC RESPONSE SPECTRUM ANALYSIS PROCEDURE

3.1 Introduction

Response spectrum is a practical means of characterizing ground motion and their effect on structures. Now a central concept in earthquake engineering, the response spectrum provides a convenient means to summarize the peak response of all possible linear single-degree-of-freedom (SDF) system to a particular component of ground motion. It also provides a particular approach to apply the knowledge of structural dynamics to the design of structures and development of lateral force requirements in building codes [3].

The main advantage of using the forces obtained from a dynamic analysis as the basis for a structural design is that the vertical distribution of forces may be significantly different from the forces obtained from an equivalent static load analysis. In the modal analysis procedure, the distribution is based on properties of the natural vibration modes which are determined from the mass and stiffness distribution. In the ELF procedure, the distribution is based on simplified formulas that are appropriate for regular structures [5]. Furthermore a response spectrum is suitable for the general earthquake direction.

The ELF procedure and the modal analysis procedure are both based on the approximation that the effects of yielding accounted by linear analysis using the response modification (behavioral) factor. The orthogonal effect, the vertical component of ground motion, and torsional motions of the structure are all considered in the same simplified approaches in the two procedures. Otherwise, the two procedures are subject to the same limitations. Both analysis procedures are all likely to err systematically on the unsafe side if story strengths are distributed irregularly over height. This feature is likely to lead to concentration of ductility demand in a few stories of the building [5].

3.2 Design Spectra

A plot of the peak value of a response quantity as a function of the natural vibration period, T_n , of the system, or a related parameter is called response spectrum for that quantity. All

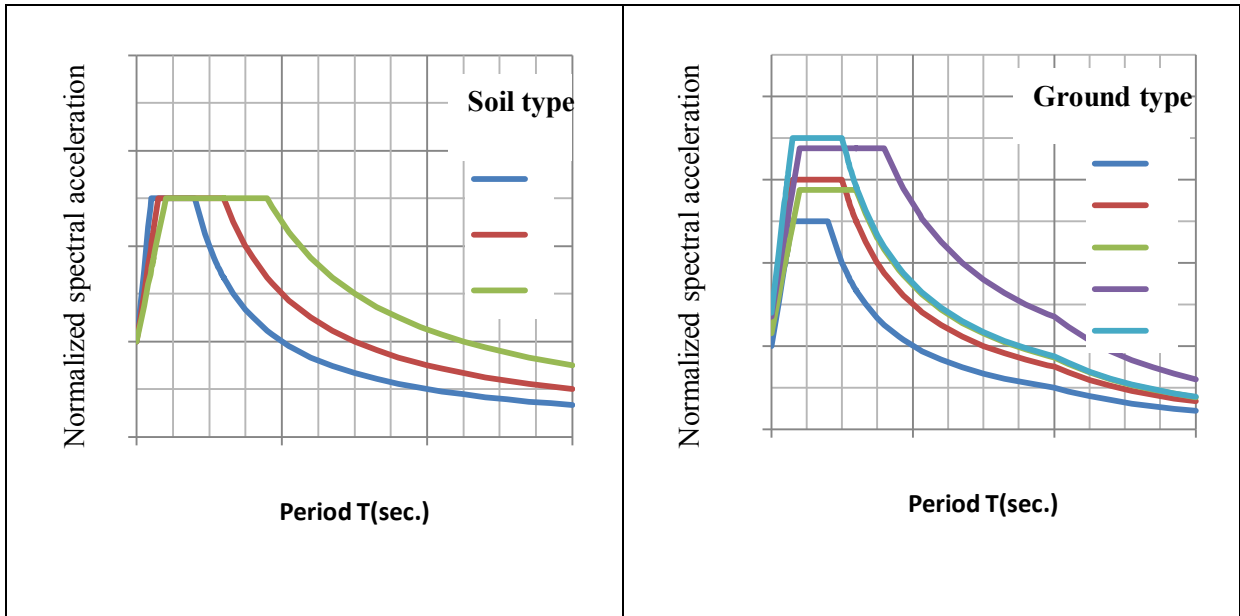
codes have defined specific equations for each range of the spectrum curve for all different soil types. The ordinates of elastic design spectra, S_e , and inelastic design spectra, S_d , for their return period defined by the earthquake can be determined using the expressions given in Table 3-1.

Table 3-25 Ordinates of Elastic and Inelastic Design Spectra for UBC, IBC, EC8 and EBCS-8

code	$T \leq T_B$	$T_B \leq T \leq T_C$	$T \geq T_C$
UBC 97	$S_e = \left[C_a + \frac{1.5C_a \cdot T}{T_B} \right] \cdot g$ $S_d = \left[C_a + \frac{1.5C_a \cdot T}{T_B} \right] \cdot g \cdot \frac{I}{R}$	$S_e = 2.5C_a \cdot g$ $S_d = 2.5C_a \cdot g \cdot \frac{I}{R}$	$S_e = \frac{C_v}{T} g$ $S_d = \frac{C_v}{T} g \cdot \frac{I}{R}$
IBC 2009	$S_e = \left[0.4S_{DS} + \frac{0.6S_{DS} \cdot T}{T_B} \right] \cdot g$ $S_d = \left[0.4S_{DS} + \frac{0.6S_{DS} \cdot T}{T_B} \right] \cdot g \cdot \frac{I}{R}$	$S_e = S_{DS} \cdot g$ $S_d = S_{DS} \cdot g \cdot \frac{I}{R}$	$T_C \leq T \leq T_D \rightarrow S_e = \frac{S_{D1}}{T} g$ $T_C \leq T \leq T_D \rightarrow S_d = \frac{S_{D1}}{T} g \cdot \frac{I}{R}$
			$T \geq T_D \rightarrow S_e = \frac{S_{D1} \cdot T_C}{T^2} g$ $T \geq T_D \rightarrow S_d = \frac{S_{D1} \cdot T_C}{T^2} g \cdot \frac{I}{R}$
EC8	$S_e = a_g S \left[1 + \frac{T}{T_B} (\eta 2.5 - 1) \right]$ $S_d = a_g S \left[\frac{2}{3} + \frac{T}{T_B} \left(\frac{2.5}{q} - \frac{2}{3} \right) \right]$	$S_e = 2.5\alpha_g \cdot S \cdot \eta$ $S_d = \frac{2.5}{q} \alpha_g \cdot S$	$T_C \leq T \leq T_D \rightarrow S_e = 2.5\alpha_g \cdot S \cdot \eta \cdot \left[\frac{T_C}{T} \right]$ $T_C \leq T \leq T_D \rightarrow S_d = \begin{cases} = \frac{2.5}{q} \alpha_g \cdot S \cdot \left[\frac{T_C}{T} \right] \\ \geq \beta \alpha_g \end{cases}$
			$T_D \leq T \leq 4s \rightarrow S_e = 2.5\alpha_g \cdot S \cdot \eta \cdot \left[\frac{T_C T_D}{T^2} \right]$ $T_D \leq T \rightarrow S_d = \begin{cases} = \frac{2.5}{q} \alpha_g \cdot S \cdot \left[\frac{T_C T_D}{T^2} \right] \\ \geq \beta \alpha_g \end{cases}$
EBCS-8	$S_e = \alpha_o + \frac{1.5\alpha_o T}{T_B}$ $S_d = \left[\alpha + \frac{1.5\alpha T}{T_B} \right] \cdot \gamma$	$S_e = 2.5\alpha_o$ $S_d = 2.5\alpha\gamma$	$S_e = \alpha_o \beta_o = \alpha_o \frac{S_o}{T}$ $S_d = \alpha \beta_o \gamma = \alpha \frac{S_o}{T} \gamma$

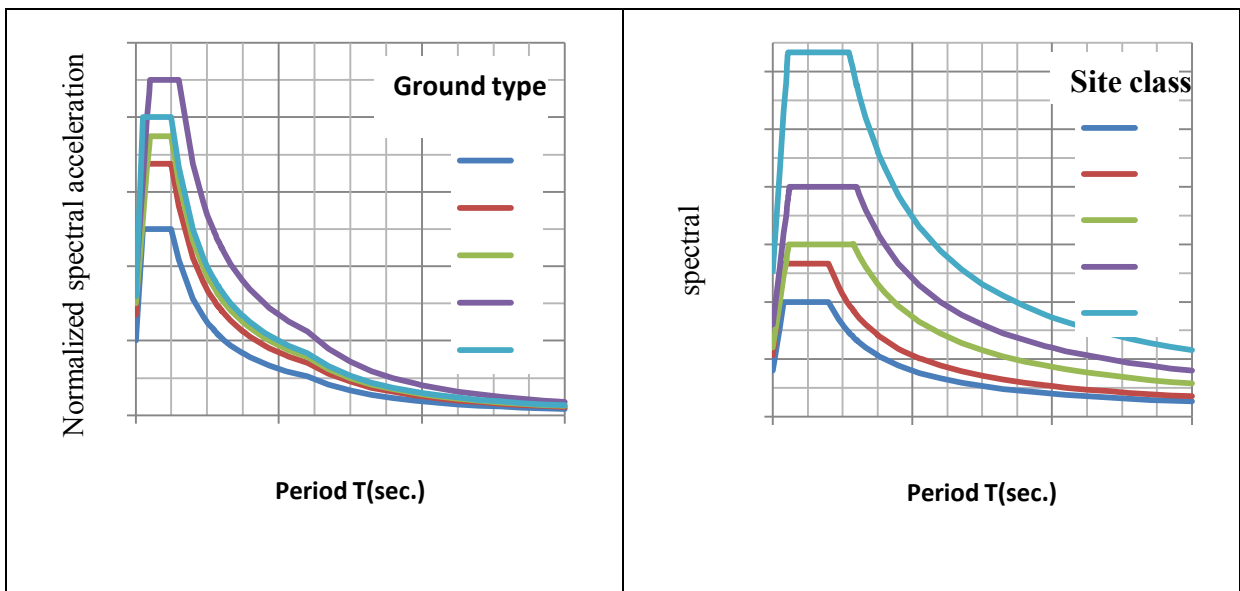
Elastic design spectra were drawn as shown in Figure 3-1 using the expressions shown in Table 3-1 for all ground types defined in the codes. Figure 3-1 shows that only EBCS-8 considers the same peak values for all ground types. The elastic response spectra of EC8 Type-2 and UBC 97 for low seismic zones are steeper for short period structures except for

rock soil type. Inelastic design spectra can be obtained considering the structure importance factor, the behaviour factor and the reference peak ground acceleration for sample structures and soil conditions.



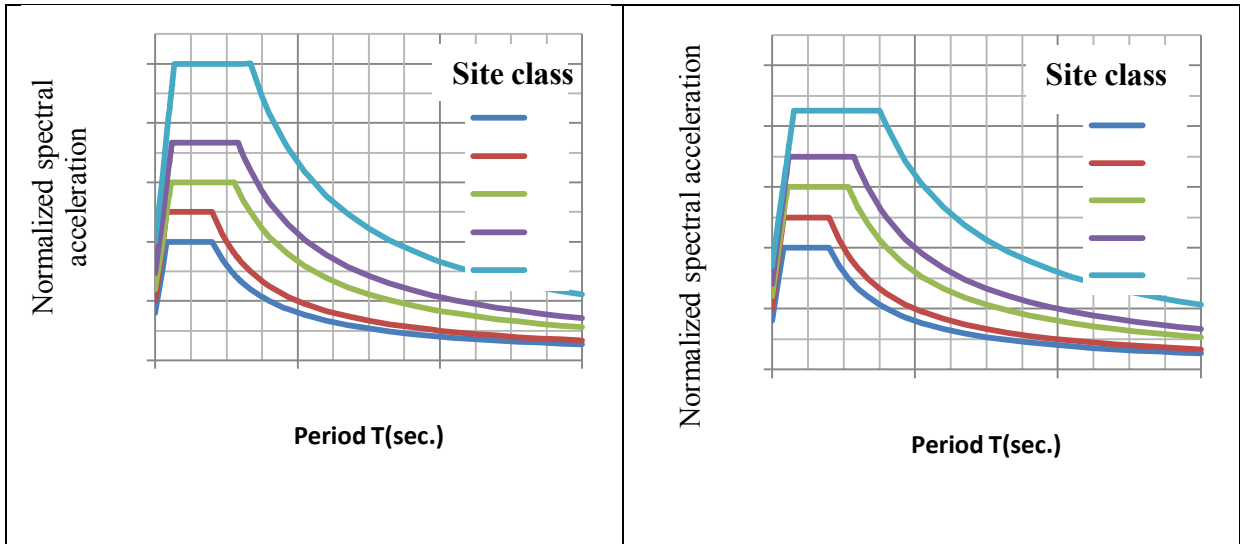
(a) EBCS-8 Elastic response spectra

(b) EC8 Type I Elastic response spectra



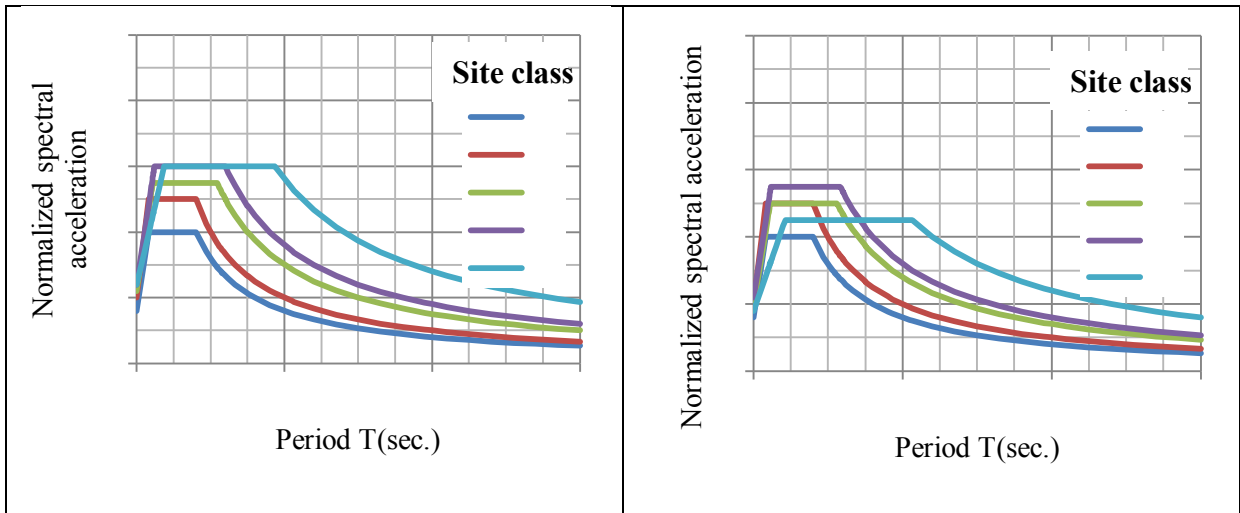
(c) EC8 type 2 Elastic response spectra

(d) UBC 97 Elastic response spectra for zone 1



(e) UBC 97 Elastic response spectra for zone 2A

(f) UBC 97 Elastic response spectra for zone 2B



(g) UBC 97 Elastic response spectra for zone 3

(h) UBC 97 Elastic response spectra for zone 4

FIGURE 3-1 Normalized Elastic Design Spectra Drawn for Ground Types Described in UBC, EC8 and EBCS-8.

3.3 Number of Modes

All the four codes, included in this study, have adopted similar provisions about the number of modes considered. All of them stated that, all significant modes shall be taken in to account may be satisfied by demonstrating that for the modes considered, at least 90% of the participating mass of the structure is included in the calculation of response for each principal direction. In addition EBCS-8 and EC8, unlike UBC 97 and IBC, specify additional requirements. First, all modes with effective modal masses greater than 5% of the total mass

taken into account. Second, if the above requirements cannot be satisfied the minimum number of modes, k , should be greater or equal to three times the square root of number of story and the period of vibration of mode k should be less or equal to 0.20s ($K \geq 3\sqrt{n}$ and $T_k \leq 0.20s$). The scaling required by section 3.5 controls the overall magnitude of design values so that incomplete mass participation does not produce non-conservative results.

3.4 Combination of Modal Response

Since the sign (positive or negative) and the time of occurrence of the maximum acceleration are lost in creating a response spectrum, there is no way to recombine modal responses exactly. However, statistical combination of modal responses produces reasonably accurate estimates of displacements and component forces [16].

UBC 97 generally stated that, the peak member forces, displacements, story forces, story shears and base reactions for each mode shall be combined by recognized methods. When three dimensional models are used for analysis, modal interaction effects shall be considered when combining modal maxima. The value for each parameter of interest calculated for the various modes shall be combined using the square root of the sum of the squares (SRSS) method or the complete quadratic combination (CQC) method, or an approved equivalent approach. The CQC method shall be used for each of the modal values where closely spaced modes have significant cross correlation of translational and torsional response. In a similar fashion, EC8 and EBCS-8 specify the same modal combination rules (SRSS and CQC) methods. However, both codes clearly specify the closely spaced (dependent) modes. According to the provisions two vibration modes may be taken as independent of each other, if the period of one mode is less or equal to 90% of the period of the other.

SRSS method is the simple and widely acceptance modal combination rule. This method assumes that all of the maximum modal values are statistically independent. For three dimensional structures a large number of frequencies are almost identical. In this case The CQC method should be used to combine modal maxima in order to minimize the introduction of avoidable errors. The CQC method has a sound theoretical basis and has been accepted by most experts in earthquake engineering. The use of the absolute sum or the SRSS method for modal combination cannot be justified [16].

3.5 Scaling Design Values of Combined Response

The current code allows the results obtained from a dynamic analysis to be normalized so that the maximum dynamic base shear is equal to the base shear obtained from a simple two-dimensional static load analysis. Even if, most members of the profession realized there is no theoretical foundation for this approach.

The modal base shear, V_i , may be less than the ELF base shear, V , because: (a) the calculated fundamental period may be longer than that used in computing V (b) the response is not characterized by a single mode, and (c) the ELF base shear assumes 100% mass participation in the first mode. The scaling required providing a minimum base shear for design. This minimum base shear is provided because the computer model often neglects nonstructural stiffness. the computed period of vibration may be the result of an overly flexible (incorrect) analytical model [14].

The scaling provisions of UBC and EBCS-8 are similar. Where the combined response for the modal base shear is less than the calculated static base shear, the forces shall be normalized to 100% of static base shear for irregular buildings and to 90% for regular buildings. IBC, unlike UBC and EBCS-8, specify magnifying to 85% for both regular and irregular buildings. The possible 15 percent reduction in design base shear may be considered as an incentive for using a modal response spectrum analysis in lieu of the equivalent lateral force procedure. In EC8 there is no scaling provision.

3.6 Directional and Orthogonal Effects

A well-designed structure should be capable of equally resisting earthquake motions from all possible directions. It is reasonable to assume that motions that take place during an earthquake have one critical direction. For most structures this direction is not known. Therefore, the only rational earthquake design criterion is that the structure must resist an earthquake of a given magnitude from any possible direction [16].

Seismic forces are delivered to a building through ground accelerations that may approach from any direction relative to the orthogonal directions of the building; therefore, seismic effects are expected to develop in both directions simultaneously [14]. The directions of

application of seismic forces used in the design shall be those which will produce the most critical load effects [13]. To satisfy this requirement codes accounted for is as follows.

1. By the square root of the sum of the squared (SRSS) values of the action effect due to each horizontal component.
2. By computed using 30 % orthogonal effect, which is the effects from 100 % of the seismic load applied in one direction with 30 % of the seismic load applied in the perpendicular direction.

IBC specify the 30 % orthogonal combination procedure except for Seismic Design Category B. For structures assigned to Seismic Design Category B, the design seismic forces are permitted to be applied independently in each of two orthogonal directions and orthogonal interaction effects are permitted to be neglected. Similar to IBC, UBC and EC8 provide the same provision. However, in UBC and EC8 there is an alternative option to account for critical load effect estimated by SRSS values of the action effect due to each horizontal component. Equivalent to IBC, UBC 97 the design seismic forces are permitted to be applied independently in each of two orthogonal directions and orthogonal interaction effects are permitted to be neglected for structures located in seismic zone 1. EBCS-8 does not provide any specific option to account the critical load effect, instead it requires the design seismic action shall be applied along all relevant horizontal directions, and their orthogonal horizontal action.

In section 4.3 The SRSS and 30% rules to the horizontal ground motion components have been evaluated and compared with the critical response. This is defined as the largest response for all possible incident angles of the seismic components.

CHAPTER FOUR

CASE STUDY

4.1 Introduction

In this chapter to show the difference between the codes in a better way, moment resistance reinforced concrete buildings of different height, employing different force reduction factor, located in area of varying seismicity and on different soil types, are selected and its base shear force, in linear static and dynamic form are obtained by four codes and compared each other. In addition, static and dynamic analyses are compared. For this comparison, different reinforced concrete moment frame buildings, located on soil type B (EBCS-8) in seismic zone 4 (EBCS-8) are selected and the member forces using the two analysis technique are compared each other. Finally, the response of the two horizontal seismic components estimated by multi-component combination rules are compared with the critical response which is defined as the largest response for all possible incident angles of the seismic components.

4.2 Base Shear Comparison

4.2.1 Description of Case Study Buildings for base shear

Five buildings which have the same floor plan and different elevation adopted for this study. The building is laid out on a rectangular grid with a typical five 6m bays in the X-direction, and three 6m bays in the Y-direction as shown in figure 4-1. For purpose of simplicity the columns have constant cross section throughout the building, and the beams and slabs have the same dimension on all stories within the practical range. The structures are hypothetical and have been chosen for the research purpose. The member dimension selected for this analysis were, beam 30cm wide X 40cm deep. Columns 30cm, 45cm, 60cm, 75cm and 90cm for 3, 6, 9, 12 and 15 stories respectively. The slab is 16cm thick. The comparison procedure for a six story concrete building, which is located in a seismic area with 0.1g base design acceleration on the soil type B (EBCS-8), type S_D (UBC 97 and IBC), or type C (EC8) is reported herein.

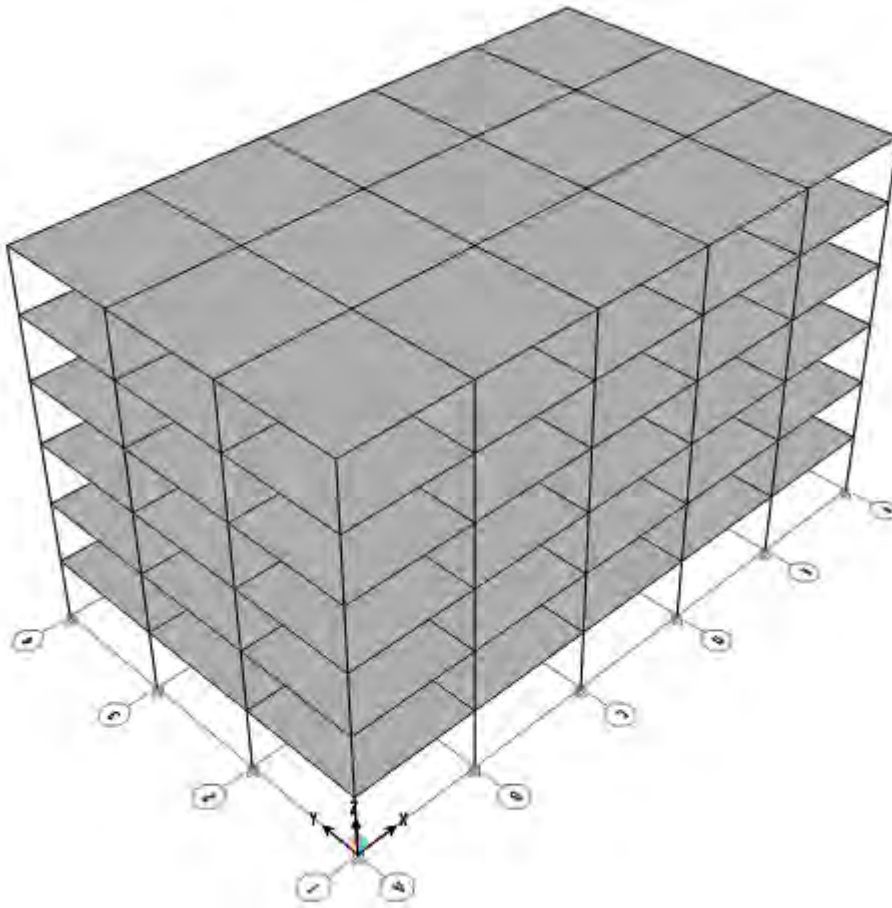


Figure 4-1 typical plane and elevation of moment frame concrete building

4.2.2 Equivalent Lateral Force Analysis (ELF)

4.2.2.1 Comparison of time period

In the IBC provision the period to be used in the ELF analysis is in the range of T_a to $C_u T_a$. If an accurate analysis such as from Rayleigh analysis or eigenvalue extraction in ETABS provides periods greater than $C_u T_a$, $C_u T_a$ should be used. If the accurate analysis produces periods less than $C_u T_a$ but greater than T_a , the period from the analysis should be used. Finally, for exceptionally very stiff buildings, their period shall not be less than T_a . In similar fashion in the UBC the period to be used in the ELF analysis is in the range of T_A to 1.3 or 1.4 times T_A . If the Rayleigh analysis provides a periods greater than $1.4T_A$, $1.4T_A$ should be used. Similar to IBC, the minimum period of the building should be T_A . In EC8 and EBCS-8 do not provide rational period for ELF analysis.

4.2.2.2 Comparison of Structural System Factor (behavioral factor)

Table 4-26 Table Comparison of Response Modification Factors in UBC, IBC, EC8 and EBCS-8

Lateral Force Resisting System	UBC (R)	IBC (R)	EC8 (q)*	EBCS-8 (1/γ)**
Special reinforced concrete moment frames (high ductility concrete)(DCH)	8.5	8	5.85	5
Intermediate reinforced concrete moment frames (medium ductility concrete)(DCM)	5.5	5	3.9	3.3
ordinary reinforced concrete moment frames (low ductility concrete)(DCL)	3.5	3	3.9-1.5	2.5

*For buildings which are not regular in elevation, the values should be reduced by 20%

**For non-regular structures the values should be reduced by 25%

According to IBC Intermediate reinforced concrete moment frames are not permitted for SDC *C* and above and ordinary reinforced concrete moment frame are not allowed for SDC *B* and above. Similarly we cannot use Intermediate reinforced concrete moment frame for seismic zone 3 and 4 in the UBC 97 provision and ordinary reinforced concrete moment frames are not permitted for seismic zone 2 also. In EC8 Concrete buildings may alternatively be designed for low dissipation capacity and low ductility in low seismicity cases in which the design ground acceleration on type A ground, a_g , is not greater than 0.08g. EBCS-8 has no any limitation on the use of ductility class related to seismicity.

Table 4-27 Base Shear Force According to UBC, IBC and EC8 and EBCS-8

	UBC 97		IBC		EC8		EBCS8	
	Notation	Scalar Value	Notation	Scalar Value	Notation	Scalar Value	Notation	Scalar Value
Period	T_A	0.63	T_a	0.62	T_1	0.65	T_1	0.65
	$T_{max.}$	0.89	$C_u T_a$	1.06				
	$T_{Reyleigh}$	1.22	Eigenvalue	1.17				
Seismic Zone Factor	Z	0.1	S_S	0.25	α_g	0.1	α_o	0.1
			S_1	0.10				
Site Coefficient	C_a/Z	1.6	F_a	1.6	S	1.15	S	1.2
	C_v/Z	2.4	F_v	2.4				
Spectral Acceleration (S_a)	$2.5C_a$	0.4	$S_{MS}=F_a S_s$	0.40	$2.5S\eta$	3	2.5α	0.25
	C_v	0.24	$S_{M1}=F_v S_1$	0.24			α	0.1908
			$S_{DS}=2/3 S_{MS}$	0.26				
			$S_{D1}=2/3 S_{M1}$	0.16				
Important Factor	I	1	I	1	γ_I	1	I	1
Reduction Factor	R	5.5	R	5	γ	3.9	γ	0.3
Base shear Coefficient	$C_v I/RT$	0.048	S_{D1}/RT	0.03	$2.5\alpha S\eta(T)$	0.057	$\alpha_o I\beta\gamma$	0.057
	Max.	$2.5C_a I/R$	$S_{DS}/(R/I)$	0.04	2.5α	0.067	$2.5\alpha_o I\gamma$	0.075
	Min.	$0.11C_a I$	$0.044S_{DS}I$	0.011	$\beta\alpha$	0.020	-	-
Base shear	V	924	V	615	F_b	1086	F_b	1085

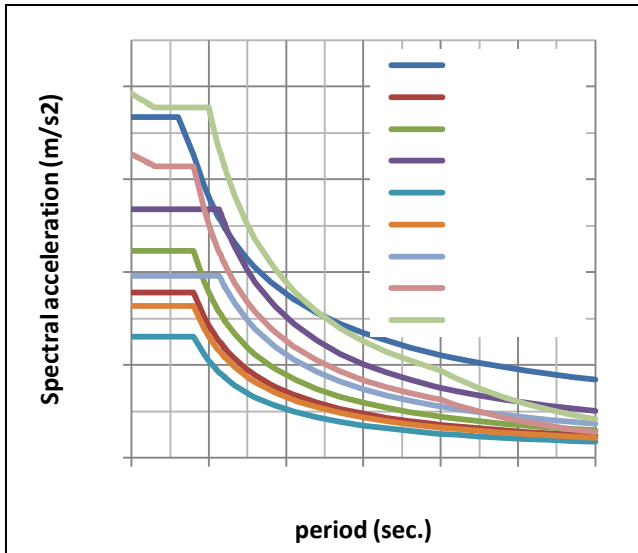


FIGURE 4-2 Computed ELF Total Acceleration Response Spectra for Soil Type A (EBCS-8), S_C , S_B , S_A (UBC And IBC), and A, B (EC8).

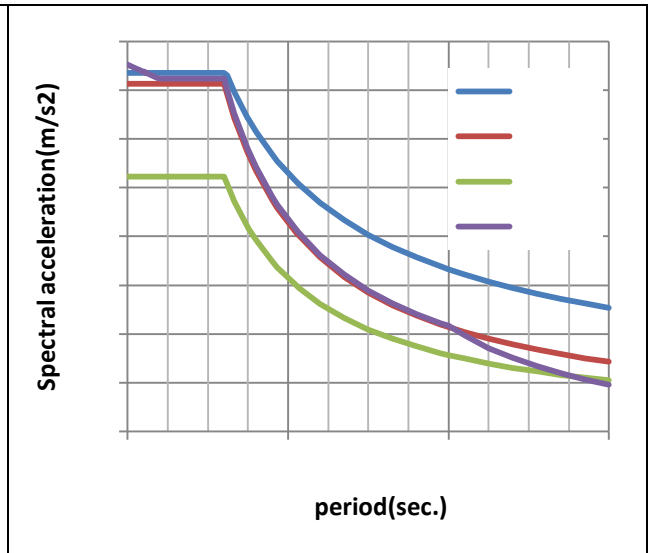


FIGURE 4-3 Computed ELF Total Acceleration Response Spectra for Soil Type B (EBCS-8), S_D (UBC and IBC), and C (EC8).

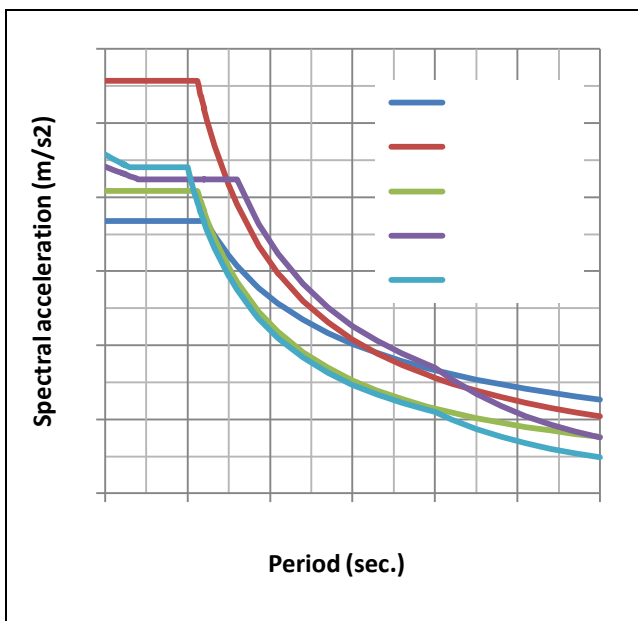


Figure 4-4 Computed ELF Total Acceleration Response Spectra for Soil Type C (EBCS-8), S_D (UBC And IBC), and D, E (EC8).

4.2.2.3 Vertical Distribution of Base Shear

After calculating the base shear force, seismic lateral force exerted on each floor is obtained and are presented in Table 4-3. (k is calculated from linear interpolation as 1.275 and $F_T = 0.07TV$ for the case of EBCS-8 and UBC 97 if period $> 0.7s$).

Table 4-28 Vertical Distribution of Base Shear According to UBC, IBC, EC8 and EBCS-8

Story	h_i	W_i	$W_i h_i$	$W_i h_i^k$	UBC	IBC	EC8	EBCS-8
6	18	3237	58266	129007	340	185	316	318
5	15	3237	48555	102249	229	146	264	265
4	12	3237	38844	76930	183	110	211	212
3	9	3237	29133	53309	137	76	158	159
2	6	3237	19422	31789	92	46	105	106
1	3	3237	9711	13136	46	19	53	53
sum		19422	203931	406422	1029	583	1107	1112

4.2.2.4 Determining the Story Drift

The values of column 3 of table 4- 4 are the story drift as reported by ETABS. These drift values, however, are much less than these that will actually occur because structures will respond inelastically to the earthquake. The true inelastic drift, which amplifies the ETABS drift, is shown in column 4, 6, 8, 10 of table 4-4 as per each codes requirement. The allowable story drift as per the codes provision is shown in column 5, 7, 9 and 11 of table 4- 4. It is clear from table 4- 4 that the allowable story drift of all codes are not exceeded at any level

Table 4-29 ELF Drift for Building Responding in Y-Direction in cm

Story	Total Elastic Story Drift	Elastic Story Drift(Δ_s)	UBC 97		IBC		EC8		EBCS8	
			Inelastic Story drift ($0.7R\Delta_s$)	Allowable Drift ($<0.02H$)	Inelastic Story drift ($C_d\Delta_s/I$)	Allowable Drift ($<0.02H$)	Inelastic Story drift(Δ_{sqd})	Allowable Drift ($0.01h-0.02h$)	Inelastic Story drift(Δ_s/γ)	Allowable Drift ($0.01h-0.015h$)
6			1.4	6	1.7	6	1.4	3-6	1.2	3-4.5
5			2.2	6	2.5	6	2.2	3-6	1.8	3-4.5
4			2.8	6	3.3	6	2.8	3-6	2.4	3-4.5
3			3.2	6	3.7	6	3.2	3-6	2.8	3-4.5
2			3.0	6	3.51	6	3.0	3-6	2.6	3-4.5
1			1.6	6	1.8	6	1.6	3-6	1.4	3-4.5

4.2.2.5 P-Delta Effects

P-delta effects are computed for the X-direction response in Table 4-5. From the table the stability limit according to EC8 and EBCS-8 are greater than 0.1, so P-delta should be included. Nowadays due to computer P-delta effect can be directly included without perform a special analysis.

Table 4-30 Computation of P-Delta Effects for X-Direction Response

Story	h(cm)	Δ_s	P_x	V_x	UBC	IBC	EC8		EBCS-8	
					(θ_x)	(θ_x)	d_r	(θ_x)	d_r	(θ_x)
6	300				0.013	0.013	1.4	0.048	1.2	0.041
5	300				0.021	0.021	2.2	0.082	1.8	0.067
4	300				0.030	0.030	2.8	0.114	2.4	0.098
3	300				0.038	0.038	3.2	0.145	2.8	0.127
2	300				0.040	0.040	3.0	0.153	2.6	0.132
1	300				0.024	0.024	1.6	0.093	1.4	0.082

The above procedures are for six story building located on soil type B (EBCS-8) and its equivalent for the other codes. Using similar procedures the base shear for other building on varies soil type using the four codes are computed and compared to each other. It is present as follow.

Table 4-31 EL Base Shear for Six Story Building in UBC, IBC, EC8 and EBCS-8

UBC 97		IBC		EC8		EBCS8		V_{EBCS8}	V_{EBCS8}	V_{EBCS8}
Soil type	Base shear	Soil type	Base shear	Soil type	Base shear	Soil type	Base shear	/ V_{UBC}	/ V_{IBC}	/ V_{EC8}
S_A	308	A	190	A	573	A	904	2.94	4.76	1.58
S_B	385	B	238					2.35	3.80	
S_C	655	C	412	B	945			1.38	2.19	0.96
S_D								1.17	1.76	1.00
S_E	1348	E	968	D	1393	C	1356	1.01	1.40	0.97
				E	1102					1.23

Table 4-32 ELF Base Shear for Three Story Building in UBC, IBC, EC8 and EBCS-8

UBC 97		IBC		EC8		EBCS8		V_{EBCS8}	V_{EBCS8}	V_{EBCS8}
Soil type	Base shear	Soil type	Base shear	Soil type	Base shear	Soil type	Base shear	/ V_{UBC}	/ V_{IBC}	/ V_{EC8}
S_A	245	A	166	A	486	A	602	2.45	3.63	1.24
S_B	308	B	208					1.95	2.89	
S_C	520	C	360	B	583			1.16	1.67	1.03
S_D								0.91	1.25	1.20
S_E	1071	E	848	D	656	C	669	0.62	0.79	1.02
				E	680					0.98

Table 4-33 ELF Base Shear for Nine Story Building in UBC, IBC, EC8 and EBCS-8

UBC 97		IBC		EC8		EBCS8		V_{EBCS8}	V_{EBCS8}	V_{EBCS8}
Soil type	Base shear	Soil type	Base shear	Soil type	Base shear	Soil type	Base shear	$/V_{UBC}$	$/V_{IBC}$	$/V_{EC8}$
S _A	370	A	214	A	754	A	1200	3.90	5.61	1.59
S _B	463	B	268					2.59	4.48	
S _C	787	C	464	B	1132			1.52	2.59	1.06
S _D								1.30	2.08	1.11
S _E	1618	E	1097	D	2039	C	1800	1.11	1.64	0.88
				E	1323					1.36

Table 4-34 ELF Base Shear for Twelve Story Building in UBC, IBC, EC8 and EBCS-8

UBC 97		IBC		EC8		EBCS8		V_{EBCS8}	V_{EBCS8}	V_{EBCS8}
Soil type	Base shear	Soil type	Base shear	Soil type	Base shear	Soil type	Base shear	$/V_{UBC}$	$/V_{IBC}$	$/V_{EC8}$
S _A	438	A	254	A	899	A	1526	4.95	6.01	1.70
S _B	548	B	318					2.78	4.80	
S _C	930	C	550	B	1349			1.64	2.77	1.13
S _D								1.39	2.23	1.18
S _E	1916	E	1296	D	2427	C	2290	1.20	1.77	0.94
				E	1573					1.46

Table 4-35 ELF Base Shear for Fifteen Story Building in UBC, IBC, EC8 and EBCS8

UBC 97		IBC		EC8		EBCS8		V_{EBCS8}	V_{EBCS8}	V_{EBCS8}
Soil type	Base shear	Soil type	Base shear	Soil type	Base shear	Soil type	Base shear	$/V_{UBC}$	$/V_{IBC}$	$/V_{EC8}$
S _A	518	A	280	A	1058	A	1908	6.19	6.81	1.80
S _B	647	B	350					2.95	5.45	
S _C	1100	C	606	B	1587			1.73	3.15	1.20
S _D								1.47	2.53	1.25
S _E	2265	E	1426	D	2857	C	2862	1.26	2.01	1.00
				E	1851					1.55

For ordinary reinforced concrete moment resisting frame (UBC 97, IBC) or DCL (EC8, EBCS8) the above ratio, $V_{EBCS-8}/V_{UBC 97}$ and V_{EBCS-8}/V_{IBC} , will be scale up by 0.85 and 0.80 respectively due to change in response modification or behavioral factor. Furthermore, for special moment resisting frame or DCH the ratio, $V_{EBCS-8}/V_{UBC 97}$, V_{EBCS-8}/V_{IBC} and V_{EBCS8}/V_{EC8} , the ratios from table 4-6-table 4-10 scaled up by 1.04, 1.06 and 0.94 respectively.

4.2.3 Modal Response Spectrum Analysis

The first step in the modal-response-spectrum analysis is computation of the structural mode shapes and associated periods of vibration. The magnitudes of the forces and moments have no meaning since the amplitude of a mode shape can be normalized to any value. The mode shapes and frequencies are automatically computed by ETABS. The computed periods of vibration for the first 6 modes are summarized in Table 4-11, which also shows values called the modal direction factor effective mass factor for each mode. The modal direction factors shown in Table 4-11 are indices that quantify the direction of the mode. The effective mass in Modes 1 through 6 is given as a percentage of total mass. The values shown in parentheses in Table 4-11 are the accumulated effective masses.

Table 4-36 Computed Period, Directional Factor and Effect Mass

Mode	Period	Modal Direction Factor			Effective Mass Factor		
		X Translation	Y Translation	Z Torsion	X Translation	Y Translation	Z Torsion
1	1.19	99.4	0.0	0.0	0.0	81.9	0
2	1.15	0.0	99.4	0.0	82.2	0(81.9)	0
3	1.06	0.0	0.0	82.0	0(82.2)	0(81.9)	0
4	0.37	0.3	0.0	0.0	0(82.2)	10.2(92.1)	0
5	0.37	0.0	0.3	0.0	10.1(92.3)	0(92.1)	0
6	0.34	0.0	0.0	10.2	0(92.3)	0(92.1)	0

4.2.3.1 Design Spectrum Coordinates for Varies Site Classes

The ordinates of the normalize response spectra of UBC 97, EC8 and EBCS-8 are given in figure 4-5 to 4-7. The normalize response spectra of IBC 2009 is 2/3 times the response spectra of UBC 97 except for long periods at which the design response spectra of IBC 2009 has reduced ordinate as indicated in table 3-1.

Using the response spectrum coordinate of figure 4-5 to 4-7 the dynamic response spectrum analysis was carried out. The first 6 modes of response were computed and superimposed using complete quadratic combination (CQC) with 5% modal damping ratio. The response spectrum used in the analysis did include structure importance factor, the response modification (behaviour) factor and the reference peak ground acceleration for sample structures and soil conditions.

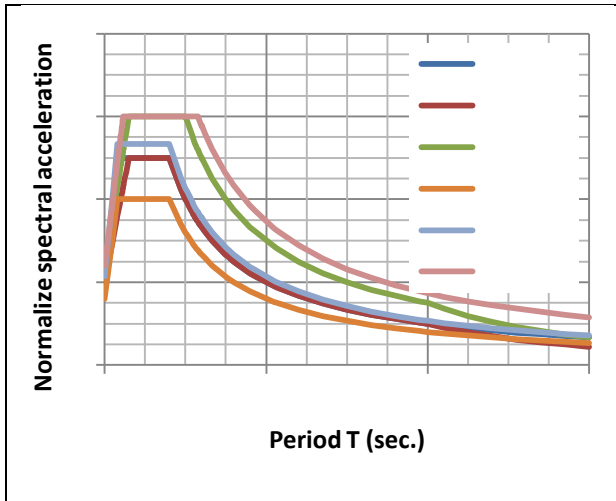


Figure 4-5 Normalize Elastic Design Spectra for Soil Type A (EBCS8), A and B (EC8), and S_A , S_B and S_C (UBC 97).

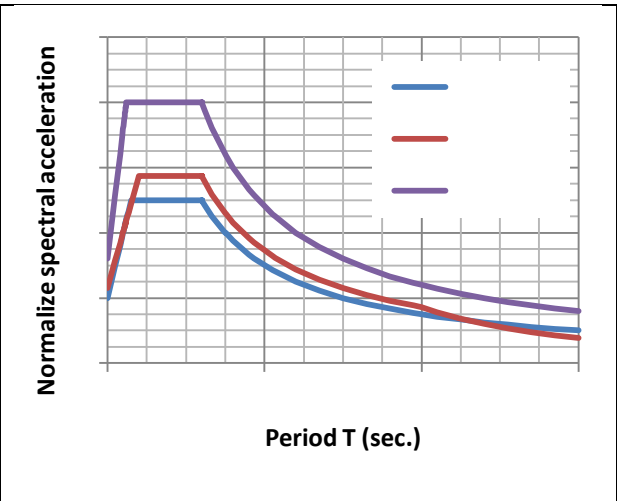


Figure 4-6 Normalize Elastic Design Spectra For Soil Type B (EBCS8), C (EC8), and S_D (UBC 97).

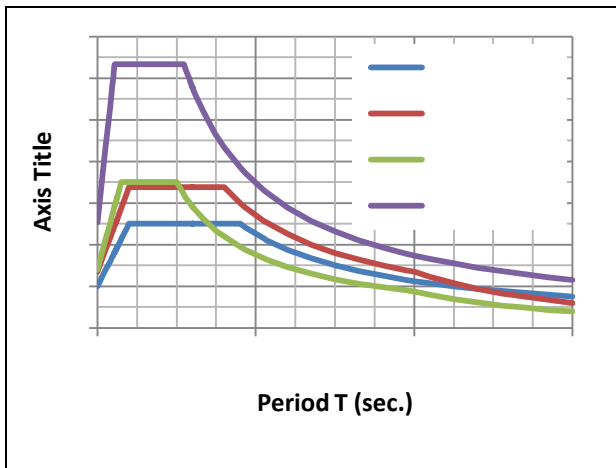


FIGURE 4-7 Normalize Elastic Design Spectra for Soil Type C (EBCS-8), D And E (EC8) and S_E (UBC 97).

4.2.3.2 Dynamic Base Shear

Table 4-37 RSA Base Shear for Six Story Building in UBC, IBC, EC8 and EBCS-8

UBC 97		IBC		EC8		EBCS8		V_{EBCS8} / V_{UBC}	V_{EBCS8} / V_{IBC}	V_{EBCS8} / V_{EC8}
Soil type	Base shear	Soil type	Base shear	Soil type	Base shear	Soil type	Base shear			
S_A		A		A	369	A	440	2.11	2.88	1.19
S_B		B						1.69	2.30	
S_C		C		B				1.02	1.40	0.81
S_D								1.06	1.44	1.04
S_E	885	E	649	D		C	925	1.05	1.43	0.98
				E						1.46

Table 4-38 RSA Base Shear for Three Story Building in UBC, IBC, EC8 and EBCS-8

UBC 97		IBC		EC8		EBCS8		V_{EBCS8}/V_{UBC}	V_{EBCS8}/V_{IBC}	V_{EBCS8}/V_{EC8}
Soil type	Base shear	Soil type	Base shear	Soil type	Base shear	Soil type	Base shear			
S _A		A		A	262	A	299	2.06	2.82	1.14
S _B		B						1.65	2.25	
S _C		C		B				0.97	1.32	0.78
S _D								1.03	1.41	1.00
S _E	635	E	666	D		C	587	0.92	0.88	0.87
				E						1.31

Table 4-39 RSA Base Shear for Nine Story Building in UBC, IBC, EC8 and EBCS-8

UBC 97		IBC		EC8		EBCS8		V_{EBCS8}/V_{UBC}	V_{EBCS8}/V_{IBC}	V_{EBCS8}/V_{EC8}
Soil type	Base shear	Soil type	Base shear	Soil type	Base shear	Soil type	Base shear			
S _A		A		A	522	A	520	2.08	2.83	1.00
S _B		B						1.66	2.27	
S _C		C		B				1.02	1.39	0.80
S _D								1.04	1.42	1.02
S _E	1049	E	770	D		C	1066	1.02	1.38	0.97
				E						1.41

Table 4-40 RSA Base Shear for Twelve Story Building in UBC, IBC, EC8 and EBCS-8

UBC 97		IBC		EC8		EBCS8		V_{EBCS8}/V_{UBC}	V_{EBCS8}/V_{IBC}	V_{EBCS8}/V_{EC8}
Soil type	Base shear	Soil type	Base shear	Soil type	Base shear	Soil type	Base shear			
S _A		A		A	742	A	637	2.07	2.83	0.86
S _B		B						1.66	2.27	
S _C		C		B				1.01	1.37	0.78
S _D								1.04	1.42	1.02
S _E	1306	E	958	D		C	1302	1.00	1.36	0.97
				E						1.40

Table 4-41 RSA Base Shear for Fifteen Story Building in UBC, IBC, EC8 and EBCS-8

UBC 97		IBC		EC8		EBCS8		V_{EBCS8}/V_{UBC}	V_{EBCS8}/V_{IBC}	V_{EBCS8}/V_{EC8}
Soil type	Base shear	Soil type	Base shear	Soil type	Base shear	Soil type	Base shear			
S _A		A		A	999	A	718	2.33	2.81	0.72
S _B		B						1.65	2.25	
S _C		C		B				1.01	1.38	0.68
S _D								1.03	1.41	0.91
S _E	1466	E	1047	D		C	1271	0.87	1.21	0.81
				E						1.10

Since for dynamic analysis all codes use the same period of vibration the base shear ratio is different from the static one. However the magnitudes of the forces and moments have no meaning because the dynamic values can be normalized to static values.

4.3 Comparison between Static and Dynamic Analysis

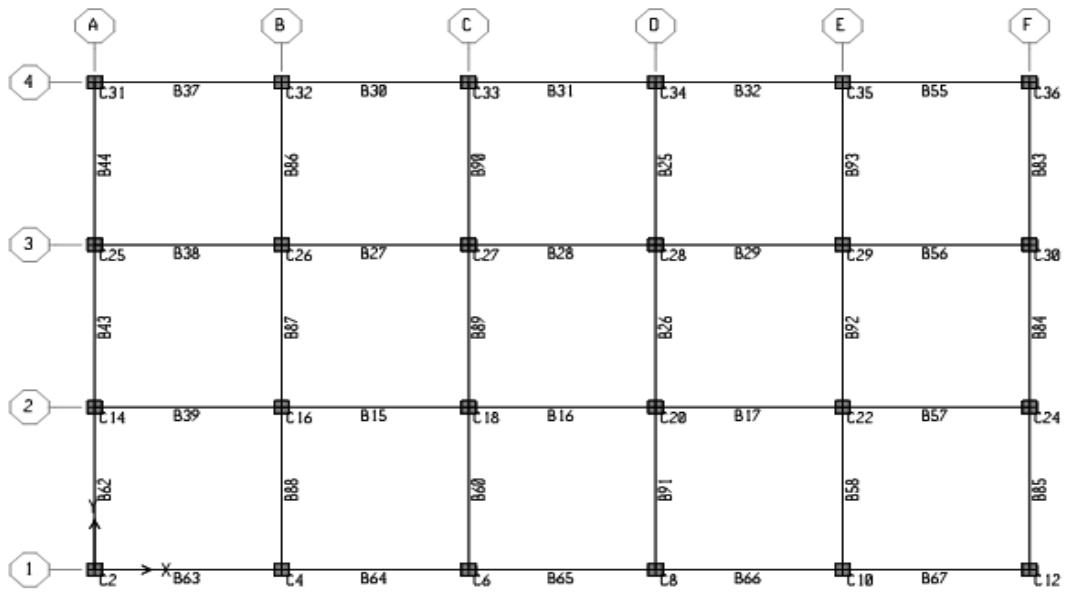
In this section the static and dynamic analyses are compared. For this comparison the response spectra and the Equivalent static load for soil type B in seismic zone 4, which is defined in the EBCS8, is used as the basic loading for the three dimensional dynamic and equivalent static load analyses. To clearly show the difference between them, accidental torsion, P-Delta and orthogonal effects are not included in the analysis. The analyses results are compared with each other, and the accuracy and relative merits of the analysis approach for different structure are discussed.

4.3.1 Description of Case Study Buildings

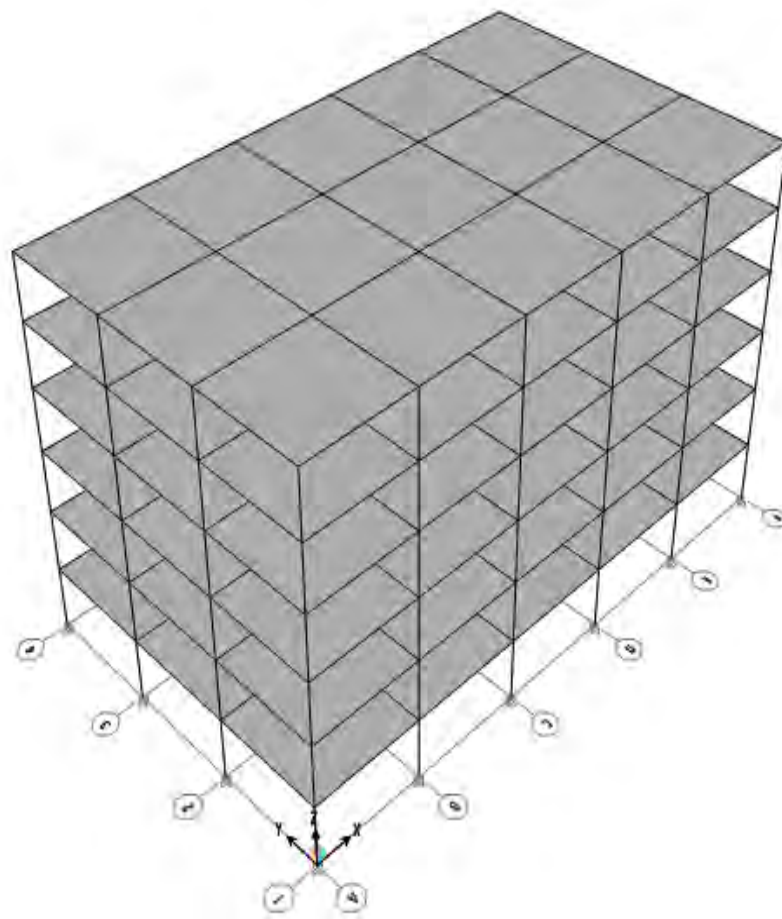
Four groups of buildings with different regularity selected for this study. The first group is highly regular buildings with different heights. The second and the third group are vertically and horizontally irregular buildings respectively with different elevation. The last group is highly irregular buildings with both plan and elevation irregularity. All buildings laid out on a rectangular grid with a typical 6m width bays in the X and Y directions. For purpose of simplicity the columns have constant cross section throughout the building and the beams and slabs have the same dimension on all stories with in the practical range. The structures are hypothetical and have been chosen for the illustrative purpose. The member dimension selected for this analysis were, beam 30cm wide X 40cm deep. Square Columns: 30cm, 45cm, 60cm, 75cm and 90cm for 3, 6, 9, 12, 15 stories respectively. And the slab is 16cm thick.

4.3.1.1 Case Study One: Highly Regular Buildings

Regular buildings which have the same layout and different elevation are selected for this analysis. a six story building is shown in figure 4-8 the other buildings is not shown here, but they have the same plan as figure 4-8 with different elevations. For these buildings column, **C16**, and beam, **B62**, from figure 4-8a are selected for comparison and the moment and shear from each analysis result are compared.



a) Plane view



b) Three dimensional view

Figure 4-8 Case Study One: Highly Regular Building.

Table 4-42 Comparison of Column Forces for Three Story Regular Building

story	ELF analysis		RSA		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}
	17.7	25.0	15.2	21.5	1.16	1.16
	29.2	43.3	25.7	38.0	1.14	1.14
	31.4	50.3	28.2	45.1	1.11	1.11

Table 4-43 Comparison of Beam Forces for Three Story Regular Building

story	ELF analysis		RSA		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

Table 4-44 Comparison of Column Forces for Nine Story Regular Building

Story	ELF Analysis		RSA		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

Table 4-45 Comparison of Beam Forces for Nine Story Regular Building

Story	ELF Analysis		RSA		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

4.3.1.2 Case Study Two: Vertically Irregular Buildings

Vertically regular buildings of story 3, 6, 9, 12 and 15 buildings have been chosen for this comparison. The numbers stories in the typical stories are equal. For instance for nine story building shown figure 4-9 each typical stories have three stories. When typical stories become 2, 4 and 5, the total stories of the building will be 6, 12 and 15 respectively. For these buildings column (2B) and Beam (2E)-(3E) are selected for the comparison purpose and the results from each analysis result are compared.

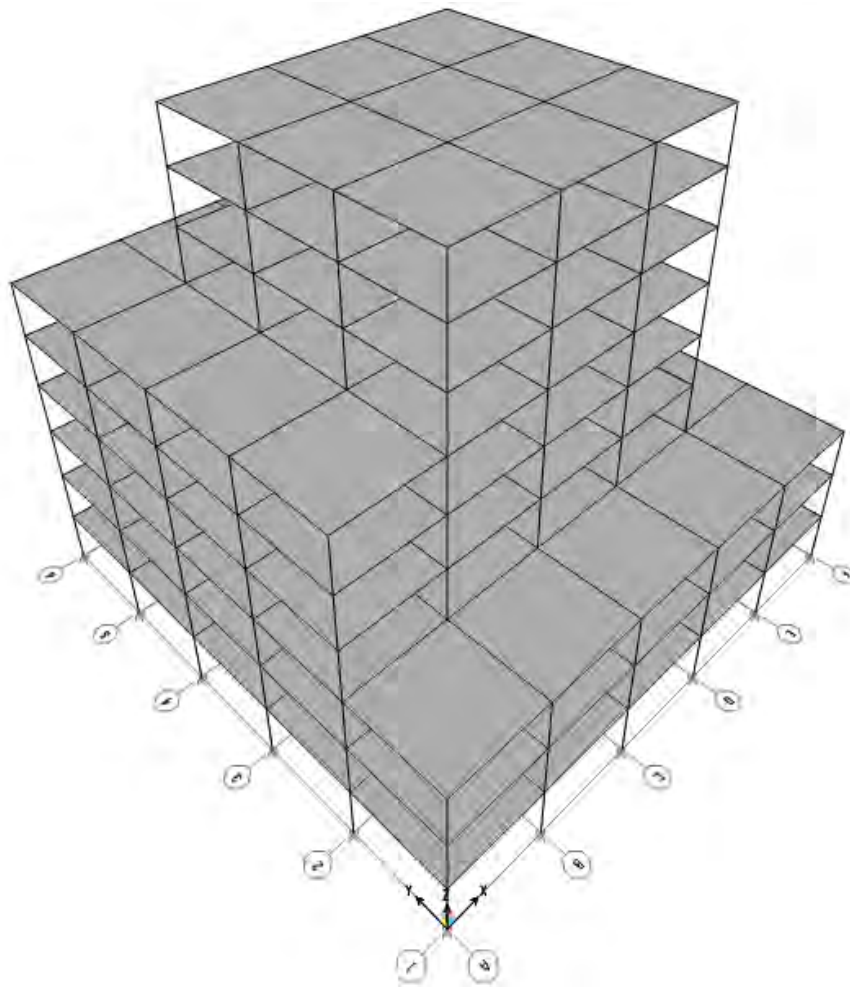


Figure 4-9 Case Study Two: Vertically Irregular Building

Table 4-48 Comparison of Column Forces for Three Story vertically Irregular Building

story	ELF analysis		RSA analysis		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

Table 4-49 Comparison of Beam Forces for Three Story Vertically Irregular Building

story	ELF analysis		RSA analysis		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

Table 4-50 Comparison of Column Forces for Six Story vertically Irregular Building

Story	ELF analysis		RSA analysis		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}
6	23.1	33.9	22.8	33.4	1.01	1.01
5	34.3	46.3	34.2	46.9	1.00	0.99
4	39.2	55.9	37.8	54.2	1.04	1.03
3	46.1	70.1	44.1	67.6	1.04	1.04
2	31.9	53.0	30.0	49.9	1.06	1.06
1	37.2	75.1	34.6	69.8	1.08	1.07

Table 4-51 Comparison of Beam Forces for Six Story Vertically Irregular Building

Story	ELF analysis		RSA analysis		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}
6	3.8	10.9	3.6	10.4	1.05	1.05
5	6.7	19.1	6.5	18.4	1.04	1.04
4	9.1	25.8	8.7	24.8	1.04	1.04
3	10.8	30.9	10.3	29.5	1.05	1.05
2	7.8	21.5	7.3	20.3	1.06	1.06
1	6.0	16.7	5.6	15.4	1.09	1.08

Table 4-52 Comparison of Column Forces for Nine Story Vertically Irregular Building

story	ELF analysis		RSA analysis		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

4.3.1.3 Case Study Building Three: Horizontally Irregular buildings

Horizontal Irregular buildings which have the same layout with three, six, nine, twelve and fifteen stories have been selected. The six story building is shown in figure 4-10. For these buildings column (6F) and Beam (5f)-(6f) are selected for the comparison and the results from each analysis result are compared.

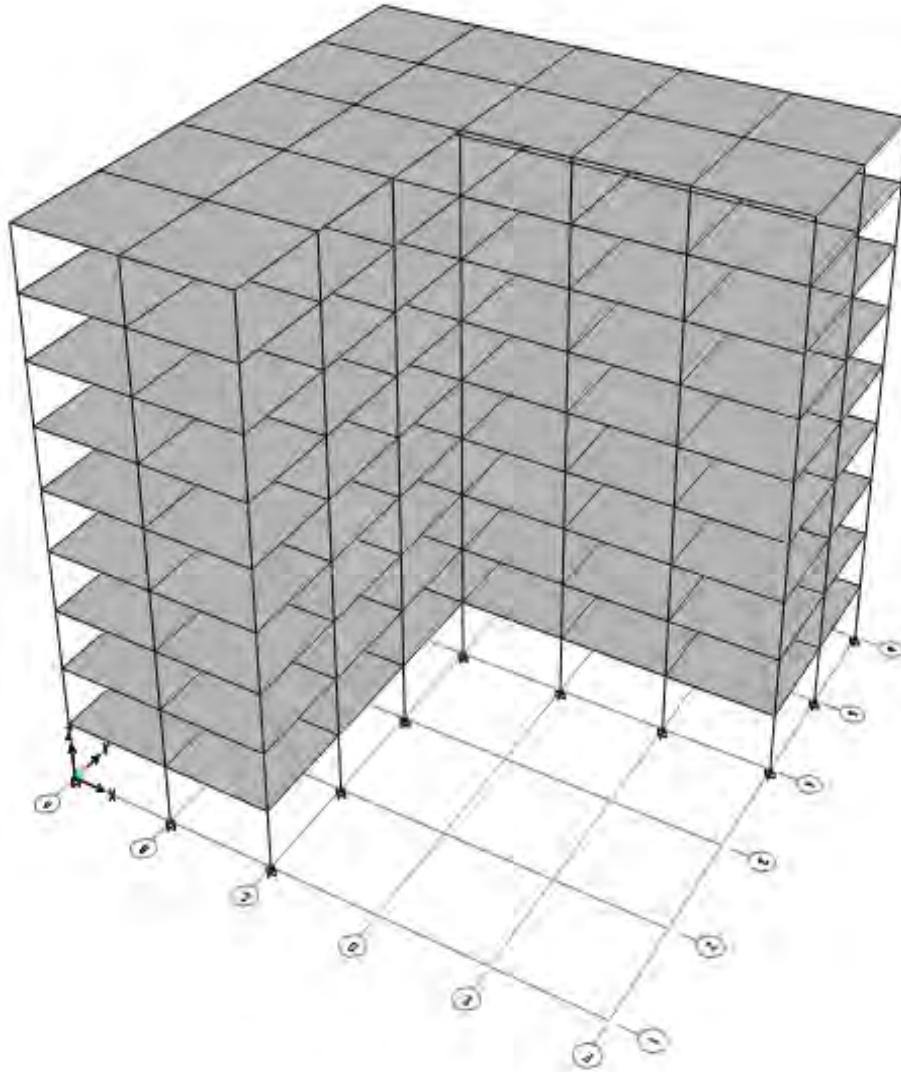


FIGURE 4-10 Case Study Four: Horizontally Irregular Building

Table 4-58 Comparison of Column Forces for Three Story Horizontally Irregular Building

Story	ELF analysis		RSA analysis		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

Table 4-59 Comparison of Beam Forces for Three Story Horizontally Irregular Building

Story	ELF analysis		RSA analysis		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

Table 4-60 Comparison of Column Forces for Six Story Horizontally Irregular Building

Story	ELF analysis		RSA analysis		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

Table 4-61 Comparison of Beam Forces for Six Story Horizontally Irregular Building

Story	ELF analysis		RSA analysis		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

Table 4- 62 Comparison of Column Forces for Nine Story Horizontally Irregular Building

Story	ELF analysis		RSA analysis		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

4.3.1.2 Case Study Four Highly Irregular buildings

We used vertically regular buildings of 3, 6, 9, 12 and 15 story buildings for this comparison. The number stories in typical stories are equal. For instance for 9 story building shown figure 4-11 each similar stories have three stories. When the number of similar stories becomes 2, 4 and 5 the total stories of the building will be 6, 12 and 15 respectively. For these buildings column (5B) and Beam (5A)-(6A) are selected for this comparison and the results from each analysis result are compared.

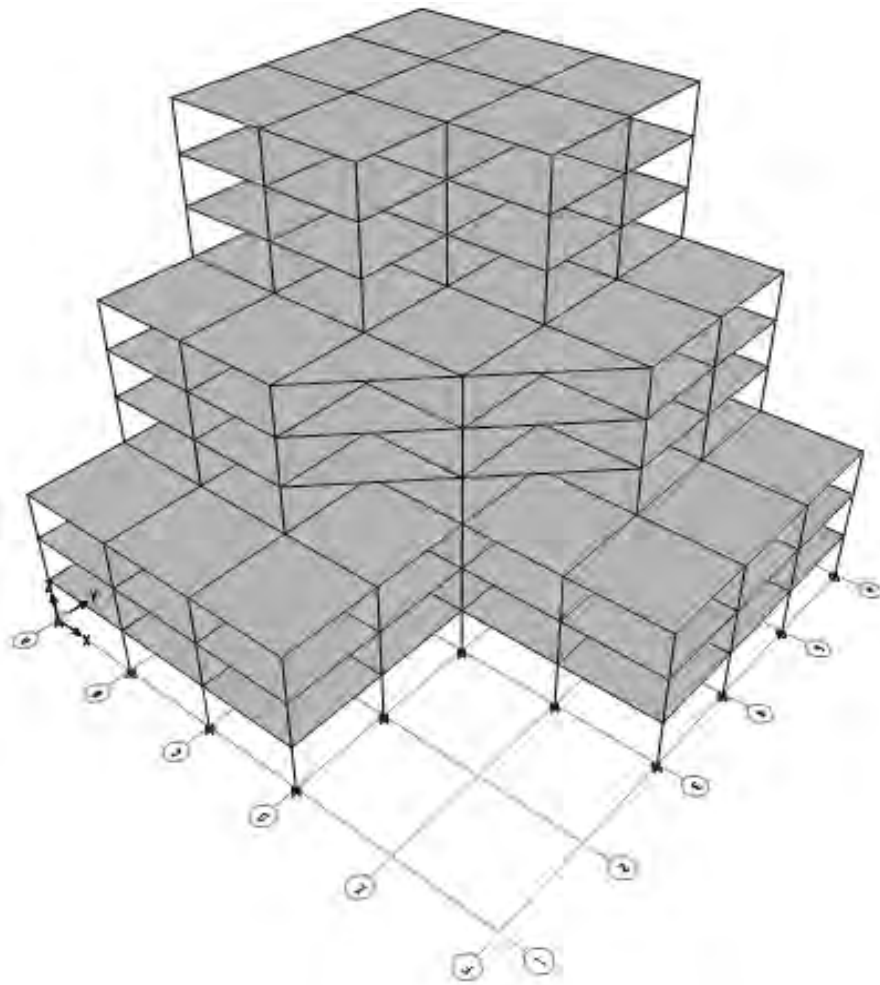


FIGURE 4-11 Case Study Building Four: Highly Irregular Building

Table 4-68 Comparison of Column Forces for Three Story Highly Irregular Building

story	ELF analysis		RSA		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

Table 4-69 Comparison of Beam Forces for Three Story Highly Irregular Building

Story	ELF analysis		RSA analysis		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

Table 4-70 Comparison of Column Forces for six Story Highly Irregular Building

Story	ELF analysis		RSA analysis		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

Table 4-71 Comparison of Beam Forces for Nine Story Highly Irregular Building

Story	ELF analysis		RSA analysis		Ratio	
	V	M	V	M	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

Table 4-72 Comparison of Column Forces for Nine Story Highly Irregular Building

story	ELF analysis		RSA analysis		Ratio	
	V2	M3	V2	M3	V_{ELF}/V_{RSA}	M_{ELF}/M_{RSA}

4.4 Evaluation of Combination Rules of Horizontal Seismic Components

A well-designed structure should be capable of equally resisting earthquake motions from all possible directions. For complex three dimensional structures the direction of the earthquake which produces the maximum stresses, in a particular member or at a specified point, is not apparent. Therefore, a large number of dynamic analyses at various angles of input in order to check all points for the critical earthquake directions should be performed. Such an elaborate study could conceivably produce a different critical input direction for each stress evaluated. However, the cost of such a study would be prohibitive. Because of this, it is reasonable to assume that motions that take place during an earthquake have one principal direction even if for most structures this direction is not known. Therefore, the only rational earthquake design criterion is that the structure must resist an earthquake of a given magnitude from any possible direction. To satisfy this requirement codes accounted using multi-component combination rules such as;

1. The square root of the sum of the squared (SRSS) values of the action effect due to each horizontal component.
2. 30% orthogonal, which is the effects from 100 percent of the seismic load applied in one direction with 30 percent of the seismic load applied in the perpendicular direction.

the response to two horizontal seismic components estimated by multi-component combination rules are compared with the critical response this is defined as the largest response for all possible incident angles of the seismic components. The critical response is obtained by performing a large number of dynamic analyses at various angles of input and selecting the maximum one.

4.4.1 Case study for Evaluation of Combination Rules

For this comparison we have chosen two simple reinforced concrete moment frame buildings to represent the building which have uniquely defined principal axes and which have not (complex three-dimensional structure). Both building are a two-story building and have circular columns.

4.4.1.1 Case study one: Non-symmetrical structure

A simple two-story structure, which have not uniquely defined principal axis, shown in Figure 4-12 is selected for evaluation and comparison of SRSS and 30% rules with the critical responses.

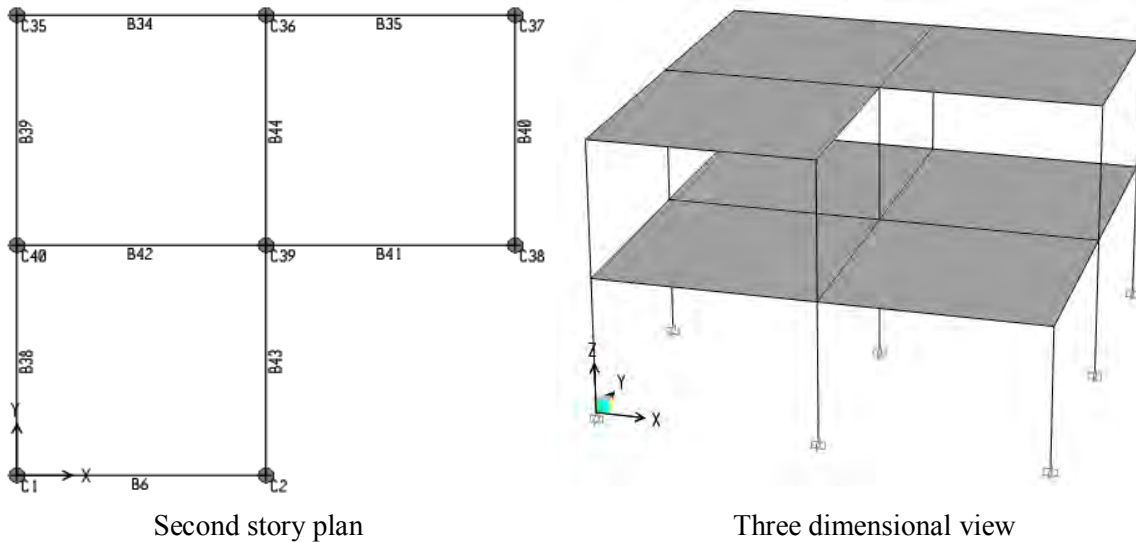


Figure 4-12 Case Study One: Non-Symmetrical Building

The computed periods of vibration, modal direction factor and modal mass Factor for the first 6 modes are summarized in Table 4-53.

Table 4-78 Computed Period, Directional Factor and Effect Mass Factor

Mode	Period	Modal Direction Factor			Modal Mass Factor		
		X Translation	Y Translation	Z Torsion	X Translation	Y Translation	Z Torsion
1	0.411	38.6	38.6	22.8	44.5	44.5	12.3
2	0.403	50.0	50.0	0.0	94.3	94.3	12.3
3	0.349	15.7	15.7	68.5	99.7	99.7	91.7
4	0.160	20.2	20.2	59.6	99.8	99.8	92.6
5	0.159	50.0	50.0	0.0	100.0	100.0	92.6
6	0.143	5.6	5.6	88.8	100.0	100.0	100.0

The resultant moments and shears of column C35 and C37 in the figure 4-13 for the spectrum applied separately at different incident angle in the interval of 15 degrees are summarized in Tables 4-54 and table 4-55. The SRSS and 30% design moments and shears are tabulated also.

4.4.1.2 Case Study Two: Symmetrical Structure

A simple two-story structure, which uniquely defined principal axis, shown in Figure 4-13 is selected for evaluation and comparison of SRSS and 30% rules with the critical responses.

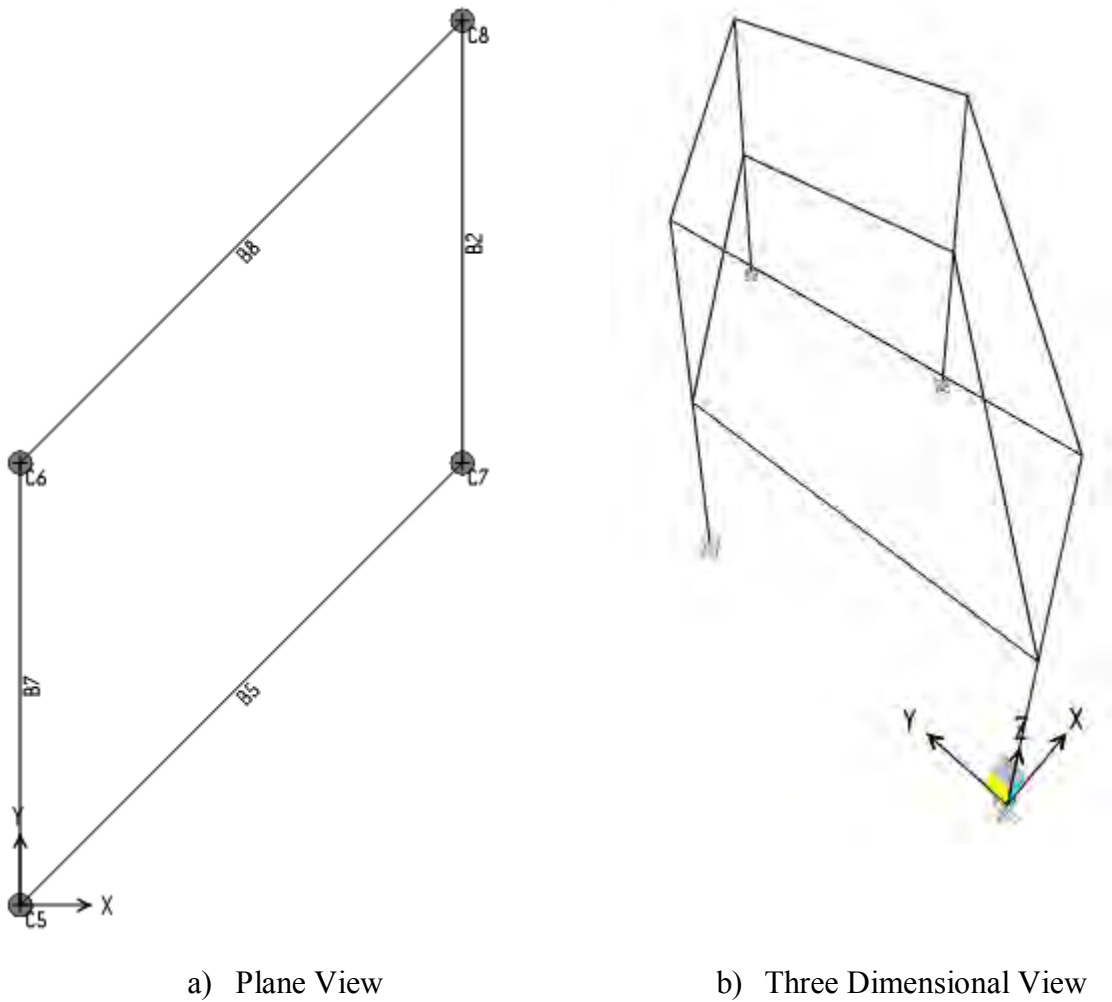


FIGURE 4-13 Case Study Building Two: Symmetrical Structure

Table 4-85 the Critical Load, SRSS and 30% orthogonal loads

column	story	critical direction	critical load		SRSS		30%	
			V	M	V	M	V	M

Table 4-86 comparison of SRSS, 30% and X or Y-directional spectrum loading with the critical responses

Column	Story	Error in % Relative to the Critical Response			
		SRSS		30%	

As shown from the table 4-61 the spectral direction which gives the maximum load is -15 degree and it's orthogonal. This direction is the direction of the base shear associated with the fundamental mode of vibration. For both cases the SRSS response always overestimates the critical response and also 30% rule is less conservatively overestimates the critical response. However, since 30% rule is empirical and the values of the member forces depend on the user's selection of the reference system, sometimes can underestimate the design forces in certain members.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

This study signifies the considerable differences in the factors effective on determining shear force in the EBCS-8 with relative to UBC 97, IBC and EC8. The differences are especially pronounced in response modification factor, soil modification factor and time period of the buildings. These differences finally lead to major differences in the base shear force value among the four codes.

In the equivalent lateral force analysis, the basic differences among the four codes are on behavioral factor, soil amplification factor and building time period. Behavioral factor and building time period leads to high base shear in the EBCS-8. In contrast, soil amplification factor results low base shear in the EBCS-8. For medium and long period buildings the base Shear force values assume greater quantity in EBCS-8 as compared to IBC and UBC, for all soil profiles and the difference becomes increasing when the height of the building increase and the soil becomes hard. The least difference in shear force value is seen for the soil type S_E and the greatest difference in soil type S_A . For short period building site amplification factor becomes dominant over the effect of behavioural factor and time period of the building w/c leads higher base shear in UBC and IBC compared to EBCS-8. However, the shear force values of EBCS-8 are somewhat similar to EC8 in most soil types. Furthermore, due to the factor $1/T^{2/3}$ the base shear of EBCS-8 becomes higher compare to the other codes.

In most cases the normalize spectral acceleration of EBCS-8 significantly lower than the rest codes. However, due to the structural system factor values the total dynamic base shear of EBCS-8 becomes similar with the other codes even higher for some soil type.

The Soil classified in EBCS-8 is in to three classes. It is too few to appropriately cover the different nature of soil profile that could be encountered in reality. In UBC and IBC, however, the soil is classified in to six classes and seven classes by EC8. In addition, in EBCS-8 the soil classification based on shear wave velocity only. The rest three codes account the undrained shear strength and SPT blow count for soil classification in addition to the shear wave velocity.

The mean spectral shapes of different soil conditions of the current Ethiopian code and the former U.S. and European codes determined directly from the study by Seed and coworkers [17]. Further studies show that the effects of level of shaking, rock stiffness, and soil type, stiffness and depth on the amplification of ground motions at short and long periods. The result shows that due to the nonlinear stress-strain behavior in the soil low peak accelerations can be amplified several times at soil sites, especially those containing soft layers and where the rock is not very deep. On the other hand, larger peak accelerations can be amplified to a lesser degree and can even be slightly deamplified at very high rock accelerations. The UBC 97 and IBC spectra have different soil amplification factor for each seismicity with higher seismic coefficients in areas of lower shaking to low seismic coefficients in areas of high shaking. A similar feature seems to be captured by the Type 1 (for high seismicity areas) and Type 2 (low seismicity areas) spectra from EC8, 2004. These approaches can influence the soil classification and amplification to be followed by EBCS-8 in the future.

Empirical period determinations of all codes are similar, with some exceptions. IBC gives an alternative formula for the determination of approximate fundamental period, T_a equal to 0.1 times number of story, of concrete and steel moment resisting frame buildings not exceeding 12 stories in height and having a minimum story height of 10 ft. This formula is not included in the other codes. An alternative estimation of the period $T_I = 2\sqrt{d}$ is give by EC8 and EBCS 8 it is not in UBC and IBC. The major difference is the rationally computed period for the static analysis. IBC and UBC provide rational computed period with upper bound limit. In EC8 and EBCS-8, however, allow rational time period but they do not provide rational equation and also they do not provide upper bound limit. This variation produces a big difference of base shear among the codes and it may provide clearly on the future edition of EBCS-8.

In previous editions of the U.S. and European codes and in the current Ethiopian code, the actual response spectra in the constant velocity region that varied in a $1/T$ relationship were replaced with design spectra that varied in a $1/T^{2/3}$ relationship to provide added conservatism in the design of tall structures. However, the current major codes (UBC 97, IBC and EC8) recommend that the true shape of the response spectrum, represented by a $1/T$ relationship, be maintained in the base shear equation.

The IBC limits on story drift depend upon the seismic use group, and become tighter for higher use groups. The UBC drift limits are dependent only on the period of the structure, drift limits being tighter for structures having periods greater than or equal to 0.7 seconds. The corresponding regulation of EC8 and EBCS-8 are similar and both are significantly different from IBC and UBC. The EBCS-8 and EC8 Limits on story drift depend on the ductility and fixity of non structural elements. For standard occupancy buildings, the EC8 and EBCS-8 requirement is more stringent than UBC and IBC requirement.

The biggest difference of IBC from other three codes is in the design ground motion parameters. UBC, EC8 and EBCS-8 use seismic zone map giving seismic zone values, often with significant change in value as a boundary between zones is crossed. However, in IBC the seismic zone map has been replaced by contour maps with a high enough spatial resolutions, then changes in hazard over small distances are always relatively subtle. The IBC approach has the advantage any site has its own seismicity and it is expected that in the future seismic zone maps will be changed to seismic contour map with anew seismic values and return period.

The IBC prescribes a linear distribution and a parabolic base shear distribution, for structures with $T \leq 0.5s$ and $T \geq 2.5s$, respectively. The UBC EC8 and EBCS-8 uses a linear distribution based on the shape of the first mode. However, UBC and EBCS-8 account the higher mode effect, for structures with $T \geq 0.7s$, by concentrating a portion of the design base shear ($0.07TV \leq 0.25V$) at the top of the building the remainder of the design base shear being distributed linearly as for short-period structures. This is not the case in EC8.

The scaling provisions of UBC and EBCS-8 are similar. Where the combined response for the modal base shear is less than the calculated static base shear, the forces shall be normalize to 100% of static base shear for irregular buildings and to 90% for regular buildings. IBC, unlike UBC and EBCS8, specify magnifying to 85% for both regular and irregular buildings. The possible 15 percent reduction in design base shear may be considered as an incentive for using a modal response spectrum analysis in lieu of the equivalent lateral force procedure. In EC8 there is no scaling provision.

All codes, considered for comparison, permit ELF analysis for regular structures with height limitation. The height limit in UBC, IBC, EC8 and EBCS-8 are 73.15m, 48.7m, 40 and 80m

respectively. In addition it permitted for irregular structure with restricted limitations. UBC permit ELF for All structures regular or irregular, in seismic zone 1, for occupancy categories 4 and 5 in seismic zone 2 and for all structures not more than five story or 19.812m irrespective of seismic zone. Somewhat similar to UBC, IBC allow ELF for all structures located in seismic category B and C, and for the structures not exceeding 48.7m in structural height and having only some irregularities. EC8 and EBCS-8 in common permit ELF for structures having fundamental periods of vibration T_1 in the two main directions less than 2 sec. and meet the criteria for regularity in plan and/or in elevation.

One of the assumptions In the ELF analyses is that 100% of the mass of the building participated in the first mode. Due to this assumption for regular buildings ELF analysis produce conservative results and becomes more conservative when the height increases. For irregular reinforced concrete moment frame buildings the analysis result of ELF and RSA have no significant differences up to 6 stories. For 12 and 15 story buildings there are some significant differences at some stories.

For complex three dimensional structures the direction of the earthquake which produces the maximum stresses, in a particular member or at a specified point, is not known. Therefore a large number of dynamic analyses at various angles of input in order to check all points for the critical earthquake directions should be performed. The term “principal direction” should not be used unless it is clearly and uniquely defined.

For regular and symmetrical structures, it is reasonable to assume that motions that take place during an earthquake have one principal direction. For these types of buildings the direction of the base shear associated with the fundamental mode of vibration can be used as the major principal direction of the structure. The minor principal direction will be ninety degrees from the major axis.

For three dimensional response spectra analyses, the design of elements using 30% rule is depend on the user’s selection of the reference system. This percentage combination rule is empirical and can be underestimate the design forces in certain members. However, in SRSS rule the maximum member forces calculated are invariant with respect to the selection system and the maximum values for all members can be exactly evaluated from one computer run in which two global dynamic motions are applied.

The SRSS response always overestimates the critical response and 30% rule also overestimates the critical response in most cases. However, 30% rule less conservative than SRSS. Furthermore, 30% rule sometimes can underestimate the design forces in certain members. For three dimensional complex structures, the only rational earthquake design is that perform a large number of dynamic analyses at various angles of input in order to check all points for the critical earthquake directions. However, the cost of such a study would be prohibitive. Therefore, to account this for complex three dimensional structures one can use one of the combination rules. For the buildings which have uniquely defined principal axes and for regular buildings, the spectrum in the direction of the base shear associated with the fundamental mode of vibration provides a good approximate result.

5.2 Recommendation

- In the future the rest of the seismic provisions of EBCS-8 excluded in this thesis should be evaluated.
- Further studies on comparison of EBCS with Asian, Latin American and Australian codes should be encourage.
- The total influence of seismic load variations on the structures should be study in future.
- The manual and the commentary of EBCS-8 should be prepared.

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