

ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

POST GRADUATE PROGRAM

STRUCTURAL ENGINEERING STREAM

**Potential Safety Compromising of Existing Buildings**

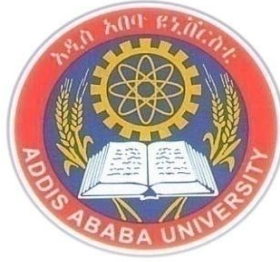
**Based on ES EN 1998:2015**

**(Case Study School Building in Mekele)**

by

**Dawit Zewdie**

Addis Ababa / April 2017



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ES EN 1998:2015  
(Case Study School Building in Mekele)**

A thesis submitted to the school of Graduate Studies of Addis Ababa  
University in partial fulfillment of the requirements for the Degree of  
Master of Science in Civil Engineering  
(Structural Engineering Stream)

by

Dawit Zewdie

Advisor Dr. Shifferaw Taye

(April 2017)

## DECLARATION

I, the undersigned, declare that this thesis entitled '**Potential Safety Compromising of Existing Buildings Based on ES EN 1998:2015**' is my original work and has not been presented for a degree in any other university. All sources of materials used for the thesis have been properly acknowledged.

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Place: Addis Ababa University

April 2017

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This is to certify that the thesis prepared by Dawit Zewdie entitled: *Potential Safety Compromising of Existing Buildings Based on ES EN 1998:2015* and submitted in partial fulfillment of the requirements for the Degree of Master of Science in Civil Engineering (Structural Engineering) complies with the regulations of the University and meets the accepted standards with respect to originality and quality.

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## **ABSTRACT**

Earthquake codes have been revised and updated depending on the improvements in the representation of ground motions, soils and structures. Ethiopian earthquake code has been revised 2015 and introduced as ES EN 1998:2015. Changes in the earthquake code have been performed on peak ground acceleration, ground type and design spectrum. The purpose of this thesis is to investigate the differences caused by code revision of the analysis and safety of existing structures of a given type of buildings. The differences in expression and some important points by EBCS 8 1995 and ES EN 1998:2015 have been considered for comparison. In this thesis the design spectra, base shear, story displacement and story drift for analyzing building were comparatively presented.

From the study, it was found that there was a significant difference in base shear, story displacement and story drift for Soil Type B, C and D accordingly ES EN 1998:2015. The difference was not significant for Soil Type A and E.

The value of demand to capacity ratio considerably increased in ES EN 1998:2015 compared to EBCS 8 1995 on soil type B, C and D.

Finally the case study building was assessed for its performance level using push over analysis. The result shows the structure's performance level not satisfied life safety level.

## **ACKNOWLEDGEMENT**

First of all I would like to thank Almighty God, Who has given me the commitment to face all the challenges encountered during my research study and helped me to pass through all the corridors with confidence and passion and finally come up with a successful accomplishment of this thesis.

I would like to express my gratitude to my advisor Dr Shifferaw Taye for believing in me and for providing me with invaluable advice throughout duration of this research. I am also thankful to him for constructive reviews of the thesis.

I would like to thank Wollo University for their full sponsorship throughout this period.

This thesis would not have been possible to produce without the support of my parents. I owe very much to them, in fact everything.

Finally, I would like to express my sincerest thanks to all AAiT instructors, staff and my friends for their encouragement.

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## SYMBOLS AND NOTATIONS

$A_{Ed}$	design value of seismic action
$a_g$	design ground acceleration on type A ground
$d_r$	design interstorey drift
$E_d$	design value of action effects
$F_i, \bar{F}_i$	horizontal seismic force at storey $i$
$F_b$	base shear force
$g$	acceleration of gravity
$H$	building height from the foundation or from the top of a rigid basement
$h$	interstorey height
$L_{max}, L_{min}$	larger and smaller in plan dimension of the building measured in orthogonal directions
$m_i$	mass of storey $i$
$n$	number of storeys above the foundation or the top of a rigid basement
$N_{SPT}$	Standard Penetration Test blow-count
$S_d(T)$	design spectrum (for elastic analysis)
$S$	soil factor
$S_a$	seismic coefficient for non-structural elements
$q$	behaviour factor
$T_l$	fundamental period of vibration of a building
$T$	vibration period of a linear single degree of freedom system
$c_u$	undrained shear strength of soil
$v_{s,30}$	average value of propagation velocity of S waves in the upper 30 m of the soil profile at shear strain of 10 <sup>-5</sup> or less
$\alpha$	ratio of the design ground acceleration to the acceleration of gravity
$\gamma_I$	importance factor
$\eta$	damping correction factor
$\xi$	viscous damping ratio (in percent)
$\psi_{2,i}$	combination coefficient for the quasi-permanent value of a variable action $i$
$\psi_{E,i}$	combination coefficient for a variable action $i$ , to be used when determining the effects of the design seismic action
$\gamma_a$	importance factor of a non-structural element (appendage)
$\theta$	interstorey drift sensitivity coefficient

# CHAPTER ONE

## INTRODUCTION

### 1.1 Background

The first seismic code for buildings in Ethiopia was introduced in 1980 [3]. That code defined four seismic regions (i.e., 0, 1, 2, and 4) with a return-period of 100 years. The next revision was introduced in 1984 as ESCP1-83 [3]. ESCP1-83 had a separate code (ESCP-2:1983 - Ethiopian Standard Code of Practice for the Structural use of Concrete) for guidelines for concrete design. Those two were followed by a substantial change introduced in 1995 as EBCS-1995 by the Ministry of Works & Urban Development [1]. The seismic zoning was an improvement over previous codes based on additional data obtained from newer earthquake records inside Ethiopia as well as in neighboring countries.

A commonality between all the three codes introduced in the country over the past 40 years is the choice of 100 years return-period in contrast with a 475 years return-period which is adopted by most codes around the world [3]. The main argument in favor of this choice has been the relatively economical construction of structures designed for a less powerful earthquake.

New revision was introduced as Ethiopian Standards based on Euro Norms ES EN 1998:2015 by Ministry of Construction [2]. This code has been prepared based on 475 years return period and considering other criteria. This code defined five seismic zones (i.e., 0, 1, 2, 3, 4 and 5) and the value of peak ground acceleration (PGA) 0, 0.04, 0.07, 0.10, 0.15 and 0.20 respectively. In general, PGA values corresponding to a return-period of 475 years are increased by about 33.3% to 50.0% those of 100 years return-period. The changes in PGA, parameters and analysis criteria change the seismic demand of the structures. This leads to check the level of safety for existing structure based on the seismic demand of ES EN 1998:2015.

Building performance evaluation involves the assessment of the extent to which a given building has to meet its design goals like intended purposes, occupant satisfaction, and withstand the load to which it is exposed to in its design life. The primary purpose of building performance evaluation is to improve design practice and to ensure the continuous improvement of design methods and design code, through the provision of feedback to

designers on the effectiveness of their design choices. This involves global perspective like story displacement, story drift, stability of structure for lateral load, regularity and so on, section perspective like capacity of the section that can withstand the stress to which it is being exposed to and member perspective, resistance to axial and flexure.

## **1.2 Case study building information**

The building is a four story school building. The building's overall dimensions are 10.34m wide by 36.00m long, and it is 17.50m tall from ground level. The building is being used as a school building. The building consists of a reinforced concrete frame system and has solid slab floor system. The building constructed in Mekele university and designed according to an old Ethiopian Building Code of standards EBCS 1995.

## **1.3 Statement of the problem**

At this time new code was introduced as Ethiopian Standards based on Euro Norms ES EN 1998:2015. The old code was constructed based on return-period of 100 years not conservative enough for buildings as well as large infrastructures. In ES EN 1998:2015 the return-period changed to 475 years. Also, seismic hazard map and soil classification was changed. The peak ground acceleration was increased 33.3% to 50.0% and other requirement in old code was changed in new code, this leads to great variation on seismic demand of structure and other requirement. Most of the existing structure was designed based on EBCS 1995 which is peak ground acceleration almost 50.0% reduced than the current. The existing buildings designed with old code should be evaluate the level of safety based on new code seismic demand.

## **1.4 Objectives**

### **1.4.1 General objective**

The general objective of this thesis is to investigate the differences caused by code revision in the analysis method and to evaluate safety compromising of the existing structure of a given type of building based on ES EN 1998:2015 seismic demand.

### **1.4.2 Specific objective**

To compare different parameters of EBCS 8 1995 and ES EN 1998:2015.

To compare the response of base shear of the two codes

To compare seismic demand differences on story displacement of the EBCS 8 1995 and ES EN 1998:2015.

To compare the story drift variation of the two codes.

To compare demand to capacity ratio of members due of EBCS 8 1995 and ES EN 1998:2015.

To evaluate the seismic performance of the existing building using pushover analysis based on new code.

## **1.5 Scope of the study**

In this thesis types of procedure which are equivalent lateral force analysis and response spectrum analysis procedure are covered. Under the two procedure ground type classifications, elastic response spectra, seismic hazard map, peak ground acceleration, regularity criteria, selection of analysis procedure criteria and limitation of story drift are addressed. Detailing mechanism is not included in this thesis. For the seismic demand and performance evaluation this thesis only addresses low rise building. During the performance evaluation of case study building only working drawing is used. Any type of damage is not considered.

## CHAPTER TWO

### LITERATURE REVIEW

#### 2.1 Introduction

Extensive amount of earthquake records on Ethiopia exist, the structural damage to infrastructures in the vast part of this period was obviously very low due to the extreme limitation of built-up environments in the country. It is only, perhaps, starting from the 1950s and 1960s that one sees what could be characterized as noticeable building and infrastructure activity in the country, particularly in the seismic-prone areas. The period between 1960 and 1978 provides a wealth of information on the response of built-up structures like buildings and bridges to some of the strongest and damaging earthquakes such as Karakore [1961] and Serdo [1969]. Interestingly, this period coincides with a growth in built-up areas and infrastructure in some of the seismically active areas. Therefore, it has increasingly become clear that structural damages to buildings and infrastructure due to earthquakes are on the rise in the Country. Figures 2.1 and 2.2 show the distribution of damage-causing earthquakes in Ethiopia with damage attributed property or injury [3].



Figure 2.1 Damage to Wetera Abbo Church in Wondo Genet during the 1983 earthquake [3]



Figure 2.2 Structural damage in December 2010 Hossana earthquake.

Seismic Zoning of Ethiopia as per CP1-78, ESCP1-83 and EBCS-8:1995 [1] all considered four seismic zones; but in ES EN 1998:2015, it changes to five. Figure 2.3 show Ethiopia's hazard map in terms of peak ground acceleration based on ES EN 1998:2015. Also in ES EN 1998:2015 the peak ground acceleration was changed for each zone due return period [2].

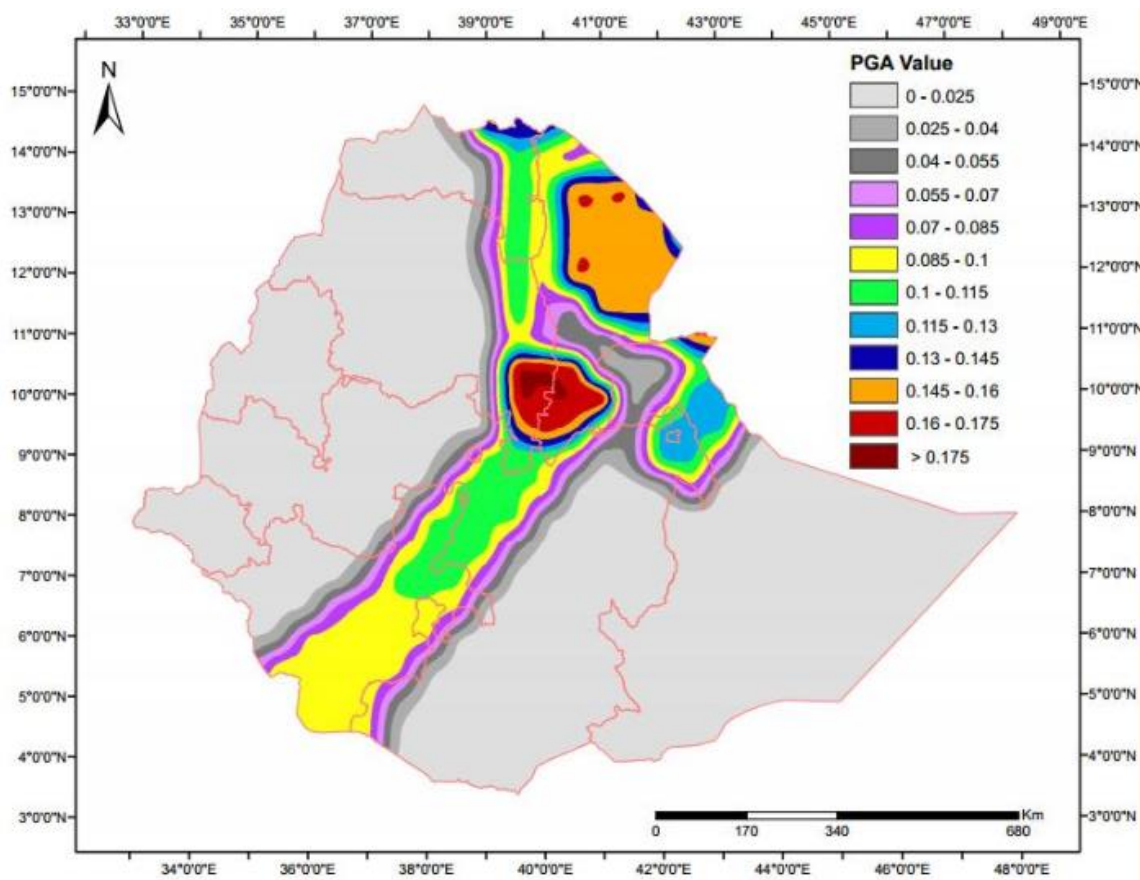


Figure 2.3 Ethiopia's seismic hazard map in terms of ground acceleration ES EN 1998:2015.

Table 2.1 Bedrock acceleration ratio

Zone	5	4	3	2	1	0
$\alpha_0=a_g/g$	0.2	0.15	0.1	0.07	0.04	0

## 2.2 Response spectrum

In order to perform the seismic analysis and design of a structure, the actual time history record is required. However, it is not possible to have such records at each and every location. Furthermore, the seismic analysis of structures cannot be carried out simply based on the peak value of the ground acceleration as the response of the structure depend upon the frequency content of ground motion and its own dynamic properties [6]. To overcome the above difficulties, earthquake response spectrum is the most popular tool in the seismic analysis of structures [6]. There are computational advantages in using the response spectrum method of seismic analysis for prediction of displacements and member forces in structural systems. The method involves the calculation of only the maximum values of the displacements and member forces in each mode of vibration using smooth design spectra that are the average of several earthquake motions.

Now a central concept in earthquake engineering, the response spectrum provides a convenient means to summarize the peak response of all possible linear SDF systems to a particular component of ground motion. It also provides a practical approach to apply the knowledge of structural dynamics to the design of structures and development of lateral force requirements in building codes [6].

A plot of the peak value of a response quantity as a function of the natural vibration period of the system, or a related parameter such as circular frequency  $\omega_n$  or cyclic frequency  $f_n$  is called the response spectrum for that quantity [6]. Each such plot is for SDF systems having a fixed damping ratio and several such plots for different values of damping ratio are included to cover the range of damping values encountered in actual structures. Whether the peak response is plotted against  $\omega_n$  or  $T_n$  is a matter of personal preference. Most engineers prefer to use natural period rather than natural frequency because the period of vibration is a more familiar concept and one that is intuitively appealing [6].

Response spectrum depends on peak ground acceleration, ground types and damping correction factor with a reference value of viscous damping [7]. Figure 2.4 show typical shape of elastic design spectra.

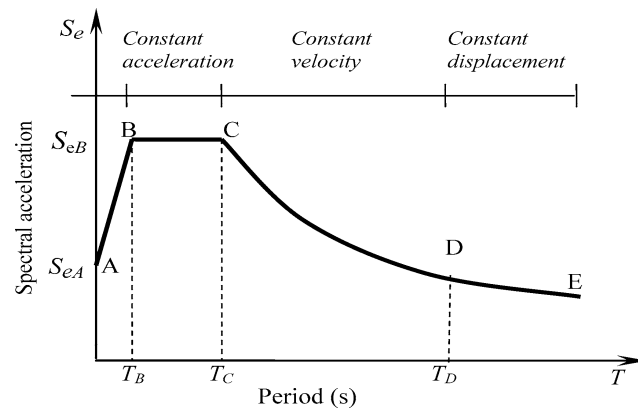


Figure 2.4 Typical shape of elastic design spectra [7].

## 2.3 Base shear

Earthquakes produce lateral forces proportional to the weight of the structure and its fixed contents; the resultant of seismic force is known as ‘base shear’. The value of the seismic base shear selected will depend on the response spectra, the level of ductility capacity assumed to be appropriate for the building type, and the acceptable probability of exceedance of the design earthquake [8]. Inelastic response spectra should be used to assess the influence of ductility and period, since the assumption, common to many building codes, of a constant force reduction factor to be applied to an elastic response spectrum to allow for ductility is un conservative for short-period structures [8].

## 2.4 Analysis type

### 2.4.1 Equivalent lateral force analysis

All design against seismic loads must consider the dynamic nature of the load. However, for simple regular structures, analysis by equivalent linear static methods is often sufficient [10]. This is permitted in most codes of practice for regular, low- to medium-rise building. It begins with an estimation of base shear load and its distribution on each story established by code requirements [10]. Equivalent static analysis can, therefore, work well for low- to medium-rise buildings without significant coupled lateral-torsional modes, in which only the first mode in each direction is considered. The method is less suitable for tall buildings

(over, say, 75 m), where second and higher modes can be important, or buildings with torsional effects; they require more complex methods to be used in these circumstances [10].

### **2.4.2 Response spectrum analysis**

The seismic forces strikes the foundation of a structure will move with the ground motion. It shows that structure movement is generally more than the ground motion. The movement of the structure as compared to the ground is referred as the dynamic amplification. It depends on the natural frequency of vibration, damping, type of foundation, method of detailing of the structure. The response “design acceleration spectrum” which refers to the max acceleration called spectral acceleration coefficient  $S_a/g$ , as a function of the structure for a specified damping ratio for earthquake excitation at the base for a single degree freedom system.

### **2.4.3 Nonlinear static analysis**

Pushover analysis is a simplified analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached. Pushover analysis consists of a series of sequential elastic analysis, superimposed to approximate a force-displacement curve of the overall structure. A two or three dimensional model which includes bilinear or trilinear load-deformation diagrams of all lateral force resisting elements is first created and gravity loads are applied initially.

A predefined lateral load pattern which is distributed along the building height is then applied. The lateral forces are increased until some members yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve.

Pushover analysis can be performed as force controlled or displacement controlled. In force controlled pushover procedure, full load combination is applied as specified, that is, force controlled procedure should be used when the load is known. Also, in force-controlled pushover procedure some numerical problems that affect the accuracy of

results occur since target displacement may be associated with a very small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects. Pushover analysis is the preferred tool for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is conceptually and computationally simple.

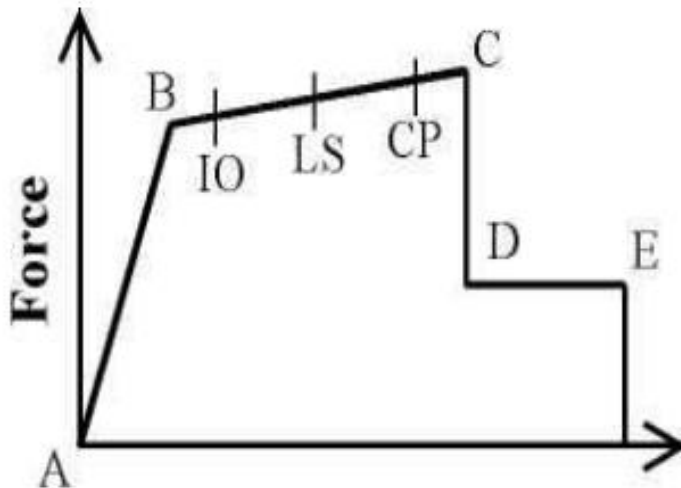


Figure 2.5 Force-deformation curve

- Point A corresponds to unloaded condition.
- Point B represents yielding of the element.
- The ordinate at C corresponds to nominal strength and abscissa at C corresponds to the deformation at which significant strength degradation begins.
- The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is usually unreliable.
- The residual resistance from D to E allows the frame elements to sustain gravity loads. Beyond point E, the maximum deformation capacity, gravity load can no longer be sustained.

### **Structural performance levels and ranges**

The structural performance level of a building shall be selected from four discrete structural performance levels and two intermediate structural performance ranges defined in this section [13].

The discrete structural performance levels are immediate occupancy, life safety, collapse prevention, and not considered. The intermediate structural performance ranges are the damage control range and the limited safety range. Acceptance criteria for performance within the damage control structural performance range shall be obtained by interpolating the acceptance criteria provided for the immediate occupancy and life safety structural performance levels. Acceptance criteria for performance within the limited safety structural performance range shall be obtained by interpolating the acceptance criteria provided for the Life safety and collapse prevention structural performance levels [13].

#### **A. Operational building performance**

Buildings meeting this target building performance level are expected to sustain minimal or no damage to their structural and nonstructural components. The building is suitable for its normal occupancy and use, although possibly in a slightly impaired mode, with power, water, and other required utilities provided from emergency sources, and possibly with some nonessential systems not functioning.

Buildings meeting this target building performance Level pose an extremely low risk to life safety. Under very low levels of earthquake ground motion, most buildings should be able to meet or exceed this target building performance level. Typically, however, it will not be economically practical to design for this target building performance level for severe ground shaking, except for buildings that house essential services [13].

#### **B. Immediate occupancy building performance level**

Buildings meeting this target building performance level are expected to sustain minimal or no damage to their structural elements and only minor damage to their nonstructural components. While it would be safe to reoccupy a building meeting this target building performance level immediately following a major earthquake, nonstructural systems may not function, either because of the lack of electrical power or internal damage to equipment. Therefore, although immediate re occupancy of the building is possible, it may be necessary to perform some cleanup and repair and await the restoration of utility service before the building can function in a normal mode. The risk to life safety at this target building performance level is very low.

Many building owners may wish to achieve this level of performance when the building is subjected to moderate earthquake ground motion. In addition, some owners may desire such performance for very important buildings under severe earthquake ground shaking. This level provides most of the protection obtained under the Operational Building Performance Level without the cost of providing standby utilities and performing rigorous seismic qualification of equipment performance [13].

### **C. Life safety building performance level**

Buildings meeting this level may experience extensive damage to structural and nonstructural components. Repairs may be required before re occupancy of the building occurs, and repair may be deemed economically impractical. The risk to life safety in buildings meeting this target building performance level is low.

This target building performance level entails somewhat more damage than anticipated for new buildings that have been properly designed and constructed for seismic resistance when subjected to their design earthquakes. Many building owners will desire to meet this target Building Performance Level for severe ground shaking [13].

### **D. Collapse prevention building performance level**

Buildings meeting this target building performance level may pose a significant hazard to life safety resulting from failure of nonstructural components. However, because the building itself does not collapse, gross loss of life may well be avoided. Many buildings meeting this level will be complete economic losses. This level has been sometimes selected as the basis for mandatory seismic rehabilitation ordinances enacted by municipalities, as it results in mitigation of the most severe life-safety hazards at relatively low cost [13].

## **2.4.4 Time history analysis**

It is an analysis of the dynamic response of the structure at each increment of time, when its base is subjected to a specific ground motion time history. Alternatively, recorded ground motions database from past natural events can be a reliable source for time histories but they are not recorded in any given site to include all seismological characteristics suitable for that site. Recorded ground motions are randomly selected from analogous magnitude, distance and soil condition category; three main parameters in time history

generation. Adding more constraints to characteristics of each soil condition category make to be more definite and similar site characteristics. However it may put serious availability limit for real records in the soil condition category. Selected ground motion response spectrum fundamental period of the structure can be different than target response spectrum determined from seismic hazard analysis. Therefore records are scaled by a single factor scales to have their mean spectral accelerations complied with target spectrum. Nevertheless, not much close agreement between the response spectrum of the record and target will be achieved with simply a single factor scaling of the records [10].

## **2.5 Stability of structure for lateral load**

A frame structure can be considered to be braced in a given direction if the lateral stability of the structure as a whole is provided by walls, bracing, buttresses designed to resist all lateral forces in that direction. In actuality, there is no such thing as “completely braced” frames. Other frames are connected to shear walls, elevator shafts, and so on, which clearly restrict the lateral movement of the frame. Because no wall is completely rigid, however, there will always be some lateral movement of a braced frame, and hence some  $P\Delta$  moments result from the lateral movements. For the purpose of design, a story or a frame may be considered “non-sway,” if horizontal displacements do not significantly reduce the load carrying capacity of the structure. This criterion could be restated as follows: a frame can be considered “non-sway”, if the  $P\Delta$  moments due to lateral deflections are small compared to the 1<sup>st</sup>-order moments due to lateral loads.

# CHAPTER THREE

## MATERIAL AND METHODOLOGY

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

### 3.1 Assessment of seismic specifications

Assessment seismic specifications of EBCS 8 1995 and ES EN 1998:2015 seismic code provisions from the perspective of principle of earthquake engineering have been done.

Compare some section of seismic provision of EBCS-8 1995 and ES EN 1998:2015 such as ground type classification, peak ground acceleration, regularity criteria, selection of analysis procedure, base shear coefficient, distribution of base shear, consideration of torsional effect, story drift determination and limitation and P-delta effect consideration have been compared. In addition the design spectra for all Soil Type are considered for comparison.

### 3.2 Identification of case study buildings

To accomplish this study one case study reinforced concrete school building was chosen. The building has had five stories, including ground floor. It was constructed in Mekele city, Ethiopia. Its overall dimensions are 10.34m wide by 36m long, and it is 17.5m tall from ground level. The building is being used as an school building. It consists of a reinforced concrete frame system, with the solid slab floor system.

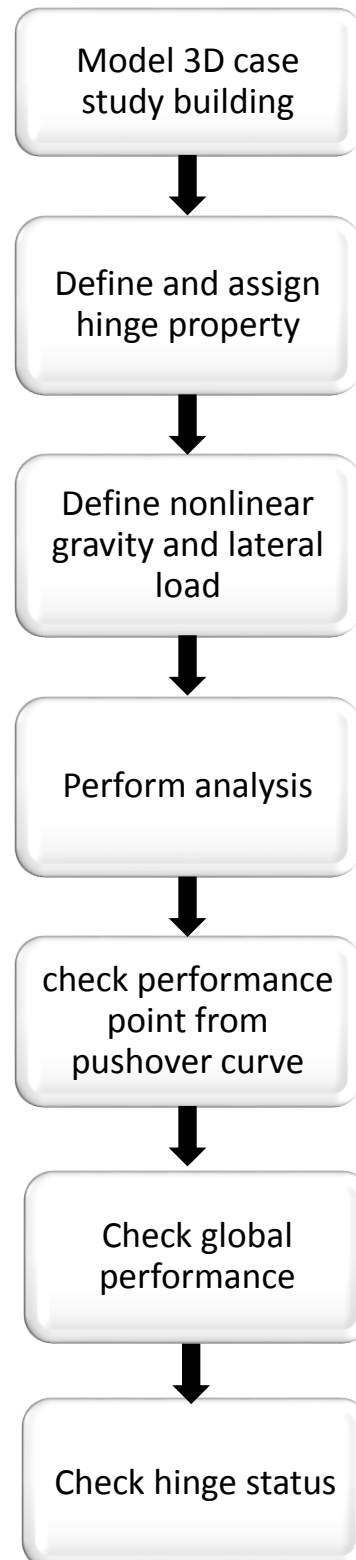
### 3.3 Performance assessment

With the help Integrated Building Design Software ETABS 2015 Version 15.0.0 and based on ES EN 1998:2015.

- ✓ Determining the conventional demand to capacity ratio of members of building.
- ✓ Identifying highly vulnerable members/areas of the structure and its level of risk.

### 3.4 Pushover analysis strategy

A displacement-controlled pushover analysis by Structural Analysis Program SAP2000 Ultimate 15.0.0 used for this study, the following steps perform during performance checking



# **CHAPTER FOUR**

## **COMPARISON OF PROVISIONS OF EBCS 8 1995 AND ES EN 1998:2015**

### **4.1 Introduction**

Earthquake codes are periodically revised and updated depending on the improvements in the representation of ground motions, soils and structures. Moreover, these revisions have been made more frequently in recent years.

A comparison of the main elements of the elements and analysis provisions of the two codes approaches is summarized as following sub topics. Both code approaches require different analysis procedures depending on the assumptions and time of preparing.

### **4.2 Ground type**

The site conditions have been classified into different categories in earthquake codes. These categories are named ground types, soil profile types, local site classes or subsoil classes. Three and five ground types are defined in EBCS 8 1995 and ES EN 1998:2015, respectively.

EBCS 8 1995 site classification system is based only on representative average shear wave velocity. ES EN 1998:2015 uses standard penetration test blow-count and unconfined compression strength in addition to a representative average shear wave velocity to classify the ground type. Table 4.1 shows the differences ground types based on EBCS 8 1995 and ES EN 1998:2015.

Table 4.1 Ground types defined in the EBCS 8 1995 and ES EN 1998:2015

EBCS 8 1995		ES EN 1998:2015				
Ground type	Description of stratigraphic profile	Ground type	Description of stratigraphic profile	$v_{s,30}$ (m/s)	NSP T (blows/30cm)	cu (kPa)
A	Rock or other geological formation $v_s > 800$ m/s, including at most 5m of weaker material at the surface and Stiff deposits of sand, gravel or over consolidated clay, at least several tens of meters thick $v_s > 400$ m/s at a depth of 10m	A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	–	–
		B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
B	Deep deposits of medium dense sand, gravel or medium stiff clays with thickness from several tens to many hundreds of meter $v_s > 200$ m/s at depth of 10m	C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.	180 – 360	15 - 50	70 - 250
C	Loose cohesionless soil deposits with or without some soft cohesive layers, $v_s < 200$ m/s in the uppermost 20m.	D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
		E	A soil profile consisting of a surface alluvium layer with $v_s$ values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $v_s > 800$ m/s.			
		S1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	–	10 - 20
		S2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S1			

Table 4.1 Site coefficient for ground types

EBCS 8 1995		ES EN 1998:2015		
Ground type	Site coefficient S	Ground type	Site coefficient S	
			Type 1	Type 2
A	1.00	A	1.00	1.00
		B	1.20	1.35
B	1.20	C	1.15	1.50
C	1.50	D	1.35	1.80
		E	1.40	1.60

In ES EN 1998:2015 soil coefficient is given depend on the type of analysis, but in EBCS 8 1995 only one type of analysis was defined and the site coefficient also not depend on analysis type.

### 4.3 Seismic zoning

Probabilistic seismic hazard zonation of Ethiopia was classified into four and five zone in EBCS 8 1995 and ES EN 1998:2015 respectively. Peak ground acceleration is depends on the zone type (location) and return period. Peak ground acceleration in EBCS 8 1995 and ES EN 1998:2015 was prepared based 100 years and 475 years return-period respectively. Also, seismic hazard map have a difference in two codes. The peak ground acceleration was increased 33.3%-50.0% in different zone type in the new code. In some city i.e Addis Ababa peak ground acceleration increased by 100%. Table 4.3 and Figure 4.1 shows the difference in zone type and peak ground acceleration of the two codes.

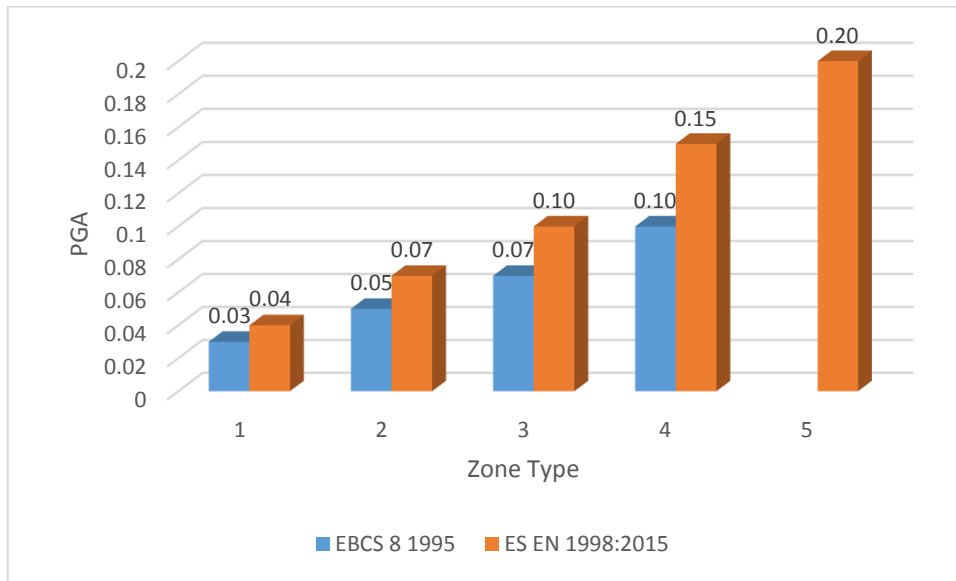


Figure 4.1 Peak ground acceleration value for EBCS 8 1995 and ES EN 1998:2015

Table 4.3 Peak ground acceleration comparison of some town based on EBCS 8 1995 and ES EN 1998:2015

Town	Hawassa	Addis Ababa	Mekele	Dire dewa	Jima	Gewane	Kombolcha
EBCS 8 1995 - $\alpha_0$	0.1	0.05	0.1	0.05	0.03	0.1	0.1
EBCS EN 1998 2014 - $\alpha_0$	0.15	0.1	0.15	0.01	0.04	0.2	0.1
Increased by	50%	100.0%	50%	100.0%	33.3%	100.0%	0.0%

The bed rock acceleration ratio for zone 4, 3, 2 and 1 was increased by 50.0%, 42.8%, 40.0%, and 33.3% respectively in ES EN 1998:2015 from EBCS 8 1995.

#### 4.4 Response spectrum

Response spectrum is depending on peak ground acceleration, ground types and damping correction factor with a reference value of viscous damping. ES EN 1998:2015 has a note starting that, if deep geology is not accounted for, the recommended choice is the use of two types of spectra: Type 1 and Type 2. If the earthquakes that contribute most to the

seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude,  $M$ , not greater than 5.5, it is recommended that the Type 2 spectrum is adopted. EBCS 8 1995 have only one type of spectrum.

EBCS 8 1995 recommend vertical component response spectrum represent by 70% of horizontal response spectrum, but ES EN 1998:2015 develops its own vertical response spectrum curve. Figure 4.2 and 4.3 shows the horizontal elastic response spectrum of the EBCS 8 1995 and ES EN 1998:2015, respectively for each ground type.

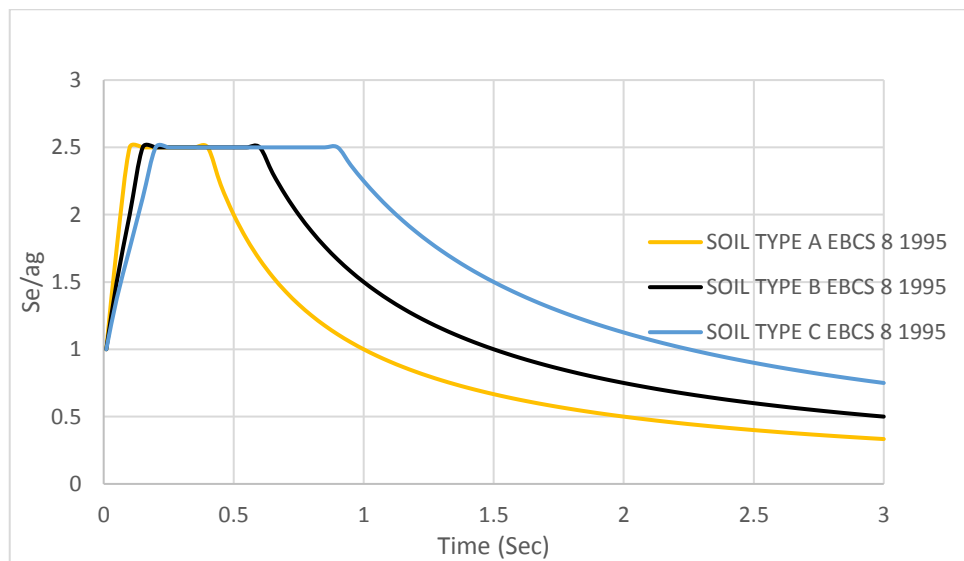


Figure 4.2 Horizontal elastic response spectrum based on EBCS 8 1995

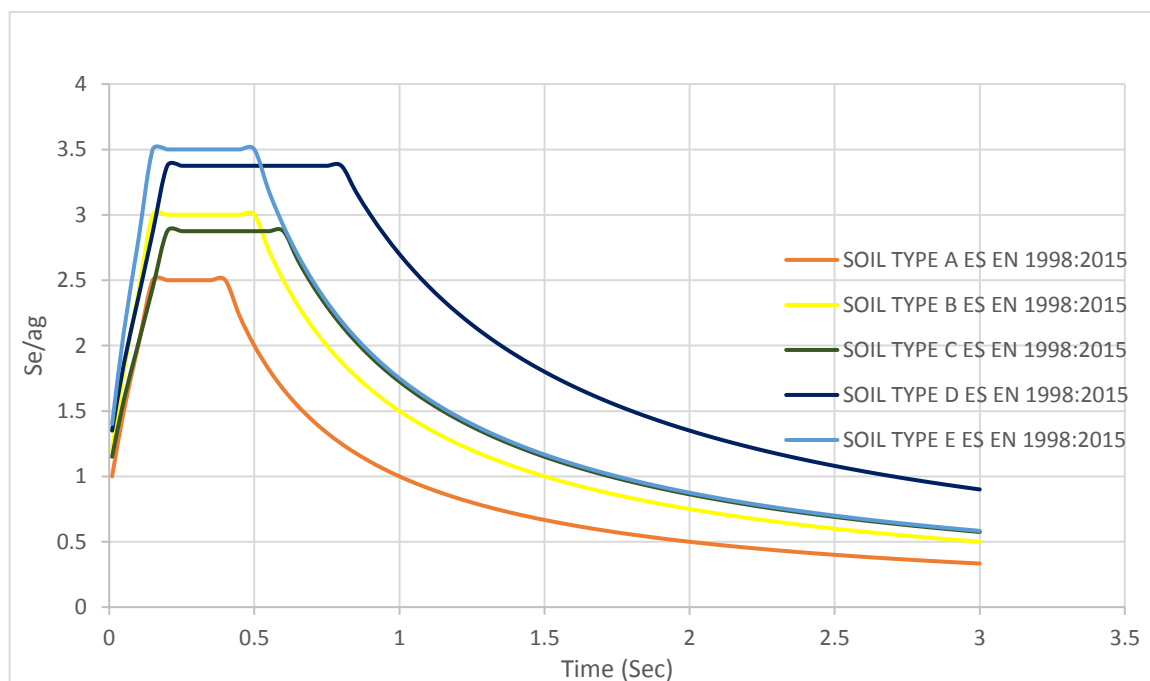


Figure 4.3 Type 1 elastic response spectrum based on ES EN 1998:2015

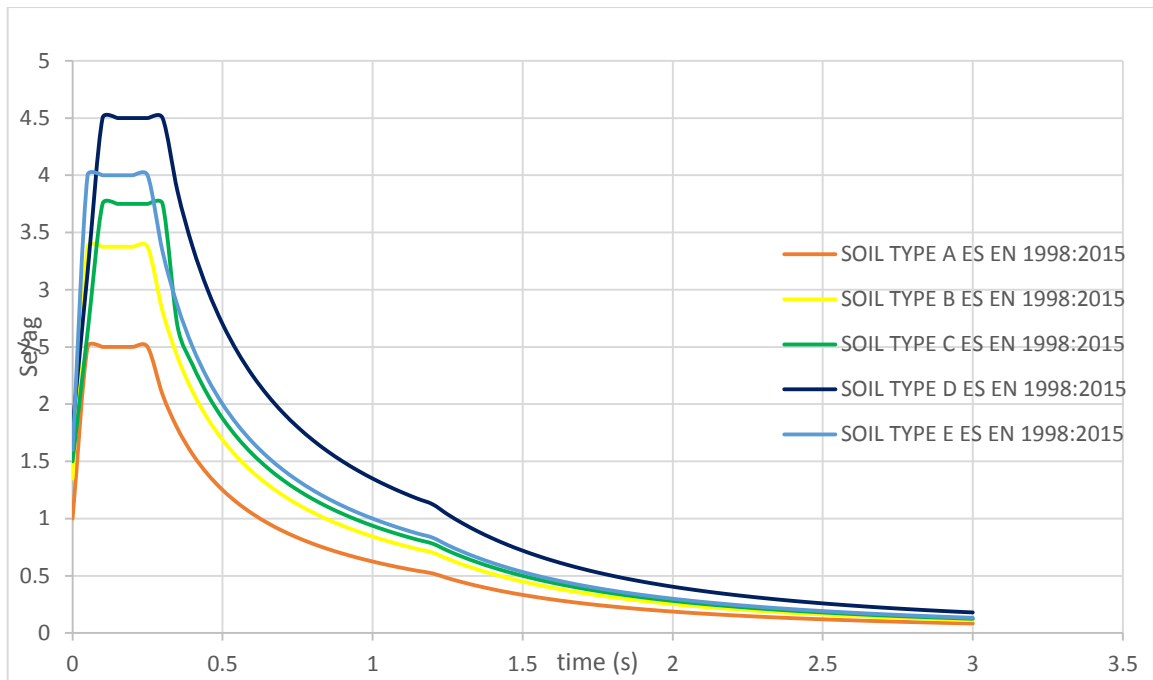


Figure 4.4 Type 2 elastic response spectrum based on ES EN 1998:2015

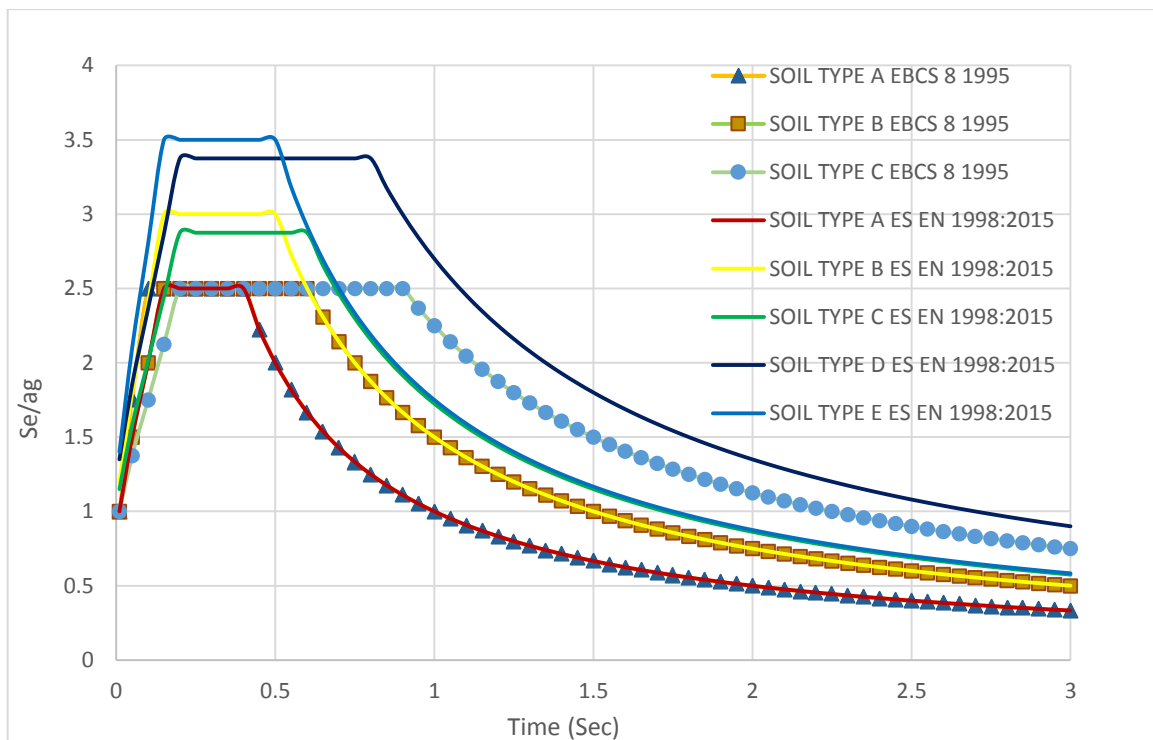


Figure 4.5 Horizontal elastic response spectrum of EBCS 8 1995 and Type 1 ES EN 1998:2015

Maximum normalized spectral acceleration of EBCS 8 1995 not more than 2.5 for all types of soil. Maximum spectral acceleration in ES EN 1998:2015 vary from 2.5 to 3.5 from soil type A to E. Figure 4.4 show normalized elastic response spectrum of the two codes.

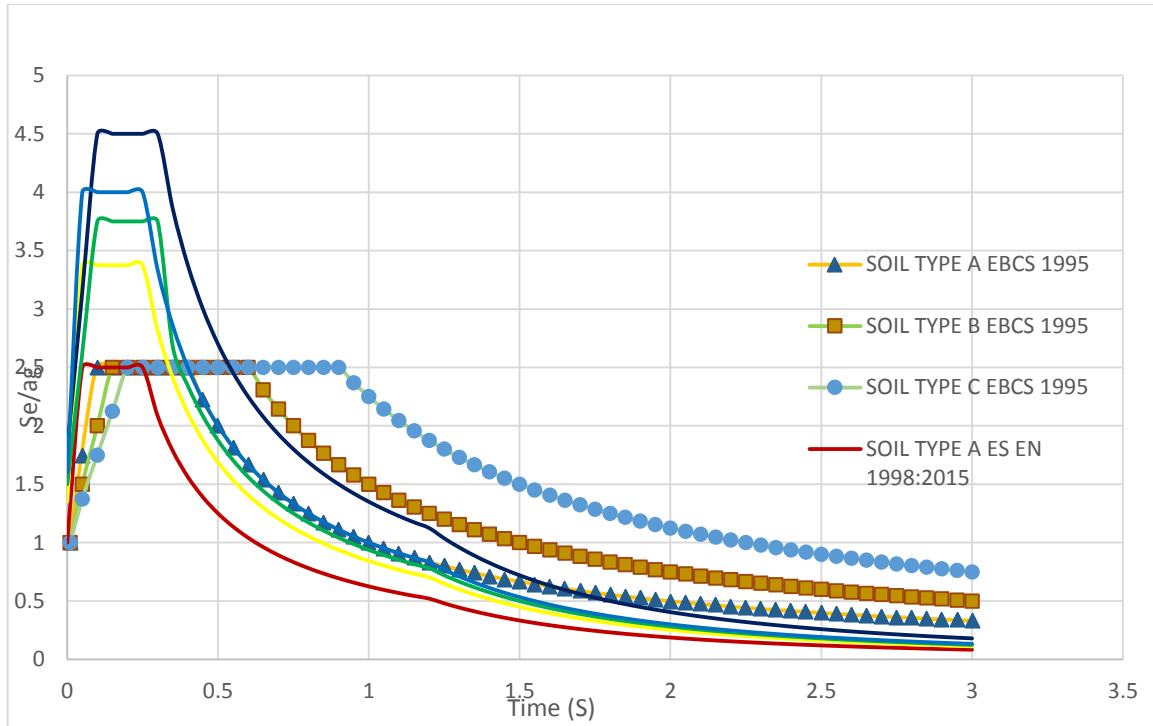


Figure 4.6 Horizontal elastic response spectrum of EBCS 8 1995 and Type 1 ES EN 1998:2015

## 4.5 Structural regularity

On structural regularity the two codes have similar criteria except the following additional points

EBCS 8 1995 mentioned that, under seismic force distribution applied with accidental eccentricity at any storey, the maximum displacement in the direction of the seismic forces shall not exceed the average storey displacement by more than 20%. But in ES EN 1998:2015 such limitation does not given.

ES EN 1998:2015 limit the Slenderness of the building ( $\lambda = L_{max}/L_{min}$ ) in plan shall be not higher than 4, where  $L_{max}$  and  $L_{min}$  are larger and smaller plan dimension of the building respectively. There is no slenderness limitation given in EBCS 8 1995

## 4.6 Structural analysis

Table 4.2 Analysis criteria of EBCS 8 1995 and ES EN 1998:2015

Analysis type	EBCS 8 1995	ES EN 1998:2015
Equivalent static analysis	<ul style="list-style-type: none"> <li>• Regular building</li> <li>• Fundamental period T less than 2 s</li> <li>• Building height less than 80m</li> </ul>	<ul style="list-style-type: none"> <li>• Regular building</li> <li>• Fundamental period T less than 2 or 4T<sub>c</sub> s</li> <li>• Building height less than 40m</li> </ul>
Response spectrum analysis	<ul style="list-style-type: none"> <li>• All type of building</li> </ul>	<ul style="list-style-type: none"> <li>• All type of building</li> </ul>
Pushover analysis	<ul style="list-style-type: none"> <li>• Not mentioned</li> </ul>	<ul style="list-style-type: none"> <li>• Available for checking performance of existing building and new design</li> </ul>
Time history analysis	<ul style="list-style-type: none"> <li>• Not mentioned</li> </ul>	<ul style="list-style-type: none"> <li>• Available for required structure.</li> </ul>

## 4.7 Base shear

Earthquakes produce lateral forces proportional to the weight of the structure and its fixed contents; the resultant of seismic force is known as ‘base shear’. The value of the seismic base shear selected will depend on the response spectra, the level of ductility capacity assumed to be appropriate for the building type, and the acceptable probability of exceedance of the design earthquake. The two codes have basic different on base shear calculation.

Table 4.3 Base shear and design spectrum calculation of EBCS 1995 and ES EN 1998:2015

EBCS 1995	ES EN 1998:2015
$F_b = S_d(T_1) \cdot W$ $S_d(T_1) = \alpha \beta \gamma$ $\alpha = \alpha_0 I$ $\beta = \frac{1.25}{T^3} \leq 2.5$	$F_b = S_d(T_1) m \lambda$ $0 \leq T \leq T_B: S_d(T) = a_g \cdot S \cdot \left[ \frac{2}{3} + \frac{T}{T_B} \cdot \left( \frac{2.5}{q} - \frac{2}{3} \right) \right]$ $T_B \leq T \leq T_C: S_d(T) = a_g \cdot S \cdot \eta \cdot \frac{2.5}{q}$ $T_B \leq T \leq T_C: S_d(T) = \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[ \frac{T_C}{T} \right] \\ \geq \beta \cdot a_g \end{cases}$ $T_D \leq T: S_d(T) = \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[ \frac{T_C \cdot T_D}{T^2} \right] \\ \geq \beta \cdot a_g \end{cases}$

$$T_1 = C_1 H^{\frac{3}{4}}$$

$$\gamma = \gamma_0 k_D k_R k_W \leq 0.7$$

Where  $S_d(T_1)$  = is the ordinate of the design spectrum

$T_1$  = is the fundamental period of vibration of the building for lateral motion in the direction considered;

$M$  = is the total mass of the building, above the foundation or above the top of a rigid basement.

$\lambda$  is the correction factor, the value of which is equal to:  $\lambda = 0.85$  if  $T_1 < 2 T_C$  and the building has more than two storeys, or  $\lambda = 1.0$  otherwise.

## 4.8 Seismic mass

EBCS 8 1995 considered seismic mass total permanent load only except for storage building it allows additional of 25% of live load from permanent load. ES EN 1998:2015 calculate seismic mass as follow.

$$W = \sum G_K + \sum \psi_{E,i} \cdot Q_K$$

The coefficient  $\psi_{E,i}$  is used to estimate a likely value of service loads and to take into account that some masses do not follow perfectly the moves of the structure, because they are not rigidly connected to the structure.  $\psi_{E,i}$  is computed as:

$$\Psi_E = \varphi \cdot \psi_{2,i}$$

Values of  $\psi_{2,i}$  and  $\varphi$  are listed in Table 4.6. It can be noticed that the coefficient  $\psi_{E,i}$  which is used to define the mass of the service load present on average over the building height can be much lower than one.

Table 4.4 Coefficients  $\psi_{2,i}$  and  $\phi$

Specific Use	$\psi_{2,i}$	Storey	$\Phi$
Cat. A : residence	0.3	Roof	1.0
Cat. B:office	0.3	Storey's with correlated occupancies	0.8
Cat. C : meeting rooms, places where people congregate	0.6	Independently occupied storey's	0.5
Cat. D: shopping area	0.6		1.0
Cat. E: storage, accumulation of goods	0.8		
Cat. F:traffic(vehicle < 30kN)	0.6		

## 4.9 Distribution of horizontal seismic force

Table 4.5 Method of horizontal static seismic load distribution in EBCS 8 1995 and ES EN 1998:2015

EBCS 1995	ES EN 1998:2015
$F_i = \frac{(F_b - F_t) W_i h_j}{\sum_{j=1}^n W_i h_j}$ $F_t = 0.07 T_1 F_b$	$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_j \cdot m_j}$ $F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum z_j \cdot m_j} \quad \text{for linear displacement shape}$

In EBCS 8, 1995 the distribution of horizontal seismic force was estimated based on considering the concentrated force at the top of the building ( $F_t$ ). But in ES EN 1998:2015  $F_t$  was not considered.

#### 4.10 Limitation of inter story drift

Table 4.6 Drift limitation criteria in EBCS 8 1995 and ES EN 1998:2015

EBCS 8 1995	ES EN 1998:2015
<ul style="list-style-type: none"> <li>➤ For having non-structural elements of brittle material attached to the structure ; <math>d_r \leq 0.01h</math></li> <li>➤ For having non-structural elements fixed in a way as not to interfere with structural deformation of brittle material attached to the structure ; <math>d_r \leq 0.015h</math></li> </ul>	<ul style="list-style-type: none"> <li>➤ for buildings having non-structural elements of brittle materials attached to the structure: <math>d_{r,v} \leq 0.005h</math>;</li> <li>➤ for buildings having ductile non-structural elements: <math>d_{r,v} \leq 0.0075h</math>;</li> <li>➤ for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements: <math>d_{r,v} \leq 0.010h</math></li> </ul>

Where  $d_r$ - inter story drift

Regarding to limitation of inter story drift ES EN 1998:2015 was classified the structure into three major components while EBCS 8 1995 classified into two major classes. For buildings having non-structural elements of brittle material attached to the structure the allowable inter story drift in ES EN 1998:2015 decreased by 50% from the EBCS 8 1995. For building having non-structural elements fixed in a way as not to interfere with structural deformation of brittle material attached to the structure, the allowable inter story drift decreased by 33.3%. For buildings having ductile non-structural elements the allowable inter story drift is mentioned in ES EN 1998:2015 but in EBCS 8 1995 was not specified.

## 4.11 Second order effect

Both EBCS 8 1995 and ES EN 1998:2015 have the same criteria for second order effect consideration except EBCS 8 1995 restrict seismicity coefficient  $\emptyset$  shall not exceed 0.25 but ES EN 1998:2015 allow it's up to 0.30 and ES EN 1998:2015 use stiffness reduction factor 0.5 for both beam and column.

## 4.12 Importance Category and Importance Factor

Buildings are classified in two codes in 4 importance classes, depending on the - consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse. Table 4.9 shows the classification and Importance Factor value of building based EBCS 8 1995 and ES EN 1998:2015.

Table 4.7 Important category classification and Impotence Factor value EBCS 8 1995 and ES EN 1998:2015

Building	Importance category EBCS 8 1995	Importance category ES EN 1998:2015	Importance Factor both EBCS 8 1995 & ES EN 1998:2015
Buildings of minor importance for public safety, e.g. agricultural buildings, etc.	IV	I	0.8
Ordinary buildings, not belonging in the other categories.	III	II	1.0
Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.	II	III	1.2
Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.	I	IV	1.4

According to importance category and importance factor the two codes are similar value the only difference is the rearrangement of the representation type of category.

## CHAPTER FIVE

### ANALYSIS AND RESULT OF CASE STUDY BUILDING

#### 5.1 Case study building

The selected building is G+4 school reinforced concrete building. It has a moment resisting frame structural system consisting of beams and columns with solid slab floor system. The building was designed according to EBCS-1995.

Because all the design drawing and specification are available, the reinforced concrete (RC) properties of structural members are assumed to be known completely.

The structure was constructed in Mekele city, Ethiopia, which is Seismic Zone 4. A design ground acceleration of 0.15g. The concrete class is C25 and the reinforcing steels class is S-300. The reinforced concrete school building has five stories, including ground floor. It has 17.5m height from ground level. Framing of the building is regular in plan and elevation where there are two bays in the X direction and thirteen bays the in Y direction. The floor plan is same for each story and slab thickness is 150 mm. The dead load for finishing material 1.73 kN/m<sup>2</sup> for all the floors. The live load is  $Q=3$  kN/m<sup>2</sup> for each floor except the top floor where the live load was considered 0.25 kN/m<sup>2</sup>.

Partition and parapet wall load was considered as the architectural plan area load on floor system and line load on beam.

The school building was analyzed in detail by performing both equivalent static and response spectrum analysis to compare seismic demand of EBCS 8 1995 and ES EN 1998:2015. Three-dimensional finite element model of the residential building was prepared in Integrated Building Design Software ETABS 2015 Version 15.0.0 shown in Figure 5.1.

The beam dimensions are 250x450mm for ground, 250x450mm and 250x650mm for typical floor, 250x400mm top tie beam. Column dimension is 500x300mm.

For this case study building Type 1 spectra was used to compare base shear story displacement and story drift of the two codes.

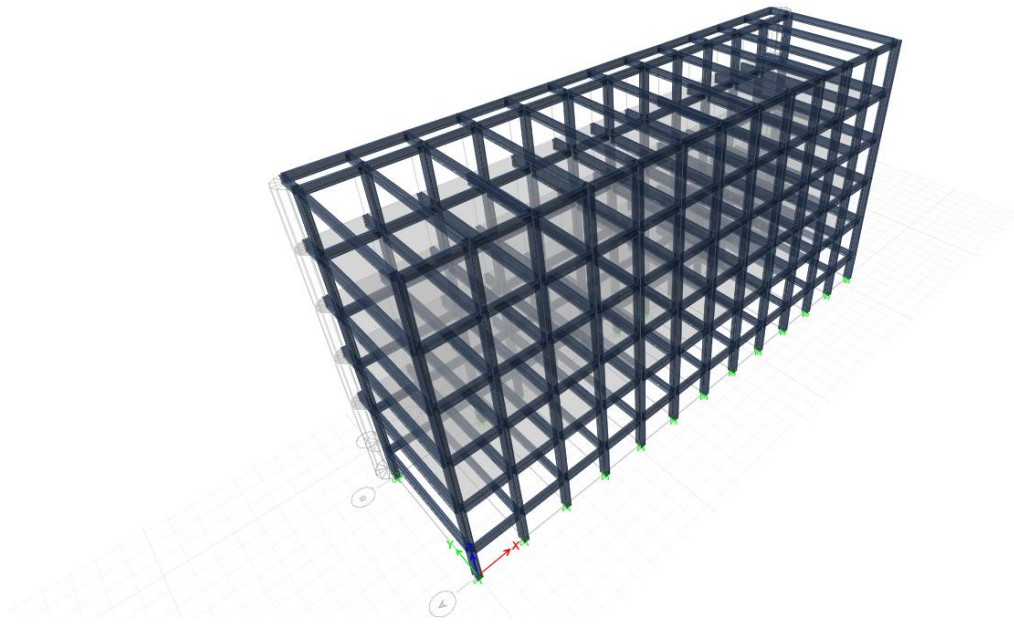


Figure 5.1 Three-dimensional finite element model of the school building

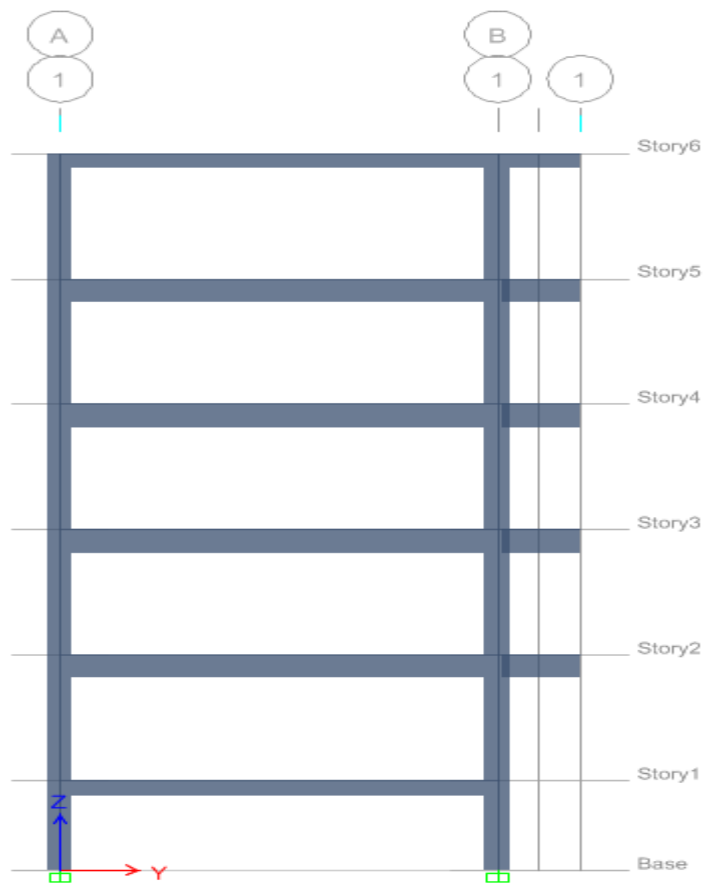


Figure 5.2 Elevation view

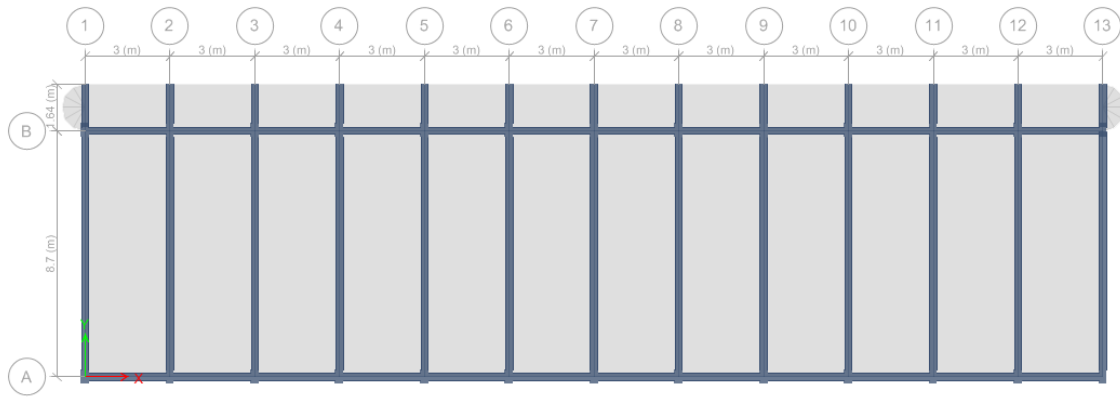


Figure 5.3 Typical floor plan view

## 5.2 Equivalent lateral force analysis

### 5.2.1 Equivalent lateral force analysis base shear comparison

As discussed in the above chapter the two codes have basic differences in ground type, peak ground acceleration and a formula to calculate base shear because of that, the result of base shear has significant differences as shown in the Table 5.1 and Figure 5.4.

Table 5.1 Equivalent lateral force analysis base shear force for case study building.

EBCS 8 1995		ES EN 1998 2015		Percent increased
Ground type	Base shear (kN)	Ground type	Base shear (kN)	
A	1205.8	A	1304.7	8.2%
		B	1957.0	62.3%
B	1447.0			34.7%
		C	2250.7	55.5%
C	1808.7			24.4%
		D	3123.3	72.7%
		E	2283.2	26.2%

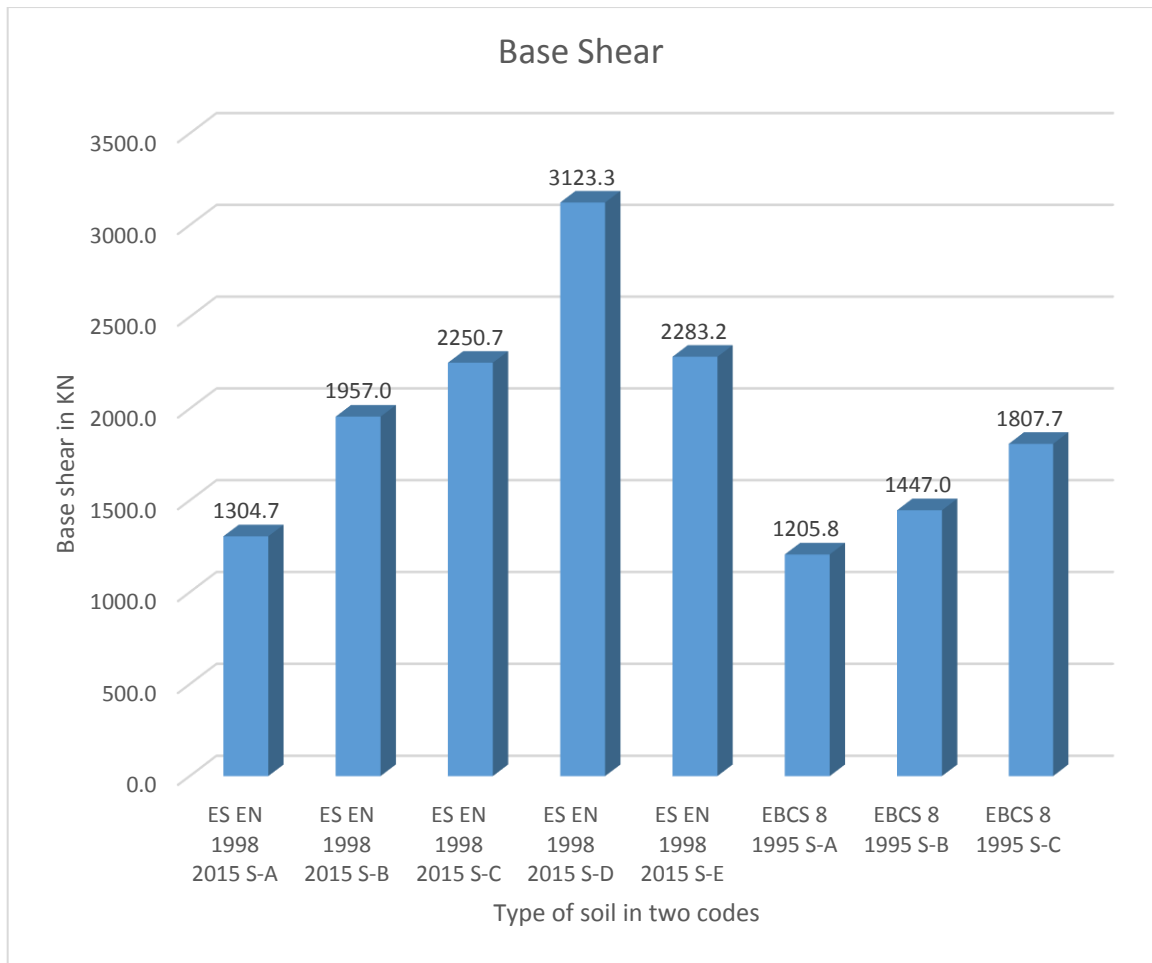


Figure 5.4 Equivalent lateral force analysis base shear of the two code

As shown in the Table 5.1 in ES EN 1998:2015 the value base shear increases 20.2% up to 91.9% in different soil types from EBCS 8 1995. It shows that the demand of earthquake resistance was increased in the new code. The value of base shear varies 1304.7 to 3123.3 kN which is 139.4% in ES EN 1998:2015, whereas it vary 1085.2 to 1627.8 kN which is a 50% increase, this figure shows the classification of ground type in ES EN 1998:2015 have great range but in old code it have low range.

### 5.2.2 Equivalent lateral force analysis story displacement comparison

The result of story displacement of the two codes is as shown in the Figure 5.5 and the Table 5.2 shows the top story displacement of building in two codes with different soil types

Table 5.2 Equivalent lateral force analysis top story displacement

EBCS 8 1995			ES EN 1998 2015			Percent increased	
Ground type	Top story displacement (mm)		Ground type	Top story displacement (mm)			
	X	Y		X	Y		
A	64.9	83.5	A	65.2	86.5	0.5%	3.6%
			B	104.5	125.5	61.0%	50.3%
B	77.9	98.2				34.1%	27.8%
			C	112.5	143.1	44.4%	45.7%
C	97.4	120.2				15.5%	19.1%
			D	166.8	195.2	71.2%	62.4%
			E	121.9	145.0	25.2%	20.6%

Since the base shear have a difference in the two codes it causes variations in story displacement. As shown in the above Table 5.2 top story displacement increases in X and Y direction by up to 71.2% and 62.4% respectively in new code.

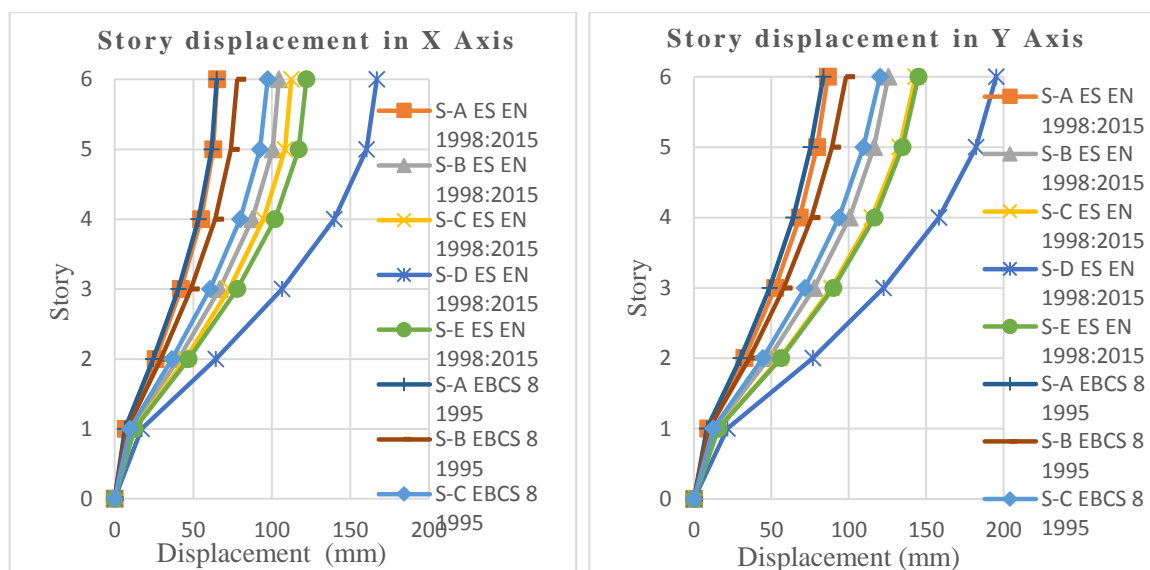


Figure 5.5 Equivalent lateral force analysis story displacement

### 5.2.3 Equivalent lateral force analysis story drift comparison

Table 5.3 Equivalent lateral force analysis maximum story drift

EBCS 8 1995			ES EN 1998:2015			Percent increased	
Ground type	Maximum story drift (10 <sup>-3</sup> )		Ground type	Maximum story drift (10 <sup>-3</sup> )		Percent increased	
	X	Y		X	Y	X	Y
A	5.1	6.2	A	5.3	6.6	3.9%	6.5%
			B	8.3	9.9	62.7%	59.7%
B	6.1	7.3	-----		36.1%	35.6%	
			C	9.2	11.3	50.8%	54.8%
C	7.6	9.1	-----		21.1%	24.2%	
			D	13.3	15.6	75.0%	71.4%
			E	9.7	11.5	27.6%	26.4%

Like base shear and story displacement the value story drift also increases 3.9% to 75.0% in ES EN 1998:2015 comparing with EBCS 8 1995. Table 5.3 shows the maximum story drift of case study building in different ground type.

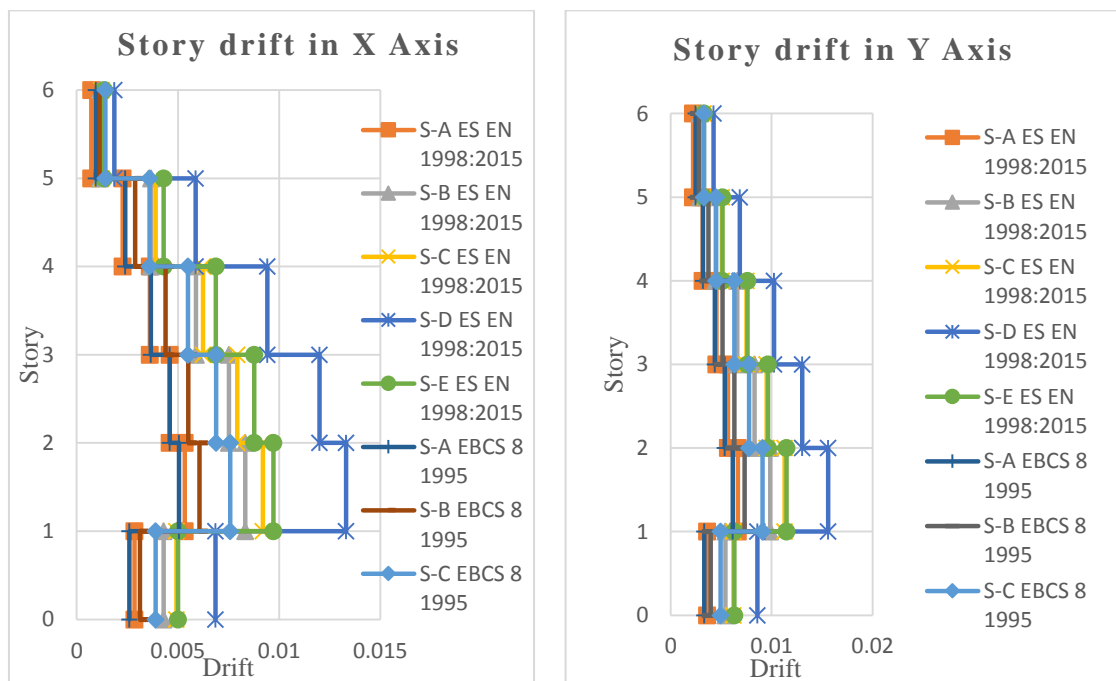


Figure 5.6 Equivalent lateral force analysis story displacement

## 5.3 Response spectrum analysis

### 5.3.1 Response spectrum analysis base shear comparison

Response spectrum analysis base shear result was less than from equivalent static analysis. Since the value base shear response spectrum analysis less than static analysis, it is increased by 90.0% in the case of EBCS 8 1995.

The demand of base shear was decreased by 58.1%, 20.6%, 25.8%, 7.89% and 38.9% on soil type A, B, C, D and E respectively in ES EN 1998:2015.

Table 5.4 Response spectrum analysis base shear result

EBCS 8 1995		ES EN 1998:2015		Percent increased
Ground type	Base shear (kN)	Ground type	Base shear (kN)	
A	1085.2	A	454.1	-58.1%
		B	861.5	20.6%
B	1302.3			-33.8%
		C	965.8	-25.8%
C	1627.8			-40.6%
		D	1500.4	-7.8%
		E	994.1	-38.9%

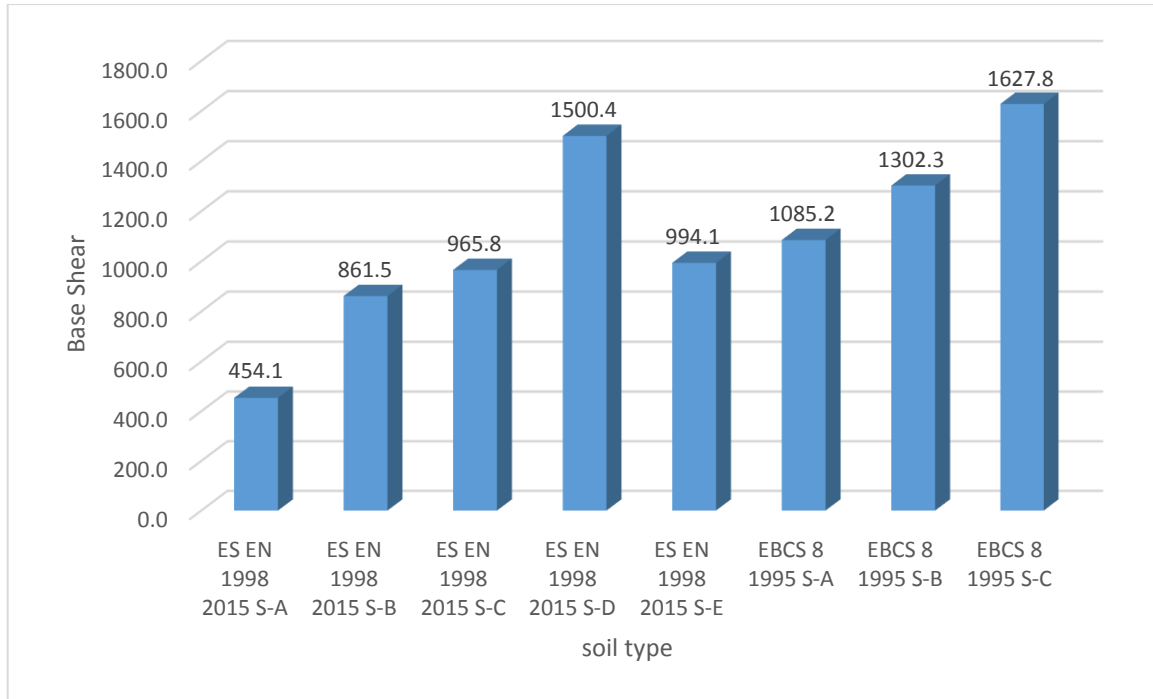


Figure 5.7 Response spectrum analysis base shear

### 5.3.2 Response spectrum analysis story displacement comparison

Table 5 5 Response spectrum analysis top story displacement result

EBCS 8 1995			ES EN 1998:2015			Percent increased			
Ground type	Top story displacement (mm)		Ground type	Top story displacement (mm)				X	Y
	X	Y		X	Y				
A	40.4	49.2	A	21.9	31.3	-45.8%	-36.4%		
			B	41.1	52.0	1.7%	5.7%		
B	59.9	67.2				-31.4%	-22.6%		
			C	47.6	57.7	-20.5%	-14.1%		
C	90.2	95.8				-47.2%	-39.8%		
			D	74.4	85.9	-17.5%	-10.3%		
			E	48.7	58.8	-46.0%	-38.6%		

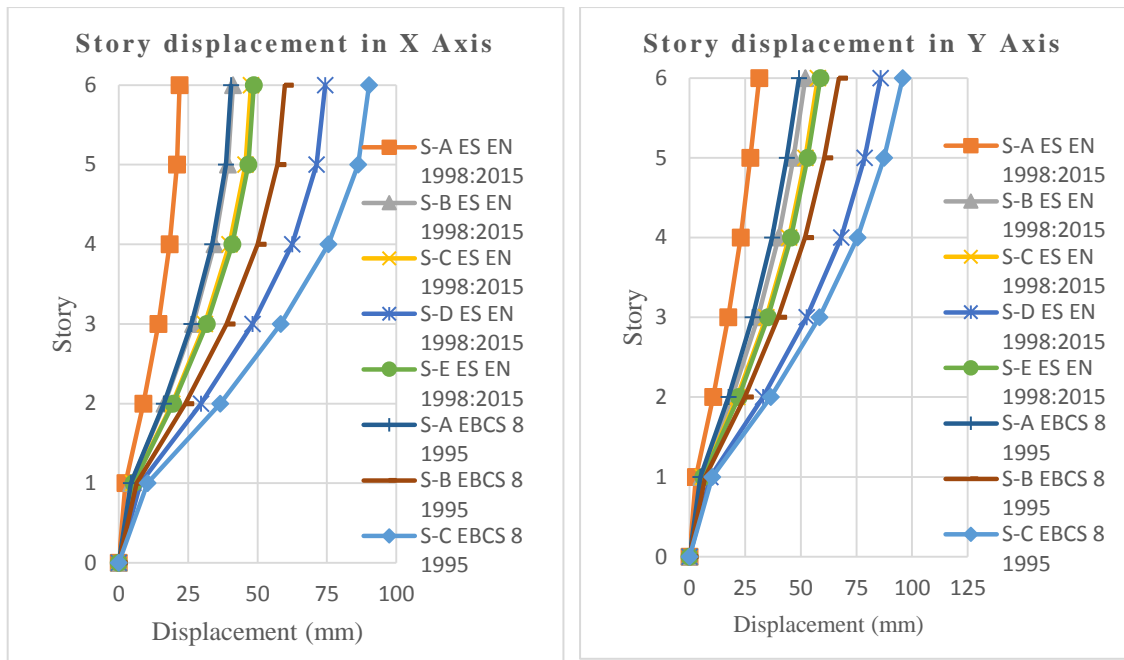


Figure 5.8 Response spectrum analysis displacement result graph

### 5.3.3 Response spectrum analysis story drift comparison

Table 5.6 Response spectrum analysis maximum story drift

EBCS 8 1995			ES EN 1998:2015			Percent increased	
Ground type	Maximum story drift ( $10^{-3}$ )		Ground type	Maximum story drift ( $10^{-3}$ )			
	X	Y		X	Y		
A	3.3	3.6	A	1.8	2.2	-45.4%	-38.8%
			B	3.4	3.9	3.0%	8.3%
B	4.9	5.1				-30.6%	-30.7%
			C	3.9	4.4	-20.4%	-13.7%
C	7.3	7.5				-46.5%	-41.3%
			D	6.1	6.8	-16.4%	-9.3%
			E	4.0	4.5	-45.2%	40.0%

The result shows the demand of drift become smaller in ES EN 1998:2015 with similar soil property.

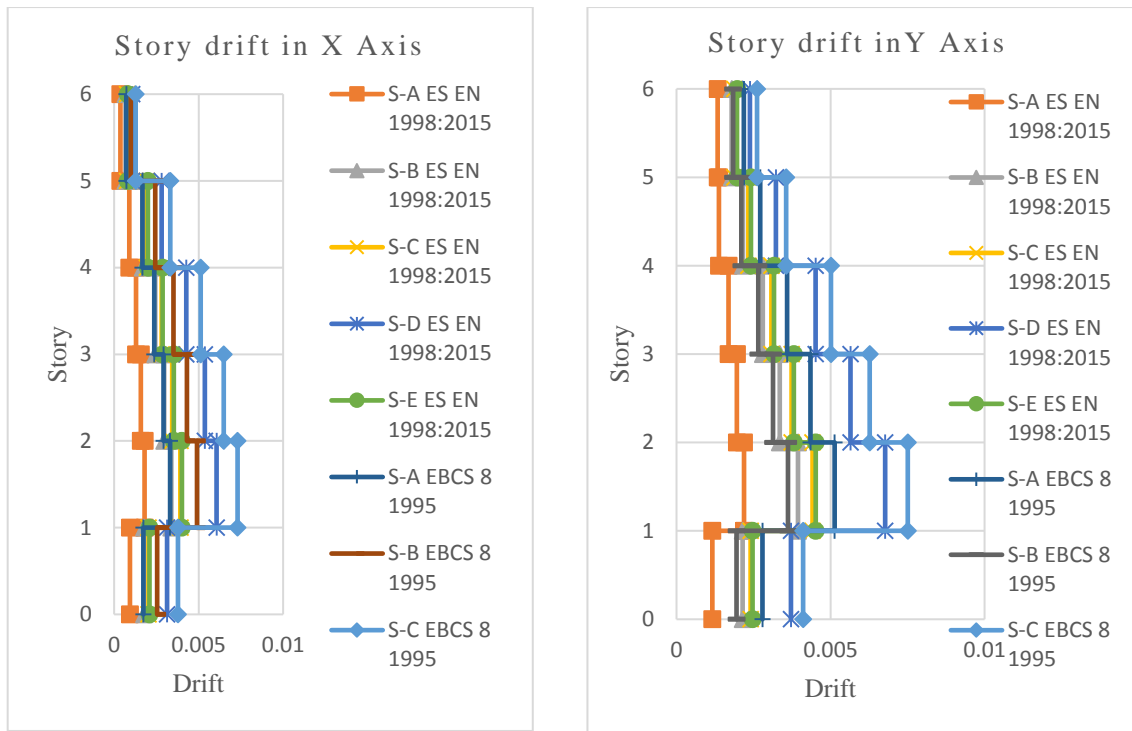


Figure 5.9 Response spectrum analysis story drift result graph

Generally, in equivalent lateral force analysis significant differences in base shear, story displacement and story drift of the structure.

## CHAPTER SIX

### SAFETY COMPROMISING OF A CASE STUDY BUILDING

Seismic demand of the building is increased because of changes in ground conditions, seismic zone and other criteria in ES EN 1998:2015. This compromise the safety of the structure.

#### 6.1 Safety compromising based equivalent lateral force analysis

##### 6.1.1 Equivalent lateral force analysis demand to capacity ratio of column

As a measure of the load condition of the column, a capacity ratio is calculated. The capacity ratio is a factor that gives an indication of the load condition of the column with respect to the load capacity of the column. Column capacity ratio factor was calculated based on the ratio of required reinforcement to available reinforcement.

Table 6.1 Equivalent lateral force analysis column on Axis A and Axis 2 demand to capacity ratio

Story	A*	B*	C*	A**	B**	C**	D**	E**
Story 1	0.539	0.604	0.700	0.562	0.791	0.899	1.292	0.913
Story 2	0.584	0.668	0.798	0.616	0.878	0.972	1.379	1.005
Story 3	0.715	0.798	0.945	0.740	1.086	1.214	1.721	1.259
Story 4	0.725	0.838	1.009	0.780	1.130	1.261	1.881	1.308
Story 5	0.625	0.690	0.798	0.590	0.805	0.895	1.237	0.924
Story 6	0.591	0.606	0.647	0.582	0.587	0.582	0.587	0.587

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.2 Percentage increase in demand to capacity ratio of column on Axis A and Axis 2 demand to capacity ratio

Story	A* to A** (%)	A* to B** (%)	B* to C** (%)	C* to D** (%)	C* to E** (%)
Story 1	4.3	46.8	48.8	84.6	30.4
Story 2	5.5	50.3	45.5	72.8	25.9
Story 3	3.5	51.9	52.1	82.1	33.2
Story 4	7.6	55.9	50.5	86.4	29.6
Story 5	-5.6	28.8	29.7	55.0	15.8
Story 6	-1.5	-0.7	-4.0	-9.3	-9.3

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.3 Equivalent lateral force analysis column on Axis B and Axis 5 demand to capacity ratio

Story	A*	B*	C*	A**	B**	C**	D**	E**
Story 1	0.677	0.735	0.822	0.661	0.844	0.922	1.209	0.935
Story 2	0.690	0.755	0.852	0.685	0.919	1.012	1.353	1.040
Story 3	0.819	0.900	1.019	0.825	1.075	1.197	1.638	1.209
Story 4	0.796	0.874	1.028	0.807	1.130	1.280	1.736	1.290
Story 5	0.741	0.799	0.890	0.708	0.880	0.966	1.233	0.976
Story 6	0.960	0.960	0.960	0.933	0.973	0.933	0.973	0.973

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.4 Percentage increase in demand to capacity ratio of column on Axis B and Axis 5 demand to capacity ratio

Story	A* to A** (%)	A* to B** (%)	B* to C** (%)	C* to D** (%)	C* to E** (%)
Story 1	-2.4	24.7	25.4	47.1	13.7
Story 2	-0.7	33.2	34.0	58.8	22.1
Story 3	0.7	31.3	33.0	60.7	18.6
Story 4	1.4	42.0	46.5	68.9	25.5
Story 5	-4.5	18.8	20.9	38.5	9.7
Story 6	-2.8	1.4	-2.8	1.4	1.4

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.5 Equivalent lateral force analysis column on Axis A and Axis 8 demand to capacity ratio

Story	A*	B*	C*	A**	B**	C**	D**	E**
Story 1	0.591	0.647	0.731	0.598	0.772	0.861	1.137	0.872
Story 2	0.646	0.708	0.828	0.660	0.899	0.986	1.328	1.018
Story 3	0.825	0.905	1.024	0.840	1.127	1.243	1.693	1.283
Story 4	0.828	0.936	1.009	0.868	1.205	1.328	1.782	1.366
Story 5	0.856	0.876	0.956	0.861	0.945	1.015	1.318	1.038
Story 6	0.996	0.996	0.996	1.000	1.007	1.000	1.007	1.007

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.6 Percentage increase in demand to capacity ratio of column on Axis A and Axis 8 demand to capacity ratio

Story	A* to A** (%)	A* to B** (%)	B* to C** (%)	C* to D** (%)	C* to E** (%)
Story 1	1.2	30.6	33.1	55.5	19.3
Story 2	2.2	39.2	39.3	60.4	22.9
Story 3	1.8	36.6	37.3	65.3	25.3
Story 4	4.8	45.5	41.9	76.6	35.4
Story 5	0.6	10.4	15.9	37.9	8.6
Story 6	0.4	1.1	0.4	1.1	1.1

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.7 Equivalent lateral force analysis column on Axis B and Axis 8 demand to capacity ratio

Story	A*	B*	C*	A**	B**	C**	D**	E**
Story 1	0.684	0.737	0.821	0.672	0.838	0.915	1.179	0.925
Story 2	0.716	0.781	0.879	0.706	0.927	1.021	1.360	1.047
Story 3	0.860	0.942	1.063	0.875	1.120	1.239	1.665	1.249
Story 4	0.829	0.909	1.051	0.841	1.164	1.321	1.749	1.328
Story 5	0.810	0.844	0.931	0.815	0.917	1.013	1.268	1.014
Story 6	1.068	1.068	1.068	1.075	1.084	1.075	1.084	1.084

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.8 Percentage increase in demand to capacity ratio of column on Axis B and Axis 8 demand to capacity ratio

Story	A* to A** (%)	A* to B** (%)	B* to C** (%)	C* to D** (%)	C* to E** (%)
Story 1	-1.8	22.5	24.2	43.6	12.7
Story 2	-1.4	29.5	30.7	54.7	19.1
Story 3	1.7	30.2	31.5	56.6	17.5
Story 4	1.4	40.4	45.3	66.4	26.4
Story 5	0.6	13.2	20.0	36.2	8.9
Story 6	0.7	1.5	0.7	1.5	1.5

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

The demand capacity ratio of most column based on EBCS 8 1995 was less than one in all types of soil. Most column fail in soil type C, D and E based on ES EN 1998:2015. The demand to capacity ratio of column increases up to 86.4% in some column with similar soil property. This percent shows the how much level of safety compromising of column sections.

### 6.1.2 Equivalent lateral force analysis demand to capacity ratio of beam

The demand to capacity ratio beam was done based on ratio analysis moment to moment capacity moment of the beam. The capacity of the beam was calculated based on euro code by considering the section of the beam area of the bottom and top reinforcement.

Calculated moment capacity divided by the analytic moment gives demand to capacity ratio of the beam. The shear capacity of the beam was not considered in this demand to capacity ratio calculation.

Table 6.9 Equivalent lateral force analysis beam on Axis 8 between Axis A to B at story 2 demand to capacity ratio

Location	A*	B*	C*	A**	B**	C**	D**	E**
Support L	0.834	0.913	1.032	0.807	1.010	1.107	1.379	1.113
Mid span	0.898	0.898	0.946	0.951	0.959	0.951	1.075	0.959
Support R	0.935	1.014	1.133	0.889	1.090	1.189	1.460	1.195

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.10 Percentage increase in demand to capacity ratio of beam on Axis 8 between Axis A to B at story 2 demand to capacity ratio

Location	A* to A** (%)	A* to B** (%)	B* to C** (%)	C* to D** (%)	C* to E** (%)
Support L	-3.2	21.1	21.2	33.6	7.8
Mid span	5.9	6.8	5.9	13.7	1.4
Support R	-4.9	16.6	17.2	28.9	5.5

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.11 Equivalent lateral force analysis beam on Axis 12 between Axis A to B at story 4 demand to capacity ratio

Location	A*	B*	C*	A**	B**	C**	D**	E**
Support L	0.564	0.616	0.666	0.510	0.637	0.709	1.010	0.707
Mid span	0.617	0.617	0.534	0.615	0.620	0.615	0.712	0.620
Support R	0.680	0.732	0.778	0.603	0.732	0.802	0.977	0.801

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.12 Percentage increase in demand to capacity ratio of beam on Axis 12 between Axis A to B at story 4 demand to capacity ratio

Location	A* to A** (%)	A* to B** (%)	B* to C** (%)	C* to D** (%)	C* to E** (%)
Support L	-9.7	13.0	15.0	51.7	6.2
Mid span	-0.4	0.4	-0.4	33.4	16.1
Support R	-11.3	7.7	9.5	25.7	3.0

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.13 Equivalent lateral force analysis beam on Axis A between Axis 6 to 7 at story 1 demand to capacity ratio

Location	A*	B*	C*	A**	B**	C**	D**	E**
Support L	0.923	1.089	1.337	0.802	1.461	1.453	2.276	1.689
Mid span	0.668	0.788	0.964	0.720	1.049	1.184	1.627	1.212
Support R	0.926	1.092	1.341	0.997	1.464	1.650	2.279	1.693

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.14 Percentage increase in demand to capacity ratio of beam on Axis A between Axis 6 to 7 at story 1 demand to capacity ratio

Location	A* to A** (%)	A* to B** (%)	B* to C** (%)	C* to D** (%)	C* to E** (%)
Support L	-13.0	58.4	33.4	70.2	26.3
Mid span	7.8	57.0	50.3	68.8	25.8
Support R	7.7	58.2	51.1	69.9	26.2

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

As shown in the above tables in some beam demand to capacity ratio increase up to 70.2% in ES EN 1998:2015.

## 6.2 Safety compromising based response spectrum analysis

### 6.2.1 Response spectrum analysis demand to capacity ratio of column

As discussed in the chapter five the seismic demand in response spectrum analysis was less than equivalent static analysis in a given building. Most of the column have demand to capacity ratio less than one.

Table 6.15 Response spectrum analysis column on Axis A and Axis 2 demand to capacity ratio

Story	A*	B*	C*	A**	B**	C**	D**	E**
Story 1	0.418	0.501	0.632	0.386	0.402	0.435	0.589	0.441
Story 2	0.459	0.556	0.743	0.418	0.440	0.477	0.650	0.484
Story 3	0.567	0.692	0.910	0.525	0.540	0.587	0.813	0.596
Story 4	0.552	0.712	0.956	0.479	0.545	0.608	0.857	0.621
Story 5	0.550	0.642	0.753	0.533	0.533	0.533	0.659	0.545
Story 6	0.600	0.600	0.613	0.648	0.648	0.648	0.634	0.648

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.16 Percentage increase in demand to capacity ratio of column on Axis A and Axis 2 demand to capacity ratio

Story	A* to A** (%)	A* to B** (%)	B* to C** (%)	C* to D** (%)	C* to E** (%)
Story 1	-7.7	-3.8	-13.2	-6.8	-30.2
Story 2	-8.9	-4.1	-14.2	-12.5	-34.9
Story 3	-7.4	-4.8	-15.2	-10.7	-34.5
Story 4	-13.2	-1.3	-14.6	-10.4	-35.0
Story 5	-3.1	-3.1	-17.0	-12.5	-27.6
Story 6	8.0	8.0	8.0	3.4	5.7

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.17 Response spectrum analysis column on Axis B and Axis 5 demand to capacity ratio

Story	A*	B*	C*	A**	B**	C**	D**	E**
Story 1	0.627	0.627	0.789	0.631	0.631	0.631	0.696	0.631
Story 2	0.636	0.636	0.848	0.639	0.639	0.639	0.749	0.639
Story 3	0.773	0.773	1.014	0.776	0.776	0.776	0.903	0.776
Story 4	0.692	0.692	1.011	0.694	0.694	0.700	0.920	0.709
Story 5	0.723	0.723	0.878	0.737	0.737	0.737	0.790	0.737
Story 6	0.968	0.968	0.968	1.026	1.026	1.026	1.015	1.026

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.18 Percentage increase in demand to capacity ratio of column on Axis B and Axis 5 demand to capacity ratio

Story	A* to A** (%)	A* to B** (%)	B* to C** (%)	C* to D** (%)	C* to E** (%)
Story 1	0.6	0.6	0.6	-11.8	-20.0
Story 2	0.5	0.5	0.5	-11.7	-24.6
Story 3	0.4	0.4	0.4	-10.9	-23.5
Story 4	0.3	0.3	1.2	-9.0	-29.9
Story 5	1.9	1.9	1.9	-10.0	-16.1
Story 6	6.0	6.0	6.0	4.9	6.0

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.19 Response spectrum analysis column on Axis A and Axis 8 demand to capacity ratio

Story	A*	B*	C*	A**	B**	C**	D**	E**
Story 1	0.522	0.585	0.717	0.525	0.525	0.525	0.636	0.525
Story 2	0.574	0.617	0.761	0.578	0.578	0.578	0.684	0.578
Story 3	0.749	0.779	0.955	0.754	0.754	0.754	0.863	0.754
Story 4	0.720	0.792	0.996	0.728	0.728	0.728	0.910	0.728
Story 5	0.855	0.864	0.930	0.882	0.882	0.882	0.878	0.882
Story 6	0.987	0.987	0.987	1.052	1.052	1.052	1.039	1.052

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.20 Percentage increase in demand to capacity ratio of column on Axis A and Axis 8 demand to capacity ratio

Story	A* to A** (%)	A* to B** (%)	B* to C** (%)	C* to D** (%)	C* to E** (%)
Story 1	0.6	0.6	-10.3	-11.3	-26.8
Story 2	0.7	0.7	-6.3	-10.1	-24.0
Story 3	0.7	0.7	-3.2	-9.6	-21.0
Story 4	1.1	1.1	-8.1	-8.6	-26.9
Story 5	3.2	3.2	2.1	-5.6	-5.2
Story 6	6.6	6.6	6.6	5.3	6.6

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.21 Response spectrum analysis column on Axis B and Axis 8 demand to capacity ratio

Story	A*	B*	C*	A**	B**	C**	D**	E**
Story 1	0.661	0.678	0.806	0.668	0.666	0.666	0.704	0.666
Story 2	0.686	0.709	0.866	0.691	0.691	0.691	0.761	0.691
Story 3	0.849	0.849	1.038	0.856	0.856	0.856	0.930	0.856
Story 4	0.760	0.824	1.009	0.768	0.768	0.768	0.922	0.768
Story 5	0.809	0.812	0.908	0.826	0.826	0.826	0.824	0.826
Story 6	1.077	1.077	1.077	1.148	1.148	1.148	1.135	1.148

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.22 Percentage increase in demand to capacity ratio of column on Axis B and Axis 8 demand to capacity ratio

Story	A* to A** (%)	A* to B** (%)	B* to C** (%)	C* to D** (%)	C* to E** (%)
Story 1	1.1	0.8	-1.8	-12.7	-17.4
Story 2	0.7	0.7	-2.5	-12.1	-20.2
Story 3	0.8	0.8	0.8	-10.4	-17.5
Story 4	1.1	1.1	-6.8	-8.6	-23.9
Story 5	2.1	2.1	1.7	-9.3	-9.0
Story 6	6.6	6.6	6.6	5.4	6.6

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

EBCS 8 1995 allow scaling of base shear on response spectrum analysis when base shear is less than from equivalent static analysis but ES EN 1998:2015 not scaled the value of base shear. This leads most elements demand to capacity ratio decreases in ES EN 1998:2015 when compare to EBCS 8 1995.

## 6.2.2 Response spectrum analysis demand to capacity ratio of beam

Table 6.23 Response spectrum analysis beam on Axis 8 between Axis A to B at story 2 demand to capacity ratio

Location	A*	B*	C*	A**	B**	C**	D**	E**
Support L	0.610	0.648	0.743	0.619	0.646	0.679	0.845	0.686
Mid span	0.947	0.947	0.947	0.960	0.960	0.960	0.960	0.960
Support R	0.745	0.749	0.844	0.756	0.756	0.760	0.926	0.767

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.24 Percentage increase in demand to capacity ratio of beam on Axis 8 between Axis A to B at story 2 demand to capacity ratio

Location	A* to A** (%)	A* to B** (%)	B* to C** (%)	C* to D** (%)	C* to E** (%)
Support L	-12.2	-8.4	-18.2	-15.5	-31.5
Mid span	1.4	1.4	1.4	1.4	1.4
Support R	-6.4	-6.4	-17.5	-15.9	-30.4

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.25 Response spectrum analysis beam on Axis 12 between Axis A to B at story 4 demand to capacity ratio

Location	A*	B*	C*	A**	B**	C**	D**	E**
Support L	0.403	0.417	0.467	0.404	0.404	0.412	0.499	0.420
Mid span	0.617	0.617	0.617	0.620	0.620	0.620	0.612	0.620
Support R	0.558	0.558	0.583	0.561	0.561	0.561	0.593	0.561

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.26 Percentage increase in demand to capacity ratio of beam on Axis 12 between Axis A to B at story 4 demand to capacity ratio

Location	A* to A** (%)	A* to B** (%)	B* to C** (%)	C* to D** (%)	C* to E** (%)
Support L	-10.9	-10.9	-20.6	-18.8	-31.7
Mid span	0.4	0.4	0.4	-0.9	0.4
Support R	-1.6	-1.6	-11.8	-18.9	-23.3

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.27 Response spectrum analysis beam on Axis A between Axis 6 to 7 at story 1 demand to capacity ratio

Location	A*	B*	C*	A**	B**	C**	D**	E**
Support L	0.356	0.482	0.670	0.367	0.605	0.686	1.005	0.700
Mid span	0.267	0.355	0.490	0.275	0.443	0.500	0.728	0.510
Support R	0.359	0.484	0.673	0.369	0.608	0.689	1.008	0.702

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Table 6.28 Percentage increase in demand to capacity ratio of beam on Axis A between Axis 6 to 7 at story 1 demand to capacity ratio

Location	A* to A** (%)	A* to B** (%)	B* to C** (%)	C* to D** (%)	C* to E** (%)
Support L	-38.1	2.1	6.5	-15.4	-41.1
Mid span	2.9	66.0	40.9	48.7	4.2
Support R	-38.0	2.0	-17.4	-15.2	-41.0

\* Soil Type based on EBCS 8 1995

\*\* Soil Type based on ES EN 1998:2015

Similarly, like column in response spectrum analysis the beam demand to capacity ratio decreases.

## 6.3 Pushover analysis

### 6.3.1 Pushover analysis result and discussion

Analyzed have been performed using Structural Analysis Program SAP2000 nonlinear version 15.0.0. A two-dimensional model of structure was created in Structural Analysis Program SAP2000 to carry out nonlinear static analysis. Beam and column elements were modeled as nonlinear frame elements by defining plastic hinges at both ends of the beams and columns. For beam user define hinge property was used, by calculating moment curvature for each section of the beam using Response2000. For column SAP2000 default hinge property accordingly FEMA 356 [13] was used. The values assigned to each of these points vary depending on the type of element, material properties, longitudinal and transverse steel content. Figure 6.3 shows base shear versus displacement of the structure.

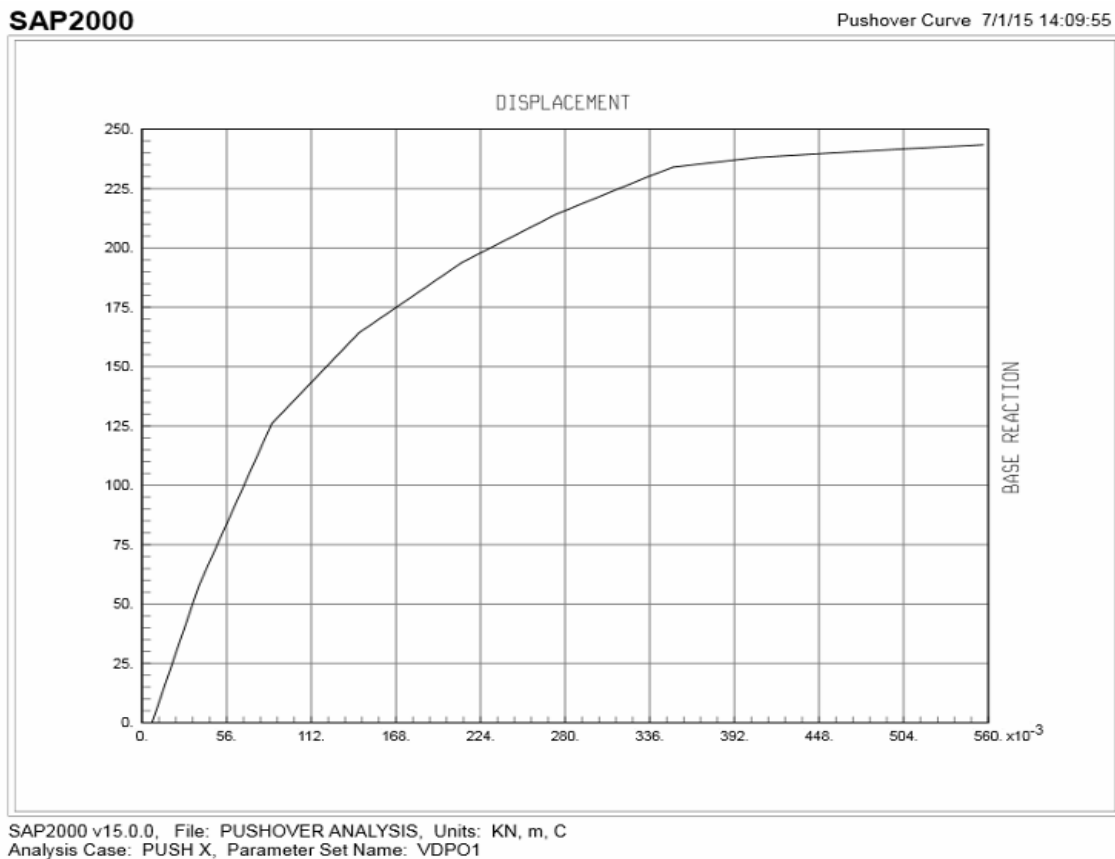


Figure 6.1 Pushover curve



Figure 6.2 Capacity curve

Figure 6.1 and Figure 6.2 shows the performance the structure and capacity curve of structure respectively.

### 6.3.2 Global performance of the structure based on story drift

As shown in the Table 6.29 the structure is not satisfy global performance drift requirement based on FEMA 356 [13]. Since the building is school building it have to be safe for life safety level.

Table 6.29 Pushover analysis story drift.

Story	Drift push X	IO <0.01	LS [0.01 - 0.02]	CP >0.02
		X	X	X
Story 1	0.011	Not ok	Ok	Ok
Story 2	0.020	Not ok	Ok	Ok
Story 3	0.022	Not ok	Not ok	Ok
Story 4	0.020	Not ok	Ok	Ok
Story 5	0.018	Not ok	Ok	Ok
Story 6	0.015	Not ok	Ok	Ok

### 6.3.3 Level of hinge formation of structure at performance point

The performance point of seismic load was reached at Step 5. The level hinges of the members were as shown in the Table 6.31 and Figure 6.3.

Table 6.30 Hinge status for different Step for pushover load on X direction

Pushover Curve - PUSH X											
Step	Displacemer m	BaseForce KN	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
0	0.006614	0.000	41	0	0	0	0	0	0	0	41
1	0.036614	55.809	41	0	0	0	0	0	0	0	41
2	0.037874	58.154	40	1	0	0	0	0	0	0	41
3	0.081557	121.488	38	3	0	0	0	0	0	0	41
4	0.115757	147.925	37	3	1	0	0	0	0	0	41
5	0.173361	177.618	35	2	4	0	0	0	0	0	41
6	0.206889	191.799	34	3	2	2	0	0	0	0	41
7	0.242144	203.850	32	4	3	2	0	0	0	0	41
8	0.272144	213.569	32	4	2	2	0	1	0	0	41
9	0.306614	222.969	31	4	2	3	0	1	0	0	41

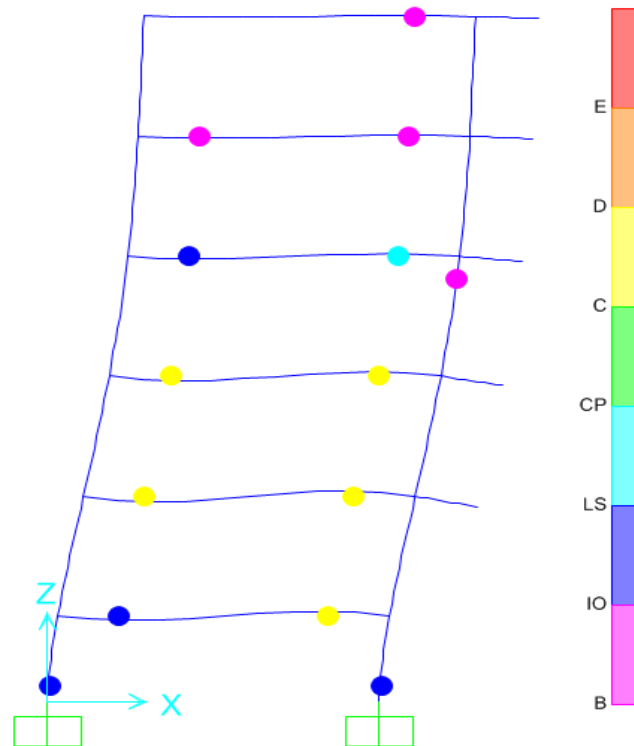


Figure 6.3 Hinge property at Step 5

Table 6.31 shows there is 1 hinge between immediate occupancy and life safety level from 41 hinges due to pushover load. This level shows the structure was not safe locally for the seismic forces.

Generally the building safety was significantly compromised for Soil Type C and D. For Soil Type A building safety was not considerably compromised. For Soil Type A beams and columns elements only compromise less than 10% in equivalent lateral force analysis.

For Soil Type C and D the level of safety of beam and column elements compromised by up to 86%. Similarly, in Soil Type D the safety of beam and column was reduced by up to 35%.

# CHAPTER SEVEN

## CONCLUSIONS AND RECOMMENDATIONS

### 6.1 Conclusions

The revisions of seismic codes cause differences in the parameter, analysis method and seismic demand of the structure. The following conclusion points are drawn after completion of the study.

In ES EN 1998:2015 soil is classified into five main category in addition, there is two special categories of soil. EBCS 8 1995 only classified soil into three, its few to appropriate to cover all types of soil. EBCS 8 1995 site classification system is based on only a representative average shear wave velocity. ES EN 1998:2015 use standard penetration test blow-count and unconfined compression strength in addition to representative average shear wave velocity.

The value of the normalized elastic response spectrum is not greater than 2.5 in EBCS 8 1995 but it increases up to 3.5 in ES EN 1998:2015 within a similar soil property.

Peak ground acceleration is increased in ES EN 1998:2015 by 33.3% to 50.0% on different seismic zone. For some city which its seismic hazard map was changed, Peak ground acceleration is double from EBCS 8 1995.

Base shear of given building has great difference in the two codes. Base shear value of ES EN 1998:2015 have a difference up to 72.7% in equivalent lateral force analysis with similar soil property.

There is significance displacement demand variation in the two codes in both static and dynamic analysis. In Soil Type B, C and D according to ES EN 1998:2015 have great displacement variation with similar soil property. For Soil Type A based new code doesn't have significant differences in displacement. Soil Type E has more than 20% difference from soil with similar property.

EBCS 8 1995 gives minimum drift value compared to ES EN 1998:2015 in both X and Y direction. The difference is higher with Soil Type B, C and D but less significant differences in Soil Type A and D accordingly soil classification of ES EN 1998:2015.

The value of demand to capacity ratio increase ES EN 1998:2015 compared to EBCS 8 1995. It shows the safety compromising of building. The value has great significant differences on Soil Type B, C and D.

Pushover analysis shows the structure is globally safe for Soil Type A and B for seismic load in X and Y direction. But there are local hinges below life safety level of Soil Type B for seismic load in Y direction. On Soil Type C it can resist seismic load in the X direction, but can't resist seismic load in the Y direction. For the remaining Soil Type the structure can't resist the seismic demand.

Generally the seismic provisions of two codes have differences, this leads to increase seismic demand such that base shear, story displacement, story drift and stresses of structure. Due to this significant difference in seismic demand considerable safety compromise is occurring on existing structure.

## **6.2 Recommendations**

Further study is required for the detailing part of EBCS 8 1995 and ES EN 1998:2015.

Further study of seismic demand variation in the two code and performance evaluation is required for building constructed on seismic zone 3, 2 and 1.

Performance of important existing structure should be checked based on ES EN 1998:2015 and give appropriate measurement.

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## APPENDIX-A: Verification of SAP2000

Three story frame taken from *Ahmet Yakut (2001)* from his numerical study. It is going to be analyzed using manual calculation of nonlinear static pushover analysis in which an incremental load is applied to the structure at each story and the corresponding internal moments are recorded and the load is applied until hinge is formed and system reached a plastic mechanism.

Both Column has 60cmx60cm a size 10 number of 10mm diameter bar and both beam has 50cmx25cm size and 3 number of 10mm diameter bar both at top and bottom. Fcd 25 concrete and Fyd -495 steel is used and concrete cover of 50mm is used

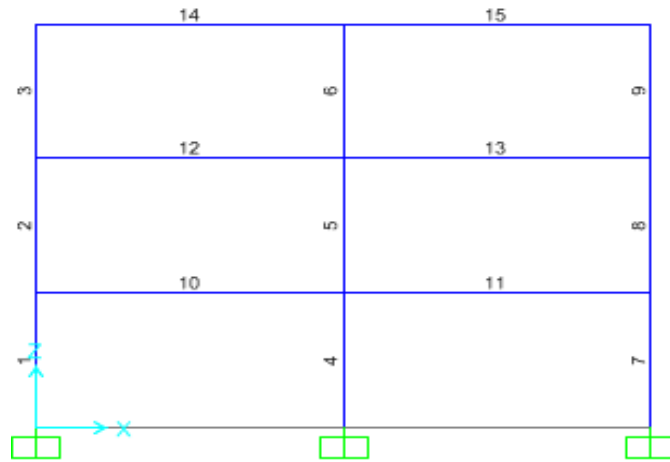


Figure A.1 Capacity Curve For Incremental and SAP2000

Table A.1 Moment Curvature Relation for beams and columns on the sample frame

Member	Element Type	N(kN)	My (kNm)	$\Phi_y$ (rad/m)	$\Phi_y$ (rad/m)
1	Column	-83.786	124.0	0.0055	0.111
2	Column	-51.347	115.5	0.0056	0.115
3	Column	-19.872	107.5	0.0056	0.119
4	Column	-253.392	166.0	0.0059	0.085
5	Column	-158.905	143.0	0.0060	0.099
6	Column	-64.797	119.0	0.0060	0.113
7	Column	-124.104	133.5	0.0056	0.105
8	Column	-77.747	122.0	0.0057	0.112
9	Column	-31.201	110.0	0.0054	0.118
10	Beam	5.606	49.0	0.0073	0.103
11	Beam	1.421	50.0	0.0069	0.102
12	Beam	-17.233	53.0	0.0069	0.099
13	Beam	5.606	49.0	0.0073	0.103
14	Beam	1.421	50.0	0.0069	0.102
15	Beam	-17.233	53.0	0.0069	0.099

As shown in the Figure 6.2 the capacity curve for both manual and pushover approach have similar pattern but in initial stage incremental approach have high base shear and become similar around performance point.

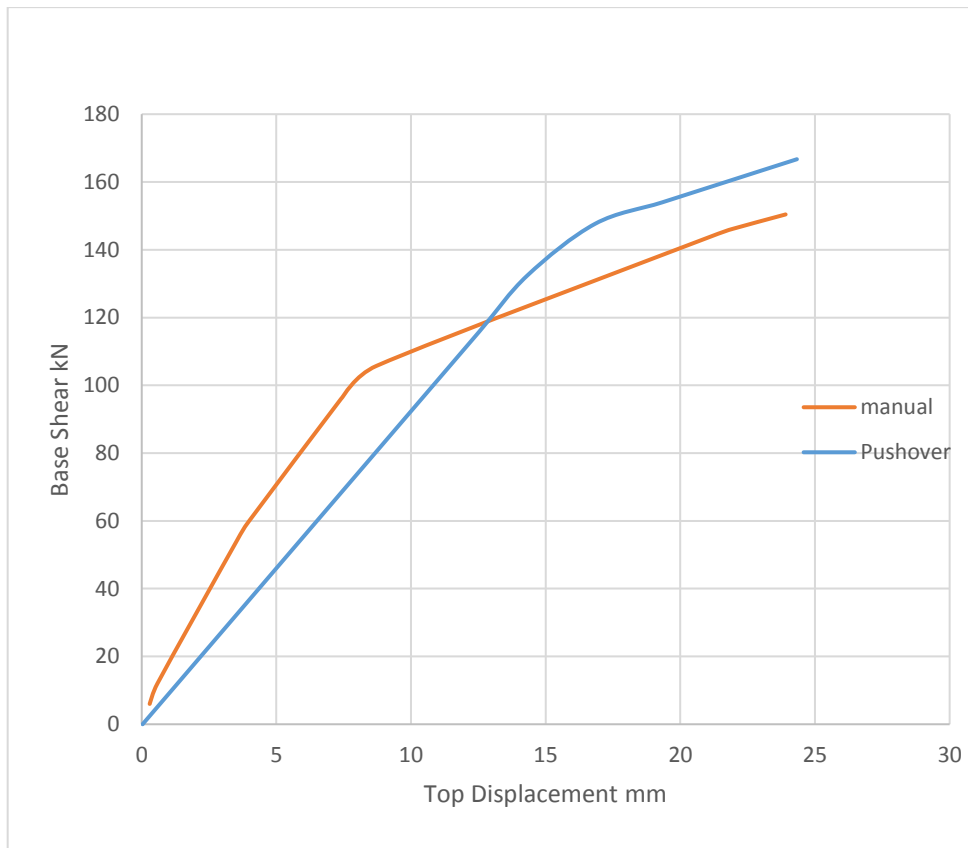


Figure A 2 Capacity curve for incremental and SAP2000

## APPENDIX-B Progress hinge formation pushover load

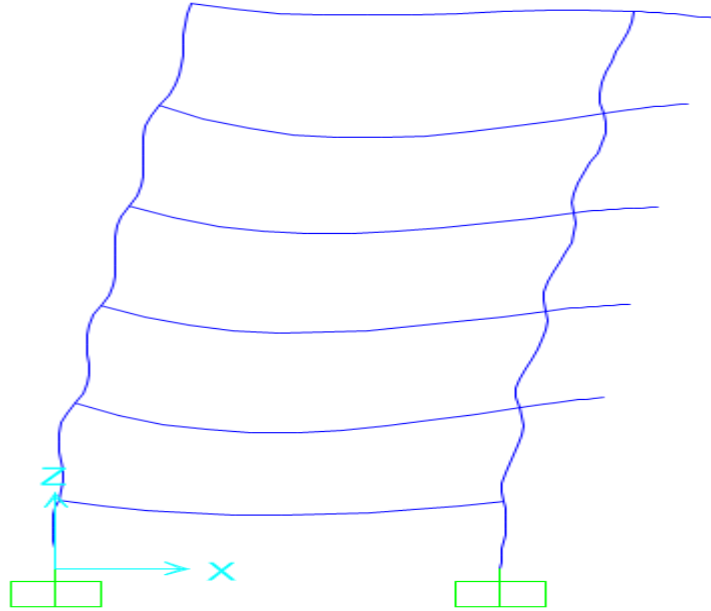


Figure B.1 Hinge property at Step 0

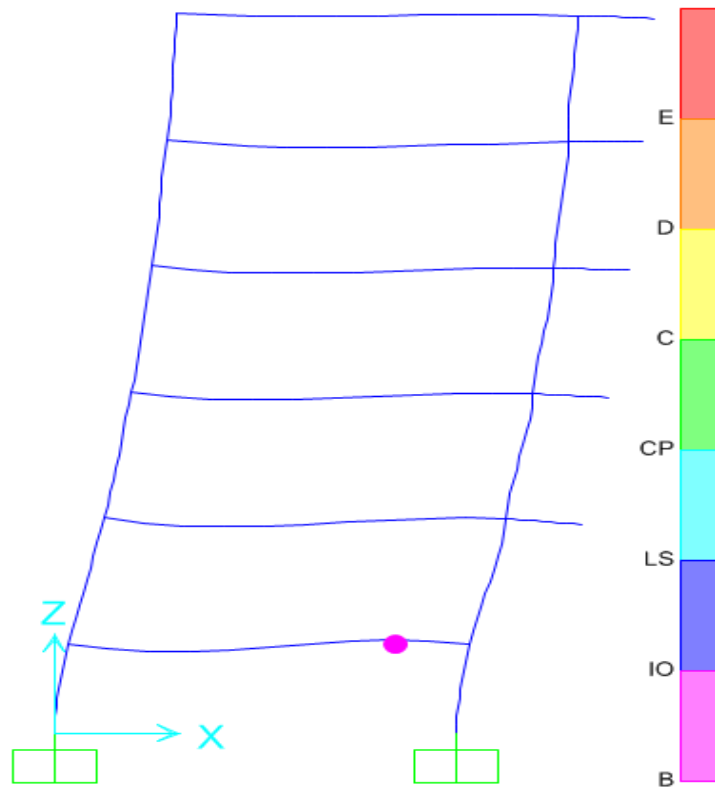


Figure B.2 Hinge property at Step 1

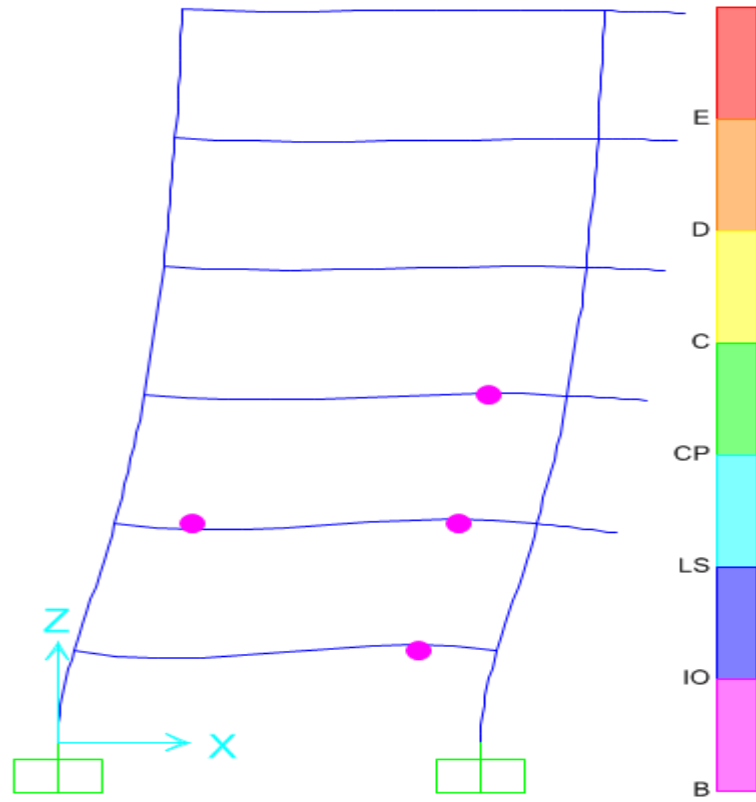


Figure B.3 Hinge property at Step 2

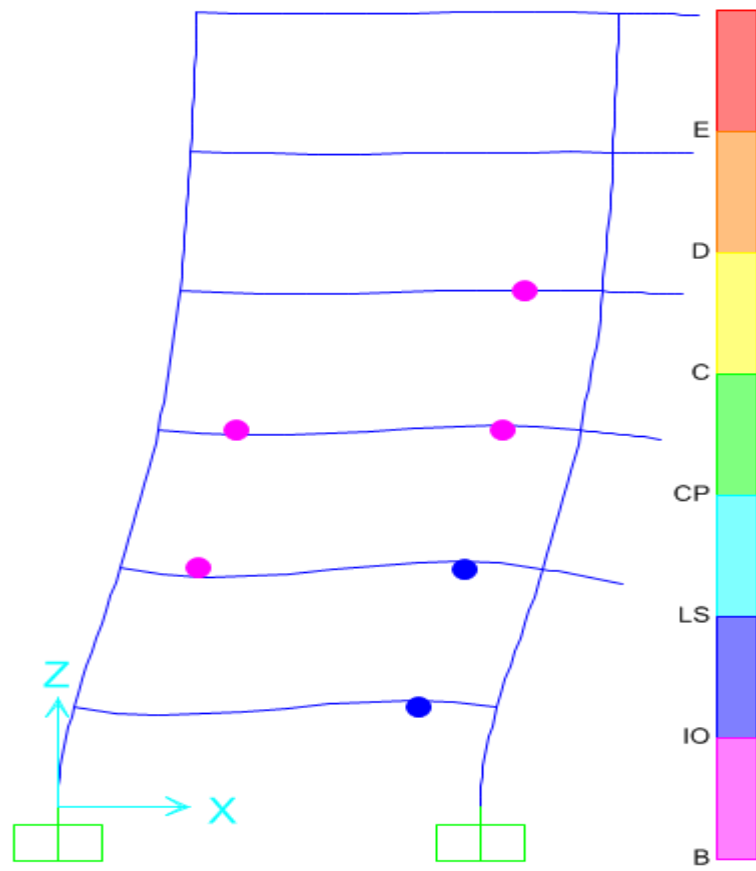


Figure B.4 Hinge property at Step 3

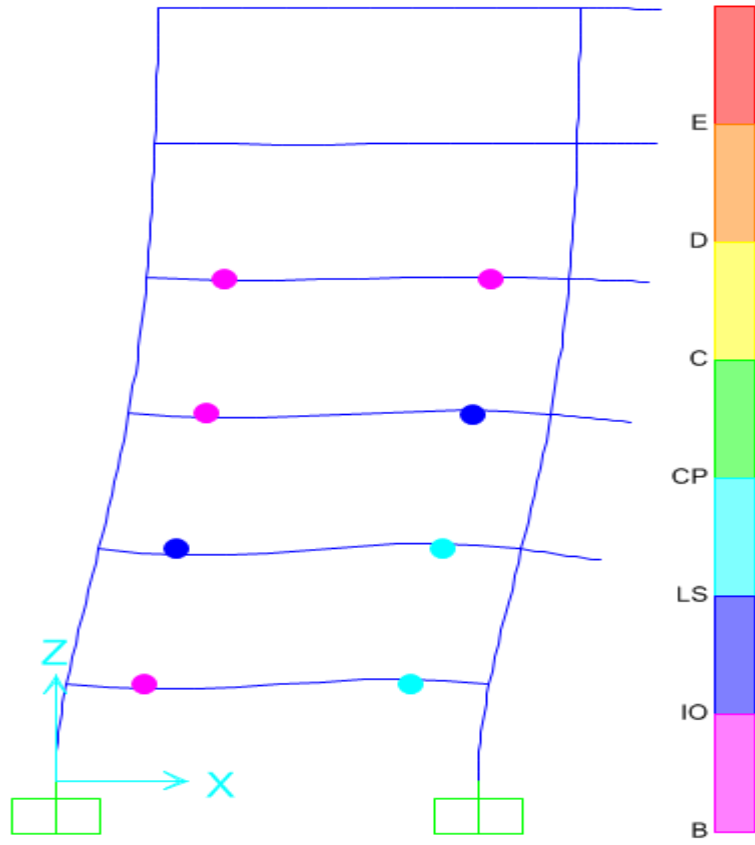


Figure B.5 Hinge property at Step 4

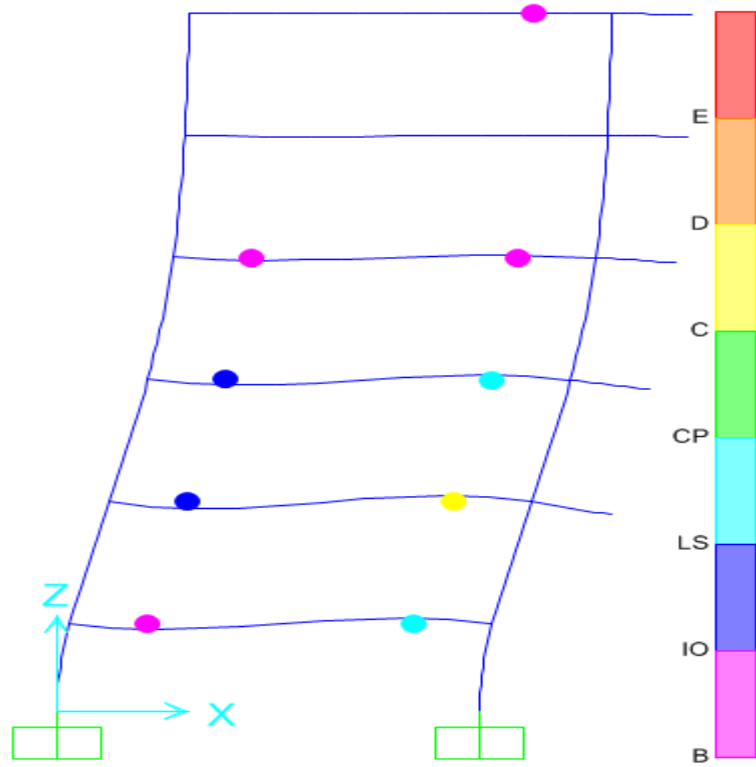


Figure B.6 Hinge property at Step 5

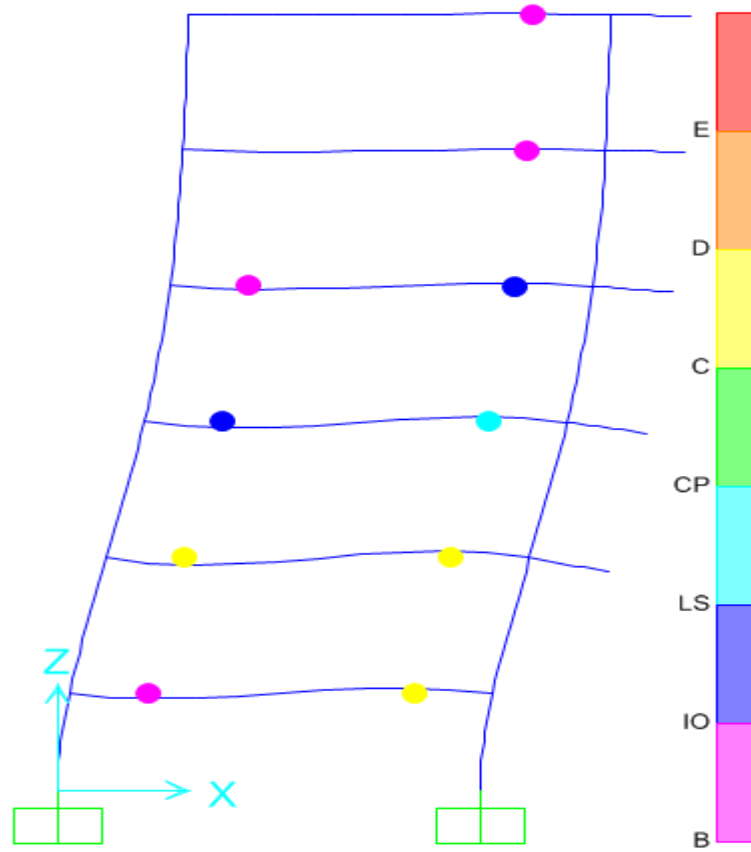


Figure B.7 Hinge property at Step 6

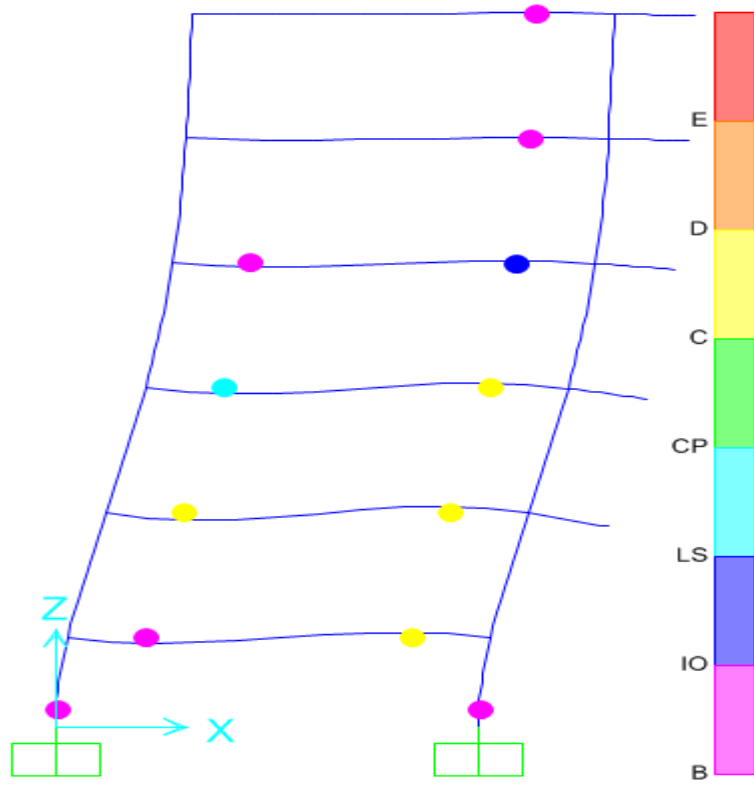


Figure B.8 Hinge property at Step 7

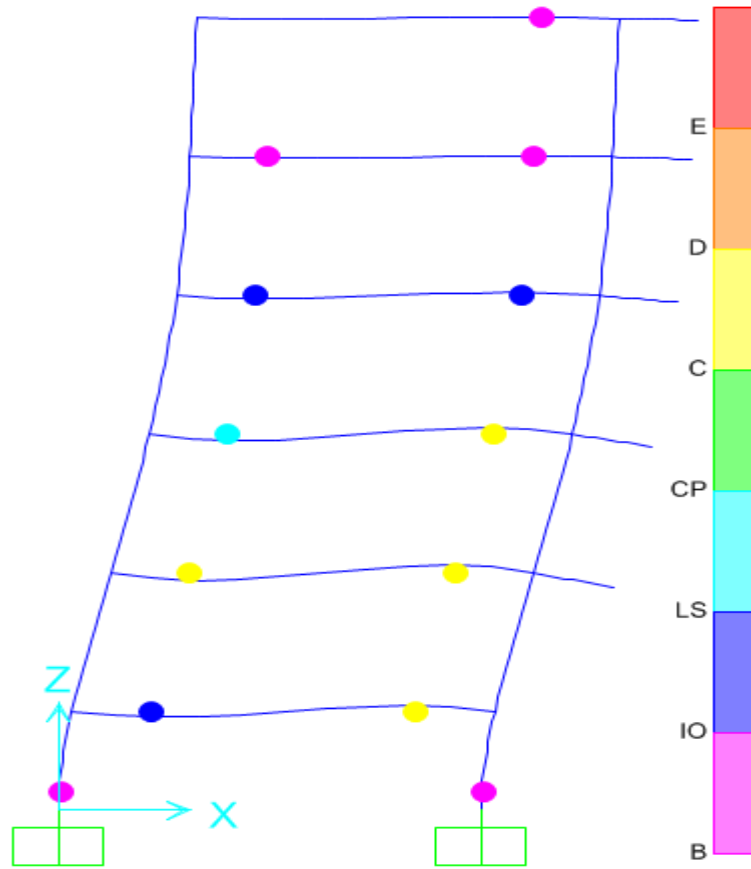


Figure B.9 Hinge property at Step 8

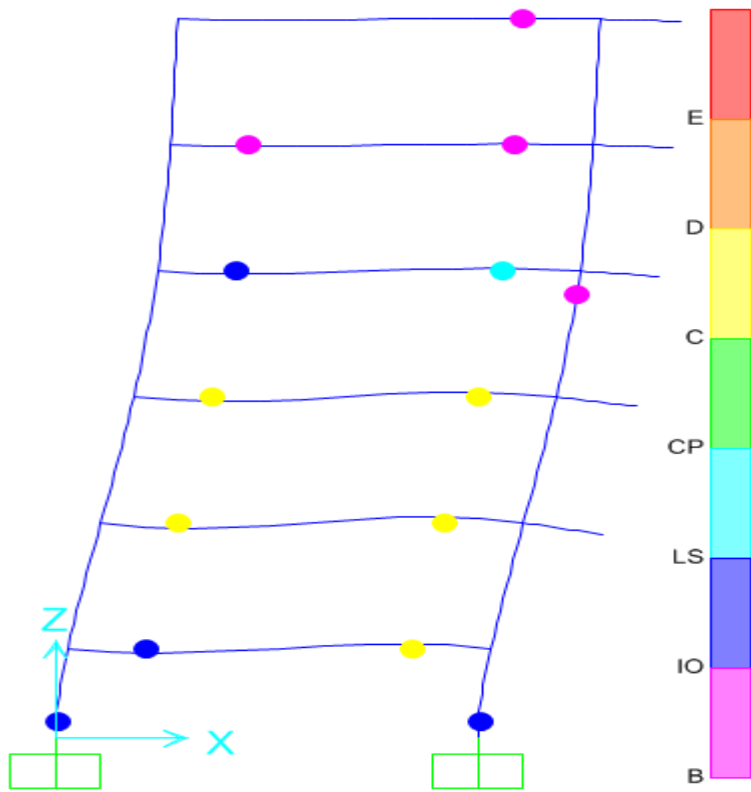


Figure B.10 Hinge property at Step 9

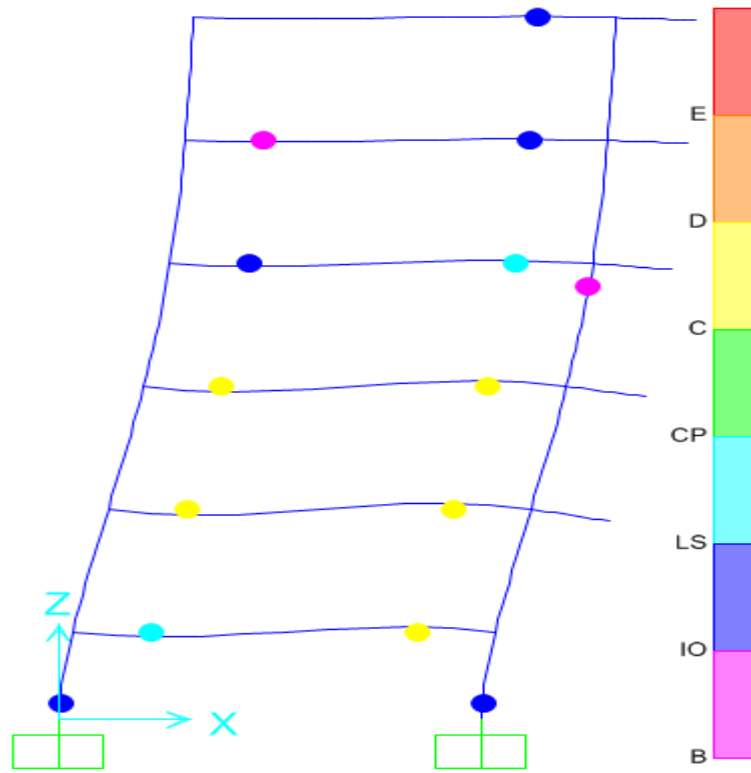


Figure B.11 Hinge property at Step 10

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