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SCHOOL OF GRADUATE STUDY

SESMIC EVALUATION USING NON LINEAR TIME HISTORY ANALYSIS

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

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The undersigned have examined the thesis entitled **'Seismic evaluation using non-linear time history analysis'** presented by **Netsanet Teferra,** a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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DECLARATION

I declare that ‘’this thesis is my original work and has not been presented for a degree in any other University, and that all sources of material used for the thesis have been duly acknowledged.’’

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NOTATION

For the purposes of this thesis, the following symbols apply. The notation used is based on ISO 3898:1987.

Latin upper case letters

C	Damping matrix
H	Height of the building
K	Stiffness matrix
M	Diagonal mass matrix
P_{tot}	Total gravity load
P_{NCR}	Reference probability of exceedance
S	Soil factor
$S_d(T_1)$	Ordinate of the design spectrum at period T1
$S_e(T)$	Elastic response spectrum
T_B	The upper limit of the period of the constant spectral acceleration branch
T_C	The upper limit of the period of the constant displacement response range of the spectrum
T_D	The beginning of the constant displacement response range of the spectrum
T_1	Fundamental periods of vibration
TH	time history
T_{NCR}	Reference return period
V_{tot}	Total seismic story shear

Latin lower case letters

a_g	The design ground acceleration
b_c	Cross-sectional dimension of column
d	Effective depth of cross section
d_r	Design inter-story drift
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
f_{cm}	Mean value of concrete cylinder compressive strength
f_{ctk}	Characteristic axial tensile strength of concrete
f_{cd}	Design value of concrete compressive strength

f_{yd}	Design value of yield strength of steel
g	Acceleration due to gravity
m	Mass of the building
x_u	The depth of the neutral axis at the ultimate limit state after redistribution
$u(t)$	Displacements of the structure
$\dot{u}(t)$	Velocities of the structure
$\ddot{u}$(t)	Accelerations of the structure

Greek lower case letters

δ	Ratio of the redistributed moment to the elastic bending moment.
ε_{cu2}	Ultimate compressive strain of unconfined concrete
ε_{cu}	Ultimate compressive strain in the concrete
ε_{c1}	Strain at peak stress
ξ	Viscous damping
η	Damping correction factor
θ	Inter-story drift sensitivity coefficient
λ	Correction factor

ABSTRACT

A building subjected to earthquake is expected to show inelastic behavior that the deformation in a member does not remain proportional to the internal force. A non-linear analysis accounts for the inelastic response. Both material non-linearity and geometric non-linearity case considered. There is plastic deformation and energy absorption in a member for higher levels of internal force. This type of nonlinear behavior is referred to as material non linearity. Several different hysteresis models are available to describe the behavior of different types of materials. For the most part, these differ in the amount of energy they dissipate in a given cycle of deformation, and how the energy dissipation behavior changes with an increasing amount of deformation. Pivot hysteresis model is used.

This thesis work uses fifteen story and two basement building of 40/60 project of City Government of Addis Ababa Saving House Enterprise as a case study. A comparison is made between linear elastic analysis and non-linear time history in-terms of story Displacement, Inter-story drift, story Shear, Axial force and Torsion. Thus it should not be surprising that buildings suffer damage during intense ground shaking. The main focus of this work is to evaluate the seismic resistance by using non-linear time history analysis so that the damage will be controlled to an acceptable degree.

Seismic input to nonlinear dynamic analyses of structures is usually defined in terms of acceleration time series. Three references Earthquake Ankober 2016, EL-Centro 1940 and Sierra-madre 1991 is used to generate artificial time history using time-domain method. The time domain method is generally considered a better approach for spectral matching since this method adjusts the acceleration time histories in the time domain by adding wavelets.

The major advantage of using the forces obtained from a time history 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.

CHAPTER ONE

INTRODUCTION

1.1 BACKGROUND

Earthquakes are one of nature greatest hazards to life on this planet. The impact of this phenomenon is sudden with little or no warning to make preparations against damages and collapse of buildings/structures. The hazard to life in case of earthquake is almost entirely associated with manmade structures such as buildings, dams, bridges etc.

A building subjected to earthquake is expected to show inelastic behavior that is the deformation in a member does not remain proportional to the internal force. A non-linear analysis accounts for the inelastic response. The calculated internal forces are better estimates than the values obtained from a linear analysis. A non-linear analysis also accounts for the redistribution of forces that occur in a structure as part of it undergoes inelastic response.

Nonlinear time history analysis methods generally provide more realistic models of structural response to strong ground shaking and, thereby, provide more reliable assessment of earthquake performance than nonlinear static analysis. It considers both material nonlinearity and geometrical non linearity. And the structural analysis is performed using ETABS finite element software.

A building subjected to earthquake is expected to show inelastic behavior that is the deformation in a member does not remain proportional to the internal force. A non-linear analysis accounts for the inelastic response.

Nonlinear time-history analyses are a very powerful tool,provided they are supported by proper approximations and modeling. The analysis is inherently complex and may be very time consuming, depending on the choice of the integration scheme, of the nonlinear incremental iterative algorithm strategy, and of the size of the mesh:

The main purpose of this paper is evaluation of the vulnerability of 40/60 project of city government of Addis Ababa saving house project enterprise by taking case study building of 2B+G+15 using nonlinear time history analysis.

1.2 OBJECTIVES

1.2.1 GENERAL OBJECTIVES

The purpose of this research is to investigate the seismic vulnerability of the case study building using nonlinear time history analysis.

1.2.2 SPECIFIC OBJECTIVES

This research has the specific objectives of:

- Seismic evaluation of 40/60 project of city government of Addis Ababa saving house project enterprise by taking case study 2B+G+15 sample buildings using nonlinear time history analysis.
- Investigate the nonlinear behavior of the case study buildings during the earthquake
- Identify critical region and check the details to assure that the structure has sufficient inelastic deformability to undergo fairly large deformations when subjected to a major earthquake.
- Proper assessment of the earth quake resistance helps the public body to ensure that all new construction should comply with design standard and for retrofitting of the existing buildings.

1.3 MATERIALS

- Information on case study buildings, drawings, building model and design report data.
- ETABS 2016 Integrated Building Design Software
- Recorded acclerograms data will be used that match the elastic response spectra from ES EN 1998-1:2015 for 5% viscous damping (ξ = 5%).
- The seismic motion will be represented in terms of ground acceleration-time histories.
- Books by different author's, code and specification and journals

1.4 METHODOLOGY

- The time-dependent response of the structure may be obtained through direct numerical integration of its differential equations of motion, using the accelerograms defined to represent the ground motions.

- Model and analyze using nonlinear time history method using ETABS2016 analysis software.

- Check whether the ductility and strength demand is satisfied with new EBCS code.

1.5 SCOPE OF THE STUDY

Only one case study building is taken and the influence of deep geology on seismic action is not considered

1.6 OUTLINE OF THE THESIS

The first chapter of this research presents general introduction.

The second chapter presents some important requirements of earth quake. The later part is type of structural model and analysis. Basic representation of ground motion is described. And finally design for ductility class medium requirement is described.

The third chapter is the case study building. Linear analysis result is shown and artificial earth quake is generated for time history analysis. Non-linear analysis is performed and comparison is made with linear analysis. To do that, non-linear time history analyses are carried out by using ETABS Integrated Building Design Software.

The fourth chapter presents conclusion and findings.

CHAPTER TWO

LITERATURE REVIEW AND CODE PROVISIONS

2.1 STRUCTURAL ANALYSIS

The analysis of a structural system to determine the deformations and forces induced by applied loads or ground excitation is an essential step in the design of a structure to resist earthquakes. A structural analysis procedure requires:

a) A model of the structure
b) A representation of the earthquake ground motion or the effects of the ground motion and
c) A method of analysis for forming and solving the governing equations

There is a range of methods from a plastic analysis to a sophisticated nonlinear, dynamic analysis of a detailed structural model that can be used, depending on the purpose of the analysis in the design process.

An important decision in a structural analysis is to assume whether the relationship between forces and displacements is linear or nonlinear. Linear analysis for static and dynamic loads has been used in structural design for decades. The emerging performance-based guidelines require representation of nonlinear behavior. There are two major sources of nonlinear behavior. The first is a nonlinear relationship between force and deformation resulting from material behavior such as ductile yielding, stiffness and strength degradation or brittle fracture. The second type of nonlinear behavior is caused by the inclusion of large displacements in the compatibility and equilibrium relationships.

An earthquake analysis generally includes gravity loads and a representation of the ground motion at the site of the structure. Earthquake ground motion induces the mass in a structure to accelerate, and the resulting response history can be computed by dynamic analysis methods. In many design procedures it is common to perform a dynamic analysis with a response spectrum representation of the ground motion expected at the site (A.K. Chopra, 2001). The structural analysis procedures used in earthquake-resistant design are summarized in Table 1. (Bozornia Y., and Bertero V.V., 2004)

The structure shall be designed and constructed to withstand the design seismic action without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events.

Table 1 Structural analysis procedures for earthquake resistant design.

Category	**Analysis Procedure**	**Force-Deformation relationship**	**Displacements**	**Earthquake load**	**Analysis method**
Equilibrium	Plastic analysis procedure	Rigid-plastic	small	Equivalent lateral load	Equilibrium analysis
Linear	Linear static procedure	linear	small	Equivalent lateral load	Linear static analysis
	Linear dynamic procedure I	linear	small	Response spectrum	Response spectrum analysis
	Linear dynamic procedure II	linear	small	ground motion history	linear response history analysis
non linear	nonlinear static procedure	nonlinear	small or large	Equivalent lateral load	nonlinear static analysis
	nonlinear dynamic procedure	nonlinear	small or large	ground motion history	nonlinear response history analysis

Nearly all structural analyses for earthquake-resistant design are performed using software that incorporates one or more of the analysis methods presented in this chapter. Modern software generally includes graphical features for visualizing the forces and deformations computed from an analysis. Before using any new structural analysis software, the

Engineer should conduct an independent verification to ensure that the software provides correct solutions. (Bozornia Y., and Bertero V.V., 2004)

2.1.1 NON-LINEAR TIME-HISTORY ANALYSIS

Nonlinear time history earthquake analysis is the most adequate and comprehensive analysis procedure to evaluate the nonlinear seismic response of structures (Sam Lee, 2008). Another statement by (Paolo Bazzurro, 2009) states that nonlinear dynamic earthquake analysis is today the current state of the art methodology for predicting building response to earthquake ground motion.

Time history analysis involves the direct integration of the equations of motion, which may be accomplished using the numerically dissipative-integration algorithm (Hilber *et al.*, 1977) or as a special case of the latter, the well-known Newmark scheme (Newmark, 1959). The nonlinearity of the analysis scheme calls for the use of an incremental iterative solution procedure: this means that loads are applied in predefined increments, equilibrated through an iterative scheme, whereby the internal forces corresponding to a displacement increment are computed until either convergence is achieved or the maximum number of iterations is reached. At the completion of each incremental solution, before proceeding to the next load increment, the stiffness matrix of the model is updated to reflect nonlinear changes in structural stiffness.

The solution algorithm may feature a hybrid incremental algorithm, obtained from a combination of the Newton-Raphson and the modified Newton-Raphson procedures, whereby the stiffness matrix is updated only in the first few iterations of a load step, thus obtaining an acceptable compromise between velocity in achieving convergence and required computational effort.

An important aspect of nonlinear time history analysis is the selection of time step size. The size of the time step has great effect on the accuracy, stability, and rate of convergence of the solution algorithm.

According to (Wilson, 2002) the major advantage of using the forces obtained from a time history 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.

As nonlinear time history analysis involves fewer assumptions than the nonlinear static procedure, it is subject to fewer limitations than nonlinear static procedure. However, the accuracy of the results depends on the details of the analysis model and how faithfully it captures the significant behavioral effects. Acceptance criteria typically limit the maximum structural component deformations to values where degradation is controlled and the nonlinear dynamic analysis models are reliable.

Given the inherent variability in the response of structures to earthquake ground motions and the many simplifying assumptions made in analysis, the results of any linear or nonlinear analysis for earthquake performance should be interpreted with care. While nonlinear dynamic analyses will, in theory, provide more realistic measures of response than other methods, the reliability of nonlinear dynamic analyses can be sensitive to modeling assumptions and parameters

For time history analysis the loading time history is divided into a number of small time increments, whereas, in the static analysis, the lateral force is divided into a number of small force increments. During a small time or force increment, the behavior of the structure is assumed to be linear elastic. As nonlinear behavior occurs, the incremental stiffness is modified for the next time or load increment. Hence, the response of the nonlinear system is approximated by the response of a sequential series of linear systems having varying stiffness (Anderson, 2000).

In a nonlinear analysis, the stiffness, damping, and load may all depend upon the displacements, velocities, and time. This requires an iterative solution to the equations of motion. Time history analysis: provides for linear or nonlinear evaluation of dynamic structural response under loading which may vary according to time function.

Time-history analysis is a step-by-step analysis of the dynamical response of a structure to a specified loading that may vary with time. The analysis may be linear or nonlinear. Time- history analysis is used to determine the dynamic response of a structure to arbitrary loading. The dynamic equilibrium equations to be solved are given by:

$$Ku(t) + C\dot{u}(t) + M\ddot{u}(\mathrm{t}) = r(t) \tag{1}$$

where

K is the stiffness matrix;

C is the damping matrix;

M is the diagonal mass matrix;

$u(t)$ is displacements of structures

$\dot{u}(t)$ is velocities of structures

$\ddot{u}$(t) is accelerations of structures

$r(t)$ is the applied load.

- The seismic motion may also be represented in terms of ground acceleration time histories and related quantities (velocity and displacement). (ES EN 1998-1:2015)
- The time-dependent response of the structure may be obtained through direct numerical integration of its differential equation of motion, using the accelerograms to represent the ground motions. (ES EN 1998-1:2015).

2.1.2 NONLINEAR TIME HISTORY ANALYSIS SOLUTION METHOD

Dynamic equation can be solved by following methods

a) Modal
b) Direct integration

These are two different solution methods, each with advantages and disadvantages. Under ideal circumstances, both methods should yield the same results to a given problem.

a) **Nonlinear Modal Time-History Analysis (Fast Non-linear Analysis):** The method is extremely efficient, particularly for structural systems which are primarily linear elastic but which have a limited number of predefined nonlinear elements. However, there is no limit on the number of nonlinear elements that can be considered, provided that adequate modes are obtained. This is best done using a sufficient number of Ritz vectors. For the FNA method, all nonlinearity is restricted to the Link/Support elements. Modal analysis is performed using the full stiffness matrix, K, and the mass matrix, M. It is strongly recommended that the Ritz vector method be used to perform the modal analysis.

b) Nonlinear Direct-Integration Time-History Analysis: While modal superposition is usually more accurate and efficient, direct-integration does offer the following advantages:

- Full damping that couples the modes can be considered
- Impact and wave propagation problems that might excite a large number of modes may be more efficiently solved by direct integration
- All types of nonlinearity may be included in a nonlinear direct integration analysis

Direct integration results are extremely sensitive to time-step size in a way that is not true for modal superposition. You should always run your direct-integration analyses with decreasing time-step sizes until the step size is small enough that results are no longer affected by it. For nonlinear direct-integration time-history analysis, the following are considered:

a) Material nonlinearity
b) Geometric nonlinearity

2.1.3 MATERIAL NON-LINEARITY

The response of structures deforming into their inelastic range during intense ground shaking is therefore of central importance in earthquake Engineering. This topic is concerned with this important subject.

In a non-linear inelastic analysis, the deformation in a member need not be proportional to the internal force. There is plastic deformation and energy absorption in a member for higher levels of internal force. This type of nonlinear behavior is referred to as material non linearity.

All material nonlinearity that has been defined in the model will be considered in a nonlinear direct-integration time-history analysis. If you are continuing from a previous nonlinear analysis, it is strongly recommended that you select the same geometric nonlinearity parameters for the current case as for the previous case.

During an earthquake structures undergo oscillatory motion with reversal of deformation.

Cyclic tests simulating this condition have been conducted on structural members, assemblages of members, reduced-scale models of structures, and on small full-scale structures. The experimental results indicate that the cyclic force-deformation behavior of a structure depends on the structural material and on the structural system. The force deformation plots show hysteresis loops under cyclic deformations because of inelastic behavior. The shape of these loops depends on the structural system and materials. (Chopra,1995).

Nonlinear time-history analyses are a very powerful tool, provided they are supported by proper approximations and modeling. The analysis is inherently complex and may be very time consuming (Pecker, 2007). Many Engineers use large values for element properties when modeling rigid parts of structures. This can cause large errors in the results for static and dynamic analysis problems. In the case of nonlinear analysis, the practice of using unrealistically large numbers can cause slow convergence and result in long computer execution times (Wilson, 2002).

2.1.4 STRESS STRAIN CURVE

Nonlinearity increases with increase in stress Post peak response is not really material response rather it is structural response under the same stress. Stronger concrete exhibits lower strain Post peak response is more brittle for stronger concrete. Confined concrete have higher strain than unconfined concrete. Post peak response is more brittle for stronger concrete. The relation between compressive stress(σ_c) and shortening strain(ε_c) for short term uniaxial loading is described by the expression. (ES EN 1992-1:2015)

$$\frac{\sigma c}{fcm} = \frac{k\eta-\eta 2}{1+(k-2)\eta} \qquad (2\ )$$

where

$\eta = \varepsilon c/\varepsilon c1$

εc1is the strain at peak stress

$k=1.05E_{cm}\varepsilon_{c1}/f_{cm}$ $\qquad$ $E_{cm}=22x\left({}^{f_{cm}}/_{10}\right)^{0.3}$

$\varepsilon_{c1}(\%o)=0.7fcm^{0.31} \leq 2.8$ $\qquad$ $f_{cm}=f_{ck}+8$Mpa

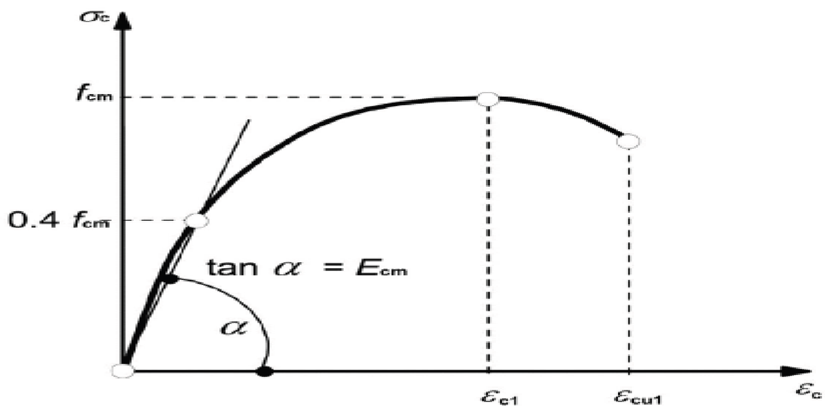

Figure 1 Schematic representative of the stress-strain relation of structural analysis.

2.1.5 HYSTERESIS BEHAVIOR

Hysteresis is the process of energy dissipation through deformation (displacement), as opposed to viscosity which is energy dissipation through deformation rate (velocity). Hysteresis is typical of solids, whereas viscosity is typical of fluids, although this distinction is not rigid.

Hysteretic behavior may affect nonlinear static and nonlinear time-history load cases that exhibit load reversals and cyclic loading.

Several different hysteresis models are available to describe the behavior of different types of materials. For the most part, these differ in the amount of energy they dissipate in a given cycle of deformation, and how the energy dissipation behavior changes with an increasing amount of deformation.

Each hysteresis model may be used for the following purposes:

- Material stress-strain behavior, affecting frame fiber hinges and layered shells that use the material
- Single degree-of-freedom frame hinges, such as M3 or P hinges. Interacting hinges, such as P-M3 or P-M2-M3, currently use the isotropic model
- Link/support elements of type multi-linear plasticity

The different types of hysteresis are:

a) Concrete b) Elastic c) Kinematic d) Takeda

e) Pivot f) BRB hardening g) Degrading h) Isotropic

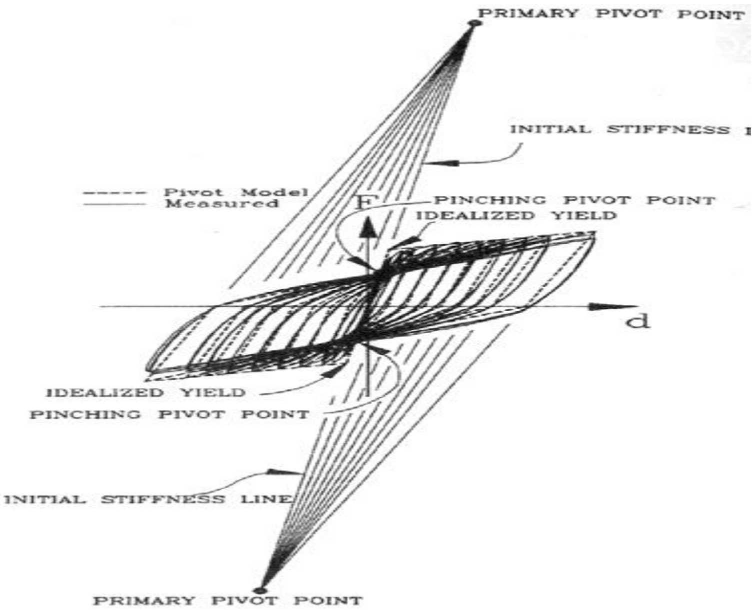

Figure 2 Hysteretic characteristics of a typical RC column and the idealization

The pivot hysteretic model utilizes the following two observations made from experimental hysteretic results of reinforced concrete members (Dowell et al, 1998):

a) Unloading stiffness decreases as displacement ductility increases, and
b) Following an inelastic excursion in one direction, upon load reversal, the force displacement path crosses the idealized initial stiffness line prior to reaching the idealized yield force (unlike elasto-plastic response).

2.1.6 GEOMETRIC NON-LINEARITY

Geometric nonlinear effects are caused by gravity loads acting on the deformed configuration of the structure, leading to an increase of internal forces in members and connections. These geometric nonlinear effects are typically distinguished between P-d effects, associated with deformations along the members, measured relative to the member chord, and P-D effects, measured between member ends and commonly associated with story drifts in buildings. In buildings subjected to earthquakes, P-D effects are much more of a concern than P-d effects, and provided that members conform to the slenderness limits for special systems in high seismic regions. P-d effects do not generally need to be modeled in nonlinear seismic analysis. On the other hand, P-D effects must be modeled as they can ultimately lead to loss of lateral resistance.

Large lateral deflections (D) magnify the internal force and moment demands, causing a decrease in the effective lateral stiffness. With the increase of internal forces, a smaller proportion of the structure's capacity remains available to sustain lateral loads, leading to a reduction in the effective lateral strength.

- P-delta effects
- Large displacement effects

2.1.7 SECOND ORDER EFFECTS

Second order effect was calculated by considering materials nonlinearity and deformed configuration in the analysis.

1) Second-order effects (P-$\Delta\ effects$) need not be taken into account if the following condition is fulfilled in all stories. (ES EN 1998-1:2015).

$$\theta = \frac{P_{tot}.d_r}{V_{tot}h} \leq 0.1 \tag{3}$$

where

θ is the inter-story drift sensitivity coefficient

P_{tot} is the total gravity load at and above the story considered in the seismic design situation

d_r is the design inter-story drift, evaluated as the difference of the average lateral displacement d_s at the top and bottom of the story under consideration.

V_{tot} is the total seismic story shear; and

h is the inter-story height.

2) If $0.1<\theta \leq 0.2$, the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1—\theta)$

3) The value of the coefficient θ shall not exceed 0.3.

2.2 BASIC REPRESENTATION OF GROUND MOTION

- Within the scope of (ES EN 1998-1:2015) the earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum, henceforth called an “elastic response spectrum”.

- The horizontal seismic action is described by two orthogonal components assumed as being independent and represented by the same response spectrum. (ES EN 1998-1:2015).
- For the three components of the seismic action, one or more alternative shapes of response spectra may be adopted, depending on the seismic sources and the earthquake magnitudes generated from them. (ES EN 1998-1:2015).
- In selecting the appropriate shape of the spectrum, consideration should be given to the magnitude of earthquakes that contribute most to the seismic hazard defined for the purpose of probabilistic hazard assessment, rather than on conservative upper limits (e.g. the Maximum Credible earthquake) defined for that purpose.
- When the earthquakes affecting a site are generated by widely differing sources, the possibility of using more than one shape of spectra should be considered to enable the design seismic action to be adequately represented. In such circumstances, different values a_g will normally be required for each type of spectrum and earthquake. (ES EN 1998-1:2015).

2.2.1 ACCELEROGRAMS

Recorded accelerograms or accelerograms generated through a numerical simulation of source and travel path mechanisms may be used, provided that the samples used are adequately qualified with regard to the seismogenetic features of the sources and to the soil conditions appropriate to the site, and their values are scaled to the value of a_gs for the zone under consideration. (ES EN 1998-1:2015).

Seismic analysis using time history require large number of seismic time history response analysis to reduce this we can use the generation of time histories whose response spectra closely match the “target” design response spectra.

Recent advances in the geological and seismological studies of earthquake ground motions have led to predictions of earthquake ground motions at a site with increasing accuracy based on the site specific parameters relating to the source, source to site travel path, and local site conditions. This implies that when generating synthetic ground motions, these motions can no longer be treated as random motions without proper considerations given to the site specific earthquake parameters. Recent Engineering studies have also shown that some characteristics inherent in actual earthquake motions such as the distribution of differential phases is closely related to the appearance and intensity envelope of the motions. It is, thus, important to preserve the actual characteristics of recorded motions as much as possible when they are utilized to generate synthetic time histories to fit target design spectra.

2.2.2 ARTIFICIAL ACCELEROGRAMS

- Depending on the nature of the application and on the information actually available, the description of the seismic motion may be made by using artificial accelerograms and recorded or simulated accelerograms. (ES EN 1998-1:2015).
- Artificial accelerograrms shall be generated so as to match the elastic response spectra given in code ES EN 1998-1:2015 for 5% viscous damping ($\xi = 5\%$).
- The duration of the accelerograrms shall be consistent with the magnitude and the other relevant features of the seismic event underlying the establishement of a_g (ES EN 1998-1:2015).
- When site-specific data are not available, the minimum duration Ts of the stationary part of the accelerograms should be equal to 10s. (ES EN 1998-1:2015).
- A minimum of 3 accelerograms should be used (ES EN 1998-1:2015).
- In the range of periods between 0.2T1 and 2T1, where T1 is the fundamental period of the structure in the direction where the accelerogram will be applied; no value of the mean 5% damping elastic spectrum, calculated from all time histories, should be less than 90% of the corresponding value of the 5% damping response spectrum.

2.2.3 RESPONSE SPECTRUM

A response spectrum is a conceptual spectrum that is different from the Fourier spectrum of time history. It is a series of maximum time history responses of one-degree-of-freedom (dof) systems having different natural frequencies. The given response spectrum does not contain important information such as phase angle. Moreover, thousands of artificial time histories can be derived from the given response spectrum.

2.2.4 HORIZONTAL ELASTIC RESPONSE SPECTRUM

- For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by the following expressions. (ES EN 1998-1:2015).

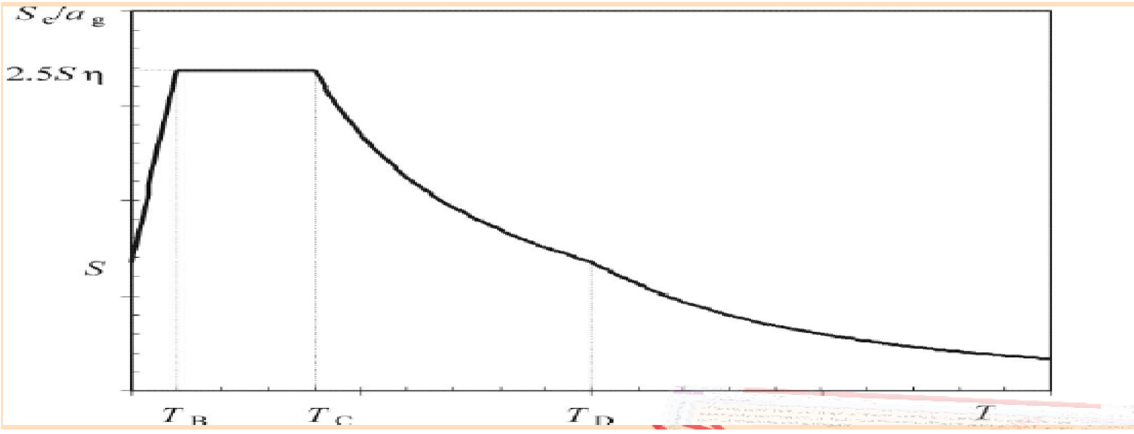

Figure 3 Shape of the elastic response spectrum

- The values of the period TB, TC and TD and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type. (ES EN 1998-1:2015).
- 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, Ms, not greater than 5.5, it is recommended that the type 2 spectrum is adopted. Otherwise use type 1. (ES EN 1998-1:2015).

$$0 \leq T \leq T_B: \quad S_e(T) = a_g.S.\left[1 + \frac{T}{T_B}.(\eta.2.5 - 1)\right] \tag{4}$$

$$T_B \leq T \leq T_C: \quad S_e(T) = a_g.S.\eta.2.5 \tag{5}$$

$$T_C \leq T \leq T_D: \quad S_e(T) = a_g.S.\eta.2.5.\left[\frac{T_C}{T}\right] \tag{6}$$

$$T_D \leq T \leq 4s: \quad S_e(T) = a_g.S.\eta.2.5\left[\frac{T_C.T_D}{T^2}\right] \tag{7}$$

where $S_e(T)$ is the elastic response spectrum;

T is the vibration period of a linear single degree-of-freedom systems;

a_g is the design ground acceleration on type A ground;

T_B is the upper limit of the period of the constant spectral acceleration branch;

T_C is the upper limit of the period of the constant displacement response range of the spectrum;

T_D is the value defining the beginning of the constant displacement response range of the spectrum;

S is the soil factor;

η is the damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping,

2.2.5 REFERENCE EARTH QUAKE

According to EBCS-8, 2014, a minimum of 3 accelerograms should be used. Therefore, three reference earth quake Ankober 2016, El-Centro 1940 and Sierra Madre 1991 earthquake are used as reference earthquake.

a) Ankober Earthquake

Ankober earth quake is recorded at furi mountain station on date December 4, 2016. It has strong motion duration 5.5sec. The magnitude is 4.6 and peak acceleration 0.00324g

b) 1940 El-Centro earth quake

The north—south component of the ground motion recorded at a site in El Centro, california during the Imperial Valley, California earthquake of May 18, 1940 is used. EL Centro 1940 north south component the instrument that recorded the accelerogram was attached to the EL Centro Terminal substation building's concrete floor, and not in free-field location. The record may have under-represented the high frequency motions of the ground because of soil structure interaction of the massive foundation with the surrounding soft soil. Nine people were killed by the may 1940 Imperial valley earth quake. At Imperial, 80 percent of the buildings were damaged to some degree. In business district of Brawley, all structures were damaged, and about 50 percent had to be condemned. The shock caused 40miles of surface faulting on imperial fault, part of the san Andreas system in southern California. Total damage has been estimated at about $6 million. The magnitude was 7.1. Numerical values for the ground acceleration in units of g. This includes 1559 data points at equal time spacing of 0.02 sec. (Chopra, 1995)

c) Sierra Madre earthquake

The 1991 Sierra Madre earthquake occurred on june 28 at 04:43:55 local time with moment magnitude of 5.6 and a maximum Mercalli intensity of Vll (very strong). The thrust earthquake resulted in two deaths, around 100injuries, and damage estimated at

$33.5-40million. The event occurred beneath the San Gabriel Mountains on the Clamshell-Sawpit Fault, which is a part of the sierra-madre Cucamonga Fault system. Depth 10Km.

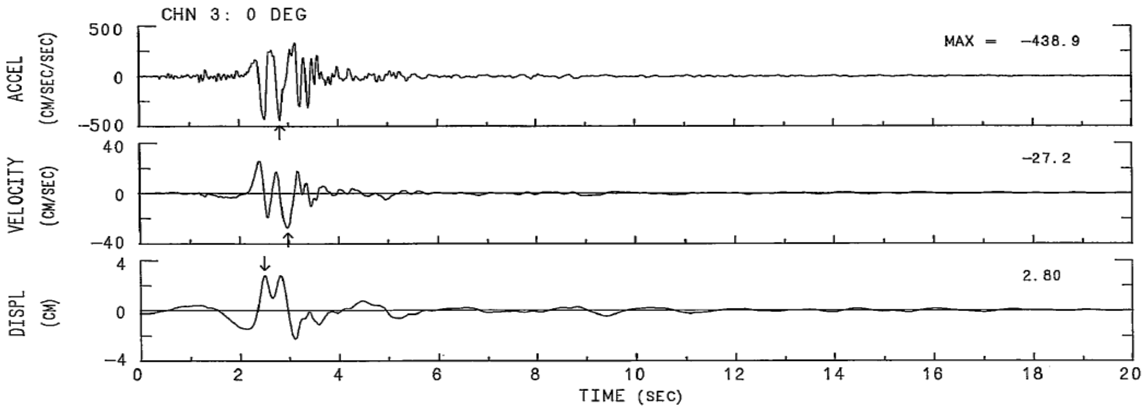

Figure 4 Sierra Madre earthquake

2.2.6 GENERATION OF SPECTRUM-COMPATIBLE TIME-HISTORY FUNCTIONS

Seismic input to nonlinear time-history analyses of structures is usually defined in terms of acceleration time series (time-history function). However, in most cases, the possibility of using real earthquake data is limited. Thus, artificial time histories are widely used instead. In many cases, however, response spectra are given. Thus, most of the artificial time histories are generated from the given response spectra. Obtaining the response spectrum from a given time history is straightforward. However, the procedure for generating artificial time histories from a given response spectrum is difficult and complex to understand. Various methods have been developed to modify a reference time series so that its response spectrum is compatible with a specified target spectrum. Two of the most widely used methods are the Frequency Domain Method and the Time Domain Method.

2.2.7 FREQUENCY DOMAIN METHOD

The procedure aimed at matching a reference strong motion to a target response spectrum in the Frequency domain uses the discrete Fourier transforms. In this case the Fourier amplitudes of a reference time history are adjusted based on a ratio between the target response spectrum and that for the reference time history. At this point the adjusted transforms are transformed back to a time domain. To assure zero acceleration at the start and end time step, the inverse Fourier transform of the modified time history is multiplied by a tapering function. The procedure is usually repeated for a specified number of iterations. The procedure adjusts the Fourier magnitudes of the time history without changing the Fourier phases and, as a result, the modified time history has the same frequency content as the reference time history. Therefore, in a case when the reference time history has a deficit in some range of frequency, this range cannot be enriched as was done for the matching procedure in the time domain. Another disadvantage of the procedure is that it does not have good convergence for multiple damping spectra. The time history matched to a target response spectrum for one damping generally produces a significant misfit between the target and the time history response spectrum for another damping. (Shakhzod M. Takhirov et al.2005)

Because the procedure adjusts only magnitudes of Fourier transforms without changing the phase angles, there is very limited change or no change at all in the frequency content of the modified signal. The changes are made for the magnitudes of the Fourier transforms only, including some amplification of the transform magnitudes in the high-frequency range. (Shakhzod M. Takhirov et al.2005)

This method adjusts the Fourier amplitude spectrum, based on the ratio of the target response spectrum to the time-series response spectrum while keeping the Fourier phase of the reference time history fixed. While this approach is relatively straightforward, it does not generally have good convergence properties. Also, this approach often alters the non-stationary character of the time series (ground motion accounting for the time variation of both the intensity and frequency content typical of real earth quake ground motions) to such a large degree that it no longer looks like a time series from an earthquake. Matching in the frequency domain invariably tends to increase the total energy in the ground motion.

ETABS Structural analysis and design software generates a synthetic time history, which response spectra match, or which are compatible with a set of specified smooth response spectra. The basis for the spectrum compatible time history generation is the relationship between the response spectrum values for specified damping and the “expected” Fourier amplitudes of the ground motion. The time history is synthesized by superimposing sinusoidal components with pseudo-random phase angles and by multiplying the resulting stationary trace by a user-specified function representing the variation of ground motion intensity with time (a tapered function).

The discrete Fourier transform (DFT) converts a finite sequence of equally-spaced samples of a function into an equivalent-length sequence of equally-spaced samples of the discrete-time Fourier transform (DTFT), which is a complex-valued function of frequency. The interval at which the DTFT is sampled is the reciprocal of the duration of the input sequence. An inverse DFT is a Fourier series, using the DTFT samples as coefficients of complex sinusoids at the corresponding DTFT frequencies. It has the same sample-values as the original input sequence. The DFT is therefore said to be a frequency domain representation of the original input sequence. If the original sequence spans all the non-zero values of a function, its DTFT is continuous (and periodic), and the DFT provides discrete samples of one cycle. If the original sequence is one cycle of a periodic function, the DFT provides all the non-zero values of one DTFT cycle. since it deals with a finite amount of data, it can be implemented in computers by numerical algorithms or even dedicated hardware. These implementations usually employ efficient fast Fourier transform (FFT) algorithms; so much so that the terms "FFT" and "DFT" are often used interchangeably.

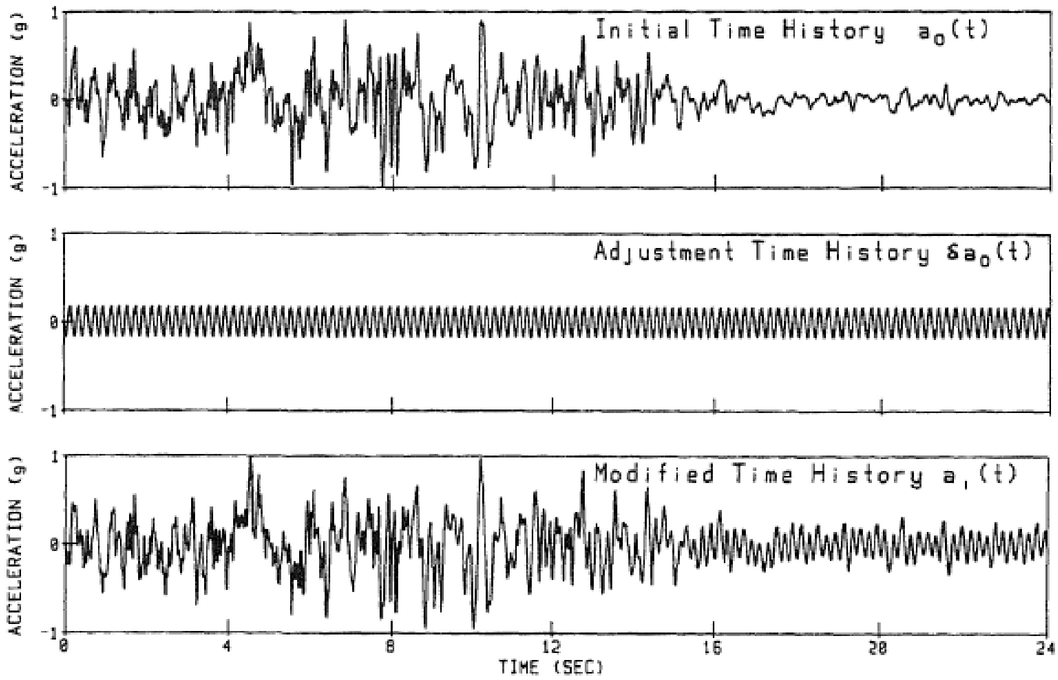

Figure 5 Time history adjustment by frequency domain adjustment procedure

2.2.8 TIME DOMAIN METHOD

The time domain method is generally considered a better approach for spectral matching since this method adjusts the acceleration time histories in the time domain by adding wavelets. A wavelet is a mathematical function that defines a waveform of effectively limited duration which has a zero average. The wavelet amplitude typically starts out at zero, increases, and then decreases back to zero. While the time domain spectral matching procedure is generally more complicated than the frequency domain approach, it has good convergence properties and in most cases preserves the non-stationary character of the reference time series.

The time domain method was first introduced Lilhanand and Tseng (1987, 1988). Lilhanand proposed an algorithm that uses reserve impulse wavelet functions to modify the initial time histories such that its response spectrum is compatible with a target spectrum. A fundamental assumption of this methodology is that the time of the peak response does not change as a result of the wavelet adjustment. This assumption is not always valid as the time of peak response may be shifted by adding the wavelet adjustments to the acceleration time history.

So how do we measure frequency, or how do we find the frequency content of a signal? The answer is Fourier transform (FT). If the FT of a signal in time domain is taken, the frequency-amplitude representation of that signal is obtained. In other words, we now have a plot with one axis being the frequency and the other being the amplitude. This plot tells us how much of each frequency exists in our signal

2.2.9 WAVELET

Like Fourier analysis, wavelet analysis deals with expansion of functions in terms of a set of basis functions. Unlike Fourier analysis, wavelet analysis expands functions not interms of trigonometric polynomials but interms of wavelets, which are generated in the form of translations and dilations of a fixed function called the mother wavelet. The wavelets obtained in this way have special scaling properties. They are localized in time and frequency, permitting a closer connection between the function being represented and their coefficients. Greater numerical stability in reconstruction and manipulation is ensured.

The objective of wavelet analysis is to define these powerful wavelet basis functions and find efficient methods for their computation. (Daniel T.L Lee and Akio yamamoto, 1994).

2.2.10 WAVELET TRANSFORM

Wavelet transform is capable of providing the time and frequency information simultaneously, hence giving a time-frequency representation of the signal. The WT was developed as an alternative to the Short time Fourier transform. It suffices at this time to say that the WT was developed to overcome some resolution related problems of the STFT.

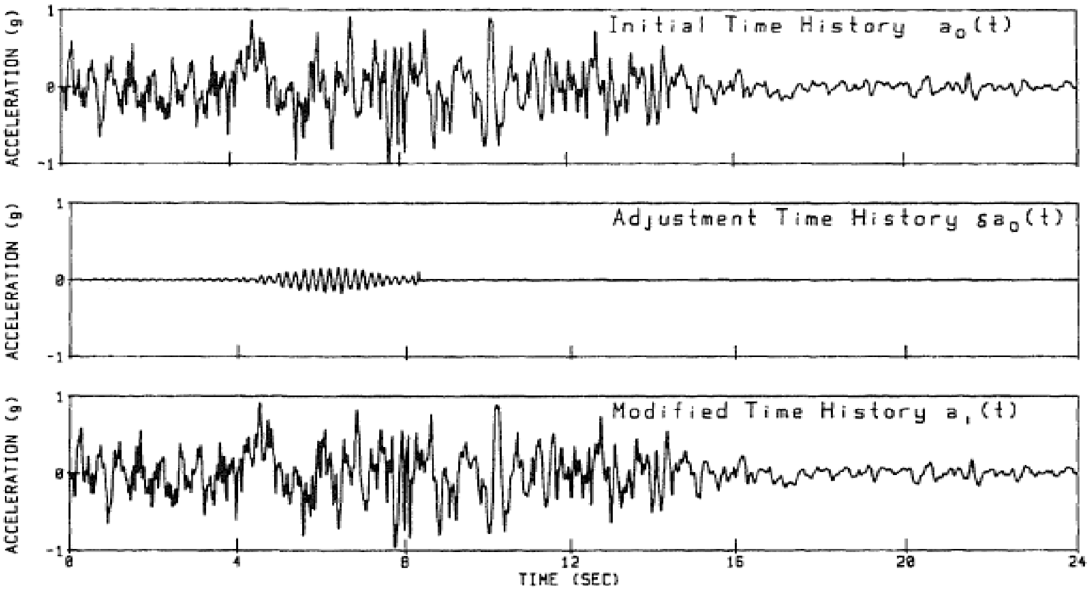

Figure 6 Time history adjustment by time domain adjustment procedure (Lilhanand K. and Tseng W.S, 1988)

To form an initial time history, the response spectrum acceleration at ω_n of the reference response spectrum is used as the magnitude An of $x\ddot{(t)}$. The phase angle φn can be chosen randomly between 0 and 2π. in reality, the earthquake gradually increases its magnitude, holds its maximum magnitude for some time, and then fades out. Therefore, to generate more realistic characteristics of earthquake acceleration, an envelope function containing build-up, intense motion (strong motion), and decay is adopted.

From the initial time history; whereas the time domain method produces a localized perturbation around t_i and, as a result, the modified time history resembles closely the initial time history.

2.3 EARTH QUAKE REQUIREMENT

Structure in seismic regions shall be designed and constructed in such a way that the following requirements are met, each with an adequate degree of reliability. (EBCS-8, 2014)

2.3.1 NO-COLLAPSE REQUIREMENT

The structure shall be designed and constructed to withstand the design seismic action without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events. The design seismic action is expressed in terms of:

a) The reference seismic action associated with a reference probability of exceedance, P_{NCR} in 50 years or a reference return period, T_{NCR}

b) The importance factor γ_1

The recommended values are P_{NCR} =10% and T_{NCR} =475 years.

2.3.2 DAMAGE LIMITATION REQUIREMENT

The structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use. The seismic action to be taken into account for the "damage limitation requirement" has a probability of exceedance, $P_{DLR,}$ in 10 years and a return period, $T_{DLR.}$ The recommended values are P_{DLR}=10% and T_{DLR}=95 years. (EBCS-8, 2014)

- An adequate degree of reliability against unacceptable damage shall be ensured by satisfying the deformation limits or other relevant limits defined in the relevant parts of ES EN 1998-1:2015.
- In structure importance for civil protection the structural system shall be verified to ensure that it has sufficient resistance and stiffness to maintain the function of the vital services in the facilities for a seismic event associated with an appropriate return period.
- To the extent possible, structures should have simple and regular forms both in plan and elevation, if necessary this may be realized by subdividing the structure by joints into dynamically independent units.
- In order to ensure an overall dissipative and ductile behavior, brittle failure or the

premature formation of unstable mechanisms shall be avoided. To this end, where required in the relevant parts of EBCS-8, 2014 resort shall be made to the capacity design procedure, which is used to obtain the hierarchy of resistance of the various structural components and failure modes necessary for ensuring a suitable plastic mechanism and for avoiding brittle failure modes.

- Since the seismic performance of a structure is largely dependent on the behavior of its critical regions or elements, the detailing of structure in general and of these regions or elements in particular, shall be such as to maintain the capacity to transmit the necessary forces and to dissipate energy under cyclic conditions. To this end, the detailing of connections between structural elements and of regions where nonlinear behavior is foreseeable should receive special care in design.
- The analysis shall be based on an adequate structural model, which, when necessary, shall take into account the influence of soil deformability and of non-structural elements and other aspects, such as the presence of adjacent structures.

2.3.3 SEISMIC VULNERABILITY

According to (Elghazouli, 2009), seismic vulnerability is the susceptibility of structures to earthquake effects and is generally defined by the expected degree of damage that would result under different levels of seismic demand.

Seismic vulnerability of structures varies as a function of construction materials and earthquake action-resistance system employed. However, causes of fatalities and extent of damage depend to a great extent on the type of constructions and the density of population in the area (Elnashai and Sarno, 2008). Earthquake can cause devastative effects in terms of loss of life and livelihood. The destructive potential of earthquake depends on many factors such as:

- Size of an event
- Focal depth and Epi-central distance
- Topographical conditions
- Local geology
- Type of building

Vulnerability can be both evaluated and reduced, by measures of retrofitting.

2.3.4 STRUCTURAL ASSESSMENT

(Eurocode–8, 2003) defines assessment as a quantitative procedure by which it is checked whether an existing undamaged or damaged building can resist the design seismic load combination as specified in the code. It further states that, assessment is made for individual buildings, in order to decide about the need for structural intervention and about the strengthening or repair measures to be implemented.

If there is a design defect, then the performance is expected to fall below the requirement of standards. But with the implementation of the codes, the structural performance improves.

The incorporation of seismic design procedures in building design was first adopted in general sense in the 1920s and 1930s, when the importance of inertial loadings of buildings began to be appreciated. In the absence of reliable measurements of ground accelerations and as a consequence of the lack of detailed knowledge of the dynamic response of structures, the magnitude of seismic inertial forces could not be estimated with any reliability. Typically, design for lateral forces corresponding to about 10% of the building weight was adopted.

The performance of a frame under lateral loads generates from the flexural action of beams and columns and the flexural rigidity of the beam-column joints. There should be adequate number of well laid out frames in the two orthogonal directions in order to generate the lateral stiffness and strength of a building, some existing building configurations that are adequate for resisting gravity loads, are not suitable for resisting earthquake forces. It is essential to identify the deficiencies in a building before undertaking retrofit and Identification of the deficiencies is also expected to create awareness for future construction.

By the 1960s accelerograms giving detailed information on the ground acceleration occurring in earthquakes were becoming more generally available. The advent of strength design philosophies, and development of sophisticated computer-based analytical procedures, facilitated a much closer examination of the seismic response of multi-degree of freedom structures.

With increased awareness that excessive strength is not essential or even necessarily desirable, the emphasis in design has shifted from the resistance of large seismic forces to the "evasion" of these forces. Inelastic structural response has emerged from the obscurity of hypotheses, and become an essential reality in the assessment of structural design for earth quake forces. The reality that all inelastic modes of deformation are not equally viable has become accepted. As noted above, some lead to failure and others provide ductility, which can be considered the essential attribute of maintaining strength while the structure is subjected to reversals of inelastic deformations under seismic response.

2.3.5 REQUIRED INPUT DATA

The data collection process includes acquisition of available documents, field observations and documentation. Similarly, (Eurocode–8, 2003) recommends the following inputs as requirement for structural assessment.

a) Identification of the structural system and of its compliance with the regularity criteria in section 4.2.3 of EN 1998-1. The information should be collected either from onsite investigation or from original design drawings, if available. In this latter case, information on possible structural changes since construction should also be collected.
b) Identification of the type of building foundations.
c) Identification of the ground conditions
d) Information about the overall dimensions and cross-sectional properties of the building elements and the mechanical properties and condition of constituent materials.
e) Information about identifiable material defects and inadequate detailing.
f) Information on the seismic design criteria used for the initial design, including the value of the force reduction factor, if applicable.
g) Description of the present and/or the planned use of the building (with identification of its importance category).
h) Information about the type and extent of previous and present structural damages, if any, including earlier repair measures.

2.4 STRUCTURAL MODELING

A structural analysis is performed on a model of the structure not on the real structure so the analysis can be no more accurate than the assumptions in the model. The model must represent the distribution and possible time variation of stiffness, strength, deformation capacity and mass of the structure with accuracy sufficient for the purpose of the analysis in the design process. All structures are three dimensional, but it is important to decide whether to use a three-dimensional model or simpler two-dimensional models. The analysis methods are the same whether the model is two-dimensional or three-dimensional. Generally, two dimensional models are acceptable for buildings with regular configuration and minimal torsion; otherwise, a three-dimensional model is necessary with a representation of the floor diaphragms as rigid or flexible components. (ATC, 1996a, Priestley et al, 1996).

For the accurate description of the nonlinear dynamic response of RC frame structures three dimensional models are the best solution. At present the refined three dimensional dynamic analyses of RC buildings is computationally very intensive.

Structural models are idealizations of the prototype and are intended to simulate the response characteristics of systems. Three levels of modeling are generally used for earthquake response analysis, substitution model; stick model and detailed model (Elnashai and Sarno, 2008).

Substitute models are idealized as an equivalent single degree of freedom system or 'substitute system'. Four parameters are needed to define the substitute system: effective mass, effective height, effective stiffness and effective damping. The height defines the location of the equivalent or effective mass of the substitute system.

Stick models consist of multi degree of freedom systems in which each element idealizes a number of members of the prototype structure. In multi - story building frames, each story is modelled by a single line of finite elements representing the deformational characteristics of all columns and their interaction with beams. For three - dimensional models, the stick element relates the shear forces along two horizontal orthogonal directions and the story torque to the corresponding inter-story translations and rotations, respectively.

Detailed models include general finite elements idealizations in which structures are discretized into a large number of elements with section analysis or spatial elements in 2D or 3D. Such a modeling approach allows representation of details of the geometry of the members, and enables the description of the history of stresses and strains at fibers along the length or across the section dimensions. In the detailed modeling approach, beams and columns of frames are represented by flexural elements, braces by truss elements, and shear and core walls by 2D elements, such as plates and shells (Elnashai and Sarno, 2008).

(Saatcioglu and Humar, 2002) illustrate the modeling of common forms of buildings structures with line elements. In frame and frame-wall interactive systems the vertical elements such as columns and flexure-dominant structural walls are represented by vertical line elements, and the horizontal elements such as beams, and slab diaphragms are represented by horizontal line elements. Sometimes shear panels, like shear walls and in-fill panels, as well as bracing elements are modeled by diagonal struts and ties.

2.5 DESIGN FOR DUCTILITY CLASS MEDIUM (DCM)

- It shall be verified that both the structural elements and the structure as a whole possess adequate ductility, taking into account the expected exploitation of ductility, which depends on the selected system and the behavior factor. (ES EN 1998-1:2015).
- The following requirements shall be satisfied in order to obtain the hierarchy of resistance of the various structural components necessary for ensuring the intended configuration of plastic hinges and for avoiding brittle failure modes.
- In multi-story buildings formation of a soft story plastic mechanism shall be prevented, as such a mechanism might entail excessive local ductility demands in the columns of the soft story (ES EN 1998-1:2015).

2.5.1 MATERIAL REQUIREMENTS

- With the exception of closed stirrups and cross-ties, only ribbed bars shall be used as reinforcing steel in critical regions of primary seismic elements. (ES EN 1998-1:2015).
- In critical regions of primary seismic elements reinforcing steel of class B or C in ES EN 1992-1-1:2015, Table C.1 shall be used.

2.5.2 GEOMETRIC CONSTRAINTS

- **Beam**
  a) The eccentricity of the beam axis shall be limited relative to that of the column into which it frames to enable efficient transfer of cyclic moments from a primary seismic beam to a column to be achieved. To enable this requirement, the distance between the centroidal axes of the two members should be limited to less than bc/4, where bc is the largest cross-sectional dimension of the column normal to the longitudinal axis of the beam. (ES EN 1998-1:2015).
  b) To take advantage of the favorable effect of column compression on the bond of

horizontal bars passing through the joint, the width bw of a primary seismic beam shall satisfy the following expression: (ES EN 1998-1:2015).

$$b_w \le min\{bc + hw; 2bc\} \quad (8\ )$$

where hw is the depth of the beam

- **Column**

Unless the inter-story drift sensitivity coefficient $\theta \le 0.1$, the cross-sectional dimensions of primary seismic columns should not be smaller than one tenth of the larger distance between the point of contra flexure and the ends of the column, for bending within a plane parallel to the column dimension considered. (ES EN 1998-1:2015).

$$\theta = \frac{P_{tot}.d_r}{V_{tot}h} \le 0.1 \quad (9\ )$$

where θ is the inter-story drift sensitivity coefficient

P_{tot} is the total gravity load at and above the story considered in the seismic design situation

d_r is the design inter-story drift, evaluated as the difference of the average lateral displacement d_s at the top and bottom of the story under consideration.

V_{tot} is the total seismic story shear; and

h is the inter-story height.

- **Ductile walls**

(a) The thickness of the web, b_{wo}(in meters) should satisfy the following expression:

$$b_{wo} \ge max\{0.15, hs/20\} \quad (10\ )$$

where hs is the clear story height in meters.

2.5.3 BEAMS SUPPORTING DISCONTINUED VERTICAL ELEMENTS

a) Structural walls shall not rely for their support on beams or slabs. (ES EN 1998-

1:2015).

b) For a primary seismic beam supporting columns discontinued below the beam, the following rules apply: (ES EN 1998-1:2015).

   i. There shall be no eccentricity of the column axis relative to that of the beam;

   ii. The beam shall be supported by at least two direct supports; such as walls or columns.

- **Beam**

a) In primary seismic beams the design shear forces shall be determined in accordance with the capacity design rule, on the basis of the equilibrium of the beam under:

   a) The transverse load acting on it in the seismic design situation and

   b) End moments Mi,d(with i=1,2 denoting the end sections of the beam), corresponding to plastic hinge formation for positive and negative directions of seismic loading. The plastic hinges should be taken to form at the ends of the beams or (if they form there first) in the vertical elements connected to the joints into which the beam ends frame. (ES EN 1998-1:2015).

- **Columns**

a) In primary seismic columns the design values of shear forces shall be determined in accordance with the capacity design rule, on the basis of the equilibrium of the column under end moments Mid (with i=1,2 denoting the end sections of seismic loading. The plastic hinges should be taken to form at ends of the beams connected to the joints into which the column end frames, or in the column. (ES EN 1998-1:2015).

2.5.4 DISPLACEMENT ANALYSIS

For both static and dynamic non-linear analysis, the displacements determined are those obtained directly from the analysis without further modification. (ES EN 1998-1:2015).

2.5.5 LIMITATION OF INTER STORY DRIFT

- For building having non-structural elements of brittle materials attached to the structure:

  $d_r v \ \leq 0.005h;$

- For buildings having ductile non-structural elements:

  $d_r v \leq 0.0075h;$

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

  $d_r v \leq 0.01h;$

where d_r is the design inter-story drift, evaluated as the difference of the average lateral displacement

ds at the top and bottom of the story under consideration.

h is the story height

v is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement. Different values of v may be defined for the various seismic zones, depending on the seismic hazard conditions and on the protection of property objectives. The recommended values of v are 0.4 for importance classes III and 1V and v = 0.5 for importance class I and II (ES EN 1998-1:2015).

CHAPTER THREE

CASE STUDY

3.1 DESCRIPTION OF THE CASE STUDY

The case study structure is a tall reinforced concrete frame-wall structure with fifteen stories and two basements. It is located at Addis Ababa, Bole Ayat site constructed by City Government of Addis Ababa Saving House Enterprise. Typical floor plan is shown in fig 10. The building consists of core wall structures and columns connected by beams to form moment resisting frames. The building design was done by ETG designers and consultants PLC Using linear elastic analysis; the data is shown in the attached file on a CD. On this work the building is examined in different directions for time history analysis and comparison is made with the original design.

Figure 8 40/60 Apartment Building 2B+G+15

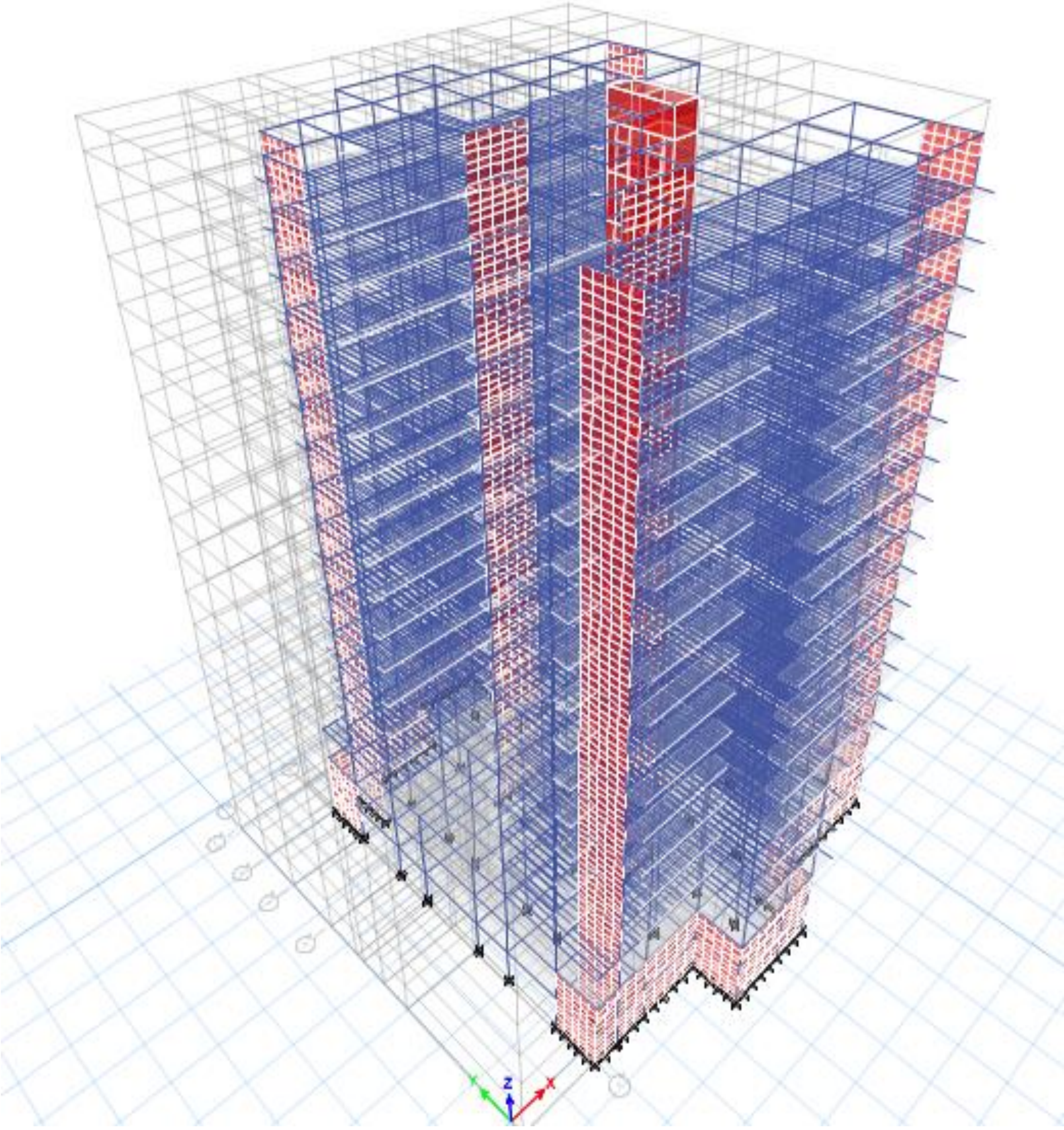

Figure 9 3D model of the case study

3.1.1 MATERIAL USED

The concrete and reinforcement material used in original design is shown in Table 2.

Table 2 Material used in the case study

Material	Type	Compressive Strength
Concrete	C-25	25MPa
	C-30	30MPa
Reinforcement bars	S300	300MPa
	S400	400MPa

3.1.2 LOAD CASE USED BY THE DESIGNER IN THE ORIGINAL DESIGN

This part describes how to define structural loads for the case study including dead, live and earth quake load

1. Earthquake-linear static analysis (EQx+, EQx-, EQy+, EQy-)

In this research all possible load combination was defined to get maximum action effect of the structural elements

Table 3 Load combination

COMB	Number	Loads
1	1	1.35DL + 1.5LL
	2	DL+ 0.3LL
2	1	COMB1-2 $\pm EQ_{X1}$ $\pm 0.3EQ_{Y1}$ $\pm Imp_x$
	2	COMB1-2 $\pm$ EQ_{X2} $\pm$ $0.3EQ_{Y1}$ $\pm Imp_x$
	3	COMB1-2 $\pm EQ_{Y1}$ $\pm$ $0.3EQ_{X1}$ $\pm$ Imp_x
	4	COMB1-2 $\pm$ EQ_{Y2} $\pm$ $0.3EQ_{X1}$ $\pm$ Imp_x

Earth quake consideration

- Earth quake direction +eccentricity
- Base shear coefficient, C=0.027(user coefficient)
- Building height exp, K=1
- Ecc.ratio(all Diaph) =0.05

3.2 ELASTIC ANALYSIS RESULT OF THE CASE STUDY STRUCTURE RESULT

3.2.1 MAXIMUM STORY DISPLACEMENTS AND DRIFTS

Table 4 and 5 shows the maximum displacements and inter-story drifts for linear elastic case of combination Combo-3 and Combo-9.

Table 4 Maximum story displacement linear static case

Story	**Elevation (m)**	**X-Dir (mm)**	**Y-Dir (mm)**	**Maximum inter-story drift**	
				X-Dir	**Y-Dir**
TTB	51.20	80.60	10.40		
15TH	48.00	76.90	9.90	0.0005781	0.0000781
14TH	44.80	72.30	9.30	0.0007188	0.0000937
13TH	41.60	67.40	8.70	0.0007656	0.0000938
12TH	38.40	63.90	8.30	0.0005469	0.0000625
11TH	35.20	60.40	7.90	0.0005469	0.0000625
10TH	32.00	56.40	7.40	0.0006250	0.0000781
9TH	28.80	51.70	6.80	0.0007344	0.0000938
8TH	25.60	46.50	6.10	0.0008125	0.0001094
7TH	22.40	40.80	5.30	0.0008906	0.0001250
6TH	19.20	34.80	4.40	0.0009375	0.0001406
5TH	16.00	28.50	3.50	0.0009844	0.0001406
4TH	12.80	22.00	2.60	0.0010156	0.0001406
3RD	9.60	15.40	1.80	0.0010313	0.0001250
2ND	6.40	9.00	1.00	0.0010000	0.0001250
1ST	3.20	3.80	0.40	0.0008125	0.0000938
GR	0.00	0.70	0.10	0.0004844	0.0000469
BSMT1	-3.20	0.20	0.05	0.0000781	0.0000078

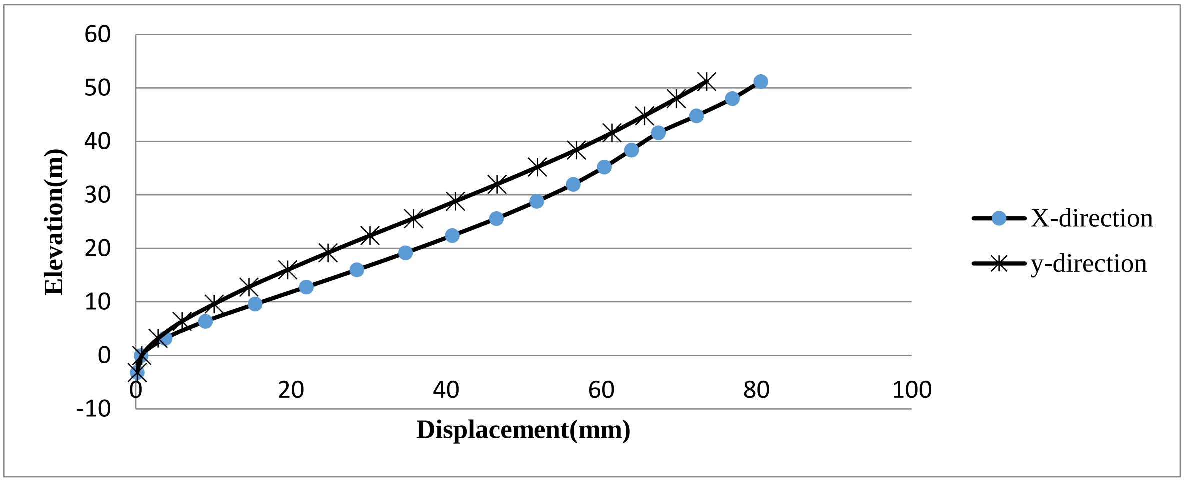

Figure 10 Linear static Elevation Vs maximum story displacement

Table 5 Maximum story displacement linear static

Story	Elevation(m)	X-Dir (mm)	Y-Dir (mm)	Maximum inter-story drift	
				X-Dir	Y-Dir
TTB	51.20	18.20	73.60		
15TH	48.00	18.70	69.70	0.0000781	0.0006094
14TH	44.80	17.90	65.60	0.0001250	0.0006406
13TH	41.60	17.00	61.40	0.0001406	0.0006562
12TH	38.40	15.90	56.80	0.0001719	0.0007188
11TH	35.20	14.60	51.80	0.0002031	0.0007813
10TH	32.00	13.20	46.60	0.0002188	0.0008125
9TH	28.80	11.80	41.20	0.0002188	0.0008438
8TH	25.60	10.40	35.80	0.0002188	0.0008438
7TH	22.40	8.90	30.20	0.0002344	0.0008750
6TH	19.20	7.40	24.80	0.0002344	0.0008438
5TH	16.00	6.00	19.60	0.0002188	0.0008125
4TH	12.80	4.60	14.60	0.0002188	0.0007813
3RD	9.60	3.20	10.10	0.0002188	0.0007031
2ND	6.40	1.90	6.00	0.0002031	0.0006406
1ST	3.20	0.90	2.90	0.0001563	0.0004844
GR	0.00	0.30	0.80	0.0000938	0.0003281
BSMT1	-3.20	0.10	0.20	0.0000313	0.0000938

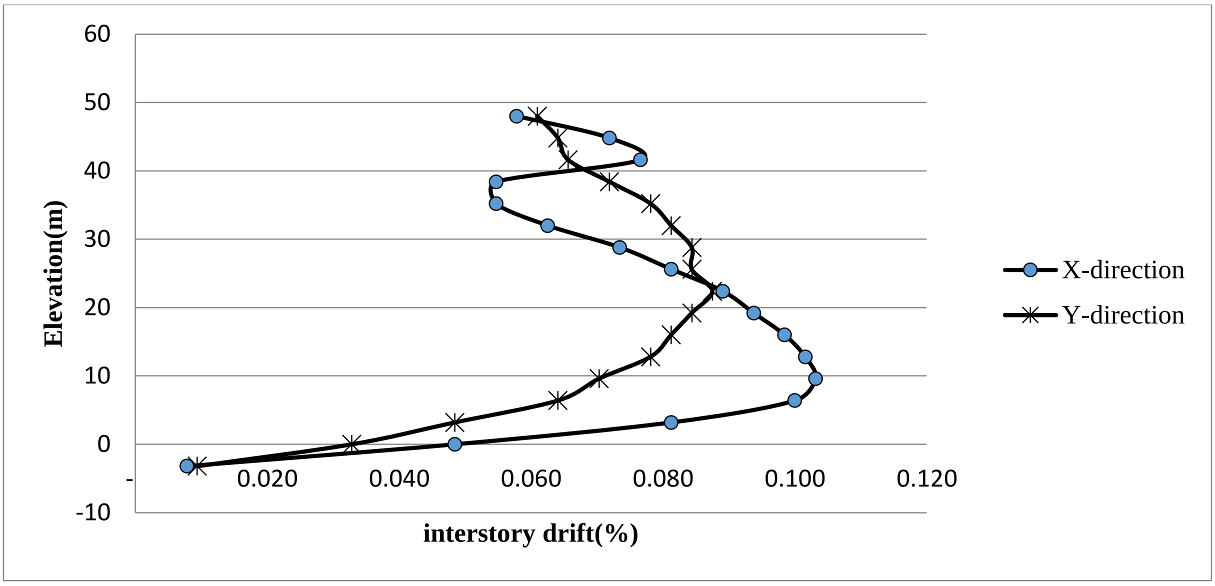

Figure 11 linear static elevation vs inter-story drift

3.2.2 SECOND-ORDER EFFECTS

To take into account the second-order effect in EBCS EN 2014, it is suggested to check the inter-story drift sensitivity coefficient. Second order effects must be taken into account when the coefficients are larger than 0.1 as discussed in chapter three. Table 6 presents the shears, gravity loads and story drifts at different floor levels. The inter-story drift sensitivity coefficients are also calculated. As the maximum inter-story drift sensitivity coefficient is 0.01956 which is below 0.1, therefore we can avoid second order effect.

$$\theta = \frac{P_{tot} x d_r}{V_{tot} x \ h} \qquad (11)$$

where

θ is inter-story drift sensitivity coefficient

Table 6 Inter-story drift sensitivity coefficients

Story	Height (m)	Shear (kN)		Gravity load(kN)	Story drift (dr/h) (%)		θ ratio	
		x-dir	y-dir		x-dir	y-dir	x-dir	y-dir
TTB	51.20	380.09	379.78	5868.03	0.0037	0.0039	0.01785	0.01883
15TH	48.00	1135.28	1134.58	20268.11	0.0046	0.0041	0.02566	0.02289
14TH	44.80	1847.44	1845.88	34668.30	0.0049	0.0042	0.02873	0.02465

Table 6 (Cont'd)

Story	Height (m)	Shear(kN)		Gravity load(kN)	Story drift(%)		θ ratio	
		x-dir	y-dir		x-dir	y-dir	x-dir	y-dir
13TH	41.60	2514.00	2513.14	49068.61	0.0035	0.0046	0.02135	0.02807
12TH	38.40	3136.61	3137.00	63469.05	0.0035	0.005	0.02213	0.03161
11TH	35.20	3717.53	3717.19	77869.58	0.004	0.0052	0.02618	0.03404
10TH	32.00	4255.21	4253.35	92270.20	0.0047	0.0054	0.03185	0.03661
9TH	28.80	4748.82	4745.10	106670.93	0.0052	0.0054	0.0365	0.03794
8TH	25.60	5197.82	5192.06	121071.74	0.0057	0.0056	0.04149	0.04081
7TH	22.40	5601.73	5593.77	135472.64	0.006	0.0054	0.04535	0.04087
6TH	19.20	5960.07	5949.76	149873.62	0.0063	0.0052	0.04951	0.04093
5TH	16.00	6272.33	6259.45	164274.66	0.0065	0.005	0.0532	0.04101
4TH	12.80	6537.90	6522.18	178675.76	0.0066	0.0045	0.05637	0.03852
3RD	9.60	6753.13	6735.96	192959.92	0.0064	0.0041	0.05715	0.0367
2ND	6.40	6904.83	6887.03	205562.85	0.0052	0.0031	0.04838	0.02892
1ST	3.20	7016.74	7003.93	219303.07	0.0031	0.0021	0.03028	0.02055
GR	0.00	7079.28	7083.58	234435.66	0.0005	0.0006	0.00517	0.00621
BSMT1	-3.20	7119.46	7122.37	250507.97	0.0002	0.0002	0.00235	0.00234

3.2.3 STORY SHEARS

Results of story shear and base moments are presented in Table 7 and 8. Figures 15 shows the distribution shear forces up the height of the building.

Table 7 Linear static story shear

Story	Elevation (m)	Story shear(kN)		Story	Elevation (m)	Story shear(kN)	
		X-Dir	Y-Dir			X-Dir	Y-Dir
TTB	51.20	-383.8	-383.4	7TH	22.40	-5603.1	-5593.3
15TH	48.00	-1139.4	-1138.4	6TH	19.20	-5960.7	-5947.9
14TH	44.80	-1850.4	-1849.5	5TH	16.00	-6271.9	-6256.0
13TH	41.60	-2516.0	-2516.7	4TH	12.80	-6535.4	-6517.1
12TH	38.40	-3140.2	-3140.4	3RD	9.60	-6746.2	-6726.2
11TH	35.20	-3721.4	-3720.2	2ND	6.40	-6893.2	-6877.3
10TH	32.00	-4258.9	-4255.8	1ST	3.20	-6994.3	-6990.9
9TH	28.80	-4752.0	-4746.8	GR	0.00	-7075.8	-7079.6
8TH	25.60	-5200.3	-5192.7	BSMT1	-3.20	-7116.5	-7117.5

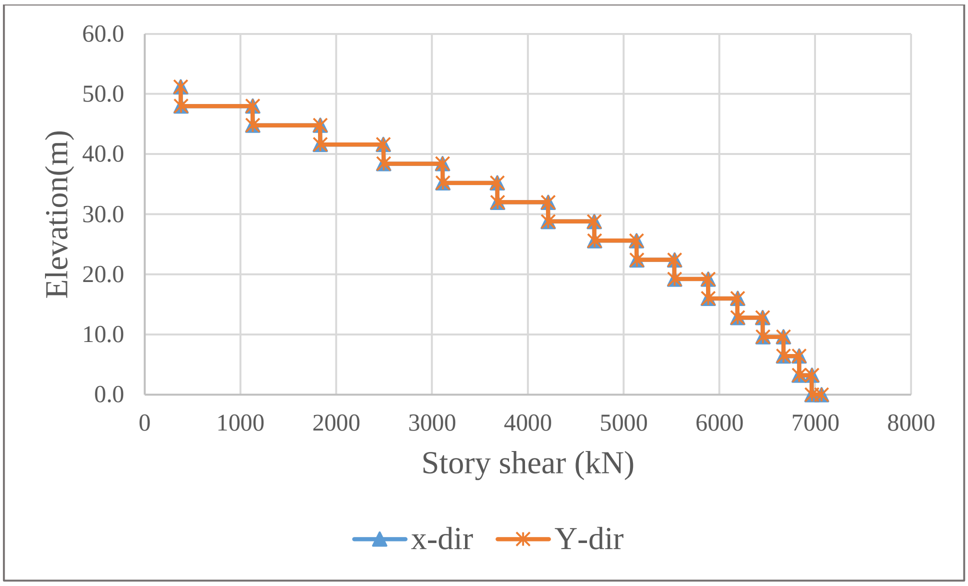

Figure 12 linear static Elevation Vs story shear

3.2.4 STORY MOMENT

The building has two basements and the shear wall is around the building thus the higher Column moment is at ground level. Table 9 shows the Column moments at Ground level for different load cases.

Table 8 Maximum beam moment on axis D/1-8

Load case	Floor	Beam	
		Axis	Maximum moment (kN-m)
EQx+	15th	Axis D/1-8	-117.26
EQx+	10th	Axis D/1-8	-150.58
EQx+	5th	Axis D/1-8	-142.49
EQx-	15th	Axis D/1-8	-110.12
EQx-	10th	Axis D/1-8	-139.17
EQx-	5th	Axis D/1-8	-130.46
EQy+	15th	Axis D/1-8	31.40
EQy+	10th	Axis D/1-8	28.16
EQy+	5th	Axis D/1-8	22.10
EQqy-	15th	Axis D/1-8	34.61
EQqy-	10th	Axis D/1-8	33.62
EQqy-	5th	Axis D/1-8	27.88

Table 9 Column maximum moment

	Case	Level	Column						
			C48	**C54**	**C58**	**C62**	**C60**	**C71**	**C73**
Maximum moment (kN-m)	EQx+	Ground	40.42	43.75	276.95	263.99	273.03	149.63	93.57
	EQx-	Ground	36.90	38.90	244.08	227.44	234.30	116.15	73.50
	EQy+	Ground	79.70	64.40	136.10	138.70	163.43	84.59	76.60
	EQqy-	Ground	102.20	48.15	78.20	122.00	168.84	95.51	72.20
	COMBO2	Ground	71.00	99.60	-112.34	19.75	6.30	50.38	25.85
Maximum(kN-m)			**102.20**	**99.60**	**276.95**	**263.99**	**273.03**	**149.63**	**93.57**

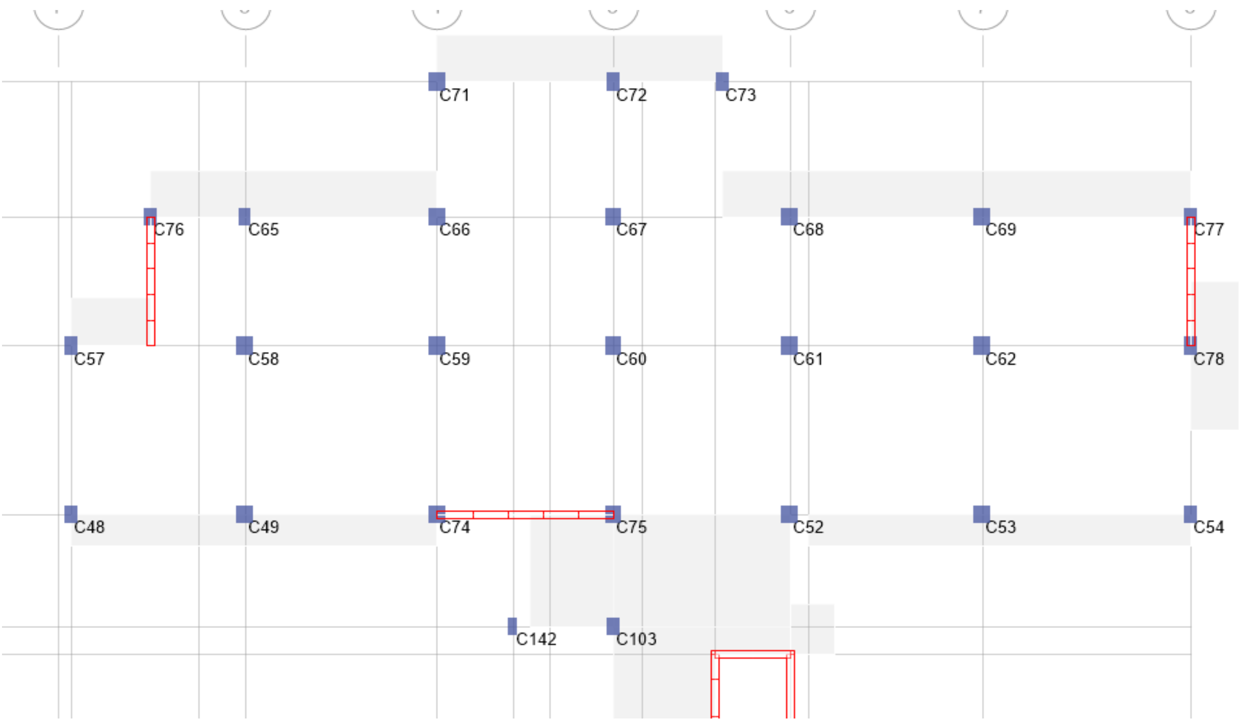

Figure 13 layout of column label

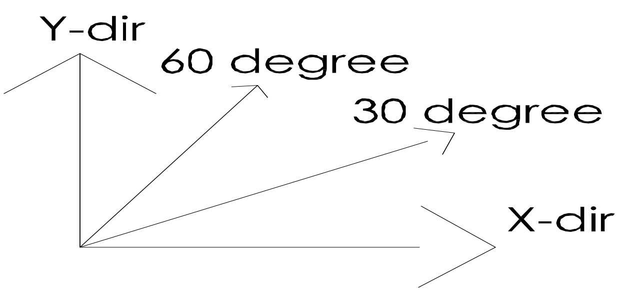

Figure 14 Earthquake direction

Table 10 Maximum beam moment on axis 5/A-D

Load case	Floor	Beam		Load case	Floor	Beam	
		Axis	Maximum moment(kN-m)			Axis	Maximum moment(kN-m)
EQx+	15th	5/A-D	33.96	EQy+	15th	5/A-D	76.00
EQx+	10th	5/A-D	49.52	EQy+	10th	5/A-D	-86.20
EQx+	5th	5/A-D	49.65	EQy+	5th	5/A-D	87.70
EQx-	15th	5/A-D	29.54	EQqy-	15th	5/A-D	79.60
EQx-	10th	5/A-D	41.55	EQqy-	10th	5/A-D	-92.31
EQx-	5th	5/A-D	41.44	EQqy-	5th	5/A-D	-91.53

3.2.5 AXIAL FORCES AND TORSION

Table 11 and table 12 shows axial force and torsion result of the original design

Table 11 Column maximum axial force and torsion

column	Load case	Axial force (kN)	Torsion (kN-m)	column	Load case	Axial Force (kN)	Torsion (kN-m)
C48	EQx+	553.73	-1.80		EQqy-	157.72	-0.39
	EQx-	383.92	-0.38		COMBO2	-8400.60	-0.00
	EQy+	398.20	0.47	C62	EQx+	-97.99	-0.15
	EQqy-	535.31	-0.66		EQx-	-53.46	0.15
	COMBO2	-3408.49	-0.22		EQy+	178.41	-0.70
C54	EQx+	-417.64	-0.09		EQqy-	142.45	-0.94
	EQx-	-244.82	-0.06		COMBO2	-8633.64	-0.00
	EQy+	559.22	-3.00	C71	EQx+	407.82	-1.85
	EQqy-	419.66	-5.00		EQx-	361.67	0.30
	COMBO2	-4344.31	-0.30		EQy+	-579.36	0.70
C58	EQx+	-180.63	-8.40		EQqy-	-542.10	-1.06
	EQx-	-294.60	-1.90		COMBO2	-4685.10	-0.25
	EQy+	574.90	3.45	C73	EQx+	-692.82	-1.09
	EQqy-	666.85	-1.80		EQx-	-519.18	0.22
	COMBO2	-5339.70	2.03		EQy+	-346.42	0.44
C60	EQx+	-155.07	-0.07		EQqy-	-486.63	-0.62
	EQx-	-148.40	0.07		COMBO2	-3813.73	1.02
	EQy+	163.13	-0.28				

Table 12 Beam torsion

Case	Beam Axsis	Floor	Torsion (KN-m)	Case	Beam Axsis	floor	Torsion (KN-m)
EQx+	D/1-8	15	-10.19	EQx+	5/A-D	15	-0.46
EQx-	D/1-8	15	-9.33	EQx-	5/A-D	15	-0.7
EQy+	D/1-8	15	-15.3	EQy+	5/A-D	15	-0.14
EQqy-	D/1-8	15	-14.47	EQqy-	5/A-D	15	0.12
COMBO2	D/1-8	15	-15.42	COMBO2	5/A-D	15	-1.3
EQx+	D/1-8	10	-12.62	EQx+	5/A-D	10	-0.46
EQx-	D/1-8	10	-11.8	EQx-	5/A-D	10	-0.55
EQy+	D/1-8	10	-18.3	EQy+	5/A-D	10	-0.1
EQqy-	D/1-8	10	17.51	EQqy-	5/A-D	10	-0.02
COMBO2	D/1-8	10	18.04	COMBO2	5/A-D	10	-1.4
EQx+	D/1-8	5	-12.07	EQx+	5/A-D	5	-1.31
EQx-	D/1-8	5	-11.29	EQx-	5/A-D	5	-1.33
EQy+	D/1-8	5	-18.52	EQy+	5/A-D	5	0.003
EQqy-	D/1-8	5	-17.9	EQqy-	5/A-D	5	0.02
COMBO2	D/1-8	5	8.2	COMBO2	5/A-D	5	-1.17

3.2.6 STORY STIFFNESS

The building story stiffness variation along the height of the building is shown on table13 and figure 15.

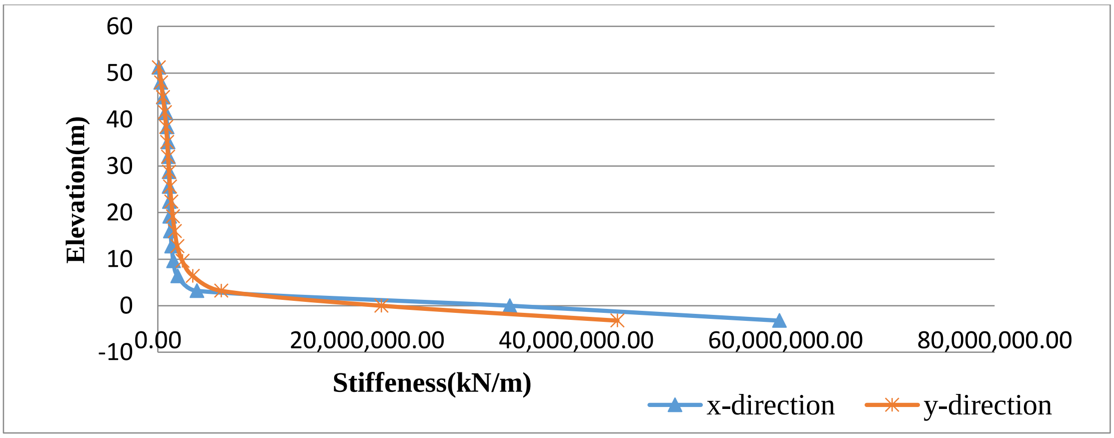

Figure 15 story stiffness for linear static case

Table 13 Maximum story stiffness

Story	Elevation (m)	X-direction (kN/m)	Y-direction (kN/m)	Story	Elevation (m)	X-direction (kN/m)	Y-direction (kN/m)
TTB	51.2	1.1 x10^5	1.1 x10^5	7th	22.4	1.1 x10^6	1.3x10^6
15th	48.0	2.9 x10^5	3.1 x10^5	6th	19.2	1.2 x10^6	1.4 x10^6
14th	44.8	5.2 x10^5	4.9 x10^5	5th	16.0	1.2 x10^6	1.6 x10^6
13th	41.6	7.4 x10^5	6.6 x10^5	4th	12.8	1.3 x10^6	1.8 x10^6
12th	38.4	8.9 x10^5	7.9 x10^5	3rd	9.6	1.5 x10^6	2.3 x10^6
11th	35.2	9.7 x10^5	8.9 x10^5	2nd	6.4	1.9 x10^6	3.3 x10^6
10th	32.0	1.1 x10^5	9.8 x10^5	1st	3.2	3.7 x10^6	6.1 x10^6
9th	28.8	1.1 x10^5	1.1 x10^5	ground	0	33.6 x10^6	21.4 x10^6
8th	25.6	1,088,065.25	1,170,992.05	BSM1	-3.2	59.4 x10^6	43.9 x10^6

3.3 ACCELERATION TIME HISTORY

To investigate whether the structure would satisfy EBCS EN 2014 design limit, inelastic time-history analyses are carried out using ETABS 2016 analysis and design software to assess the likely inelastic deformation, inter-story drift, shear, moments, Axial force & torsion.

3.3.1 MODAL PROPERTIES

Before running the nonlinear time history [N.L.T.H] analyses, Eigen-value analyses were conducted to establish the various modes of vibration of the case study structure.

Table 14 Various modes of vibration of the case study structure.

No of mode	Period(T) in second	No of mode	Period(T) in second
mode 1	2.235	mode 11	0.380
mode 2	2.201	mode 12	0.372
mode 3	1.981	mode 13	0.344
mode 4	0.843	mode 14	0.334
mode 5	0.832	mode 15	0.330
mode 6	0.554	mode 16	0.317
mode 7	0.501	mode 17	0.307
mode 8	0.498	mode 18	0.306
mode 9	0.419	mode 19	0.300
mode 10	0.417		

From the table the fundamental period of the building is **2.235sec.**

3.3.2 REFERENCE EARTH QUAKE

The reference Earth quake used in generating synthetic time history is shown in table 15. A minimum of 3 accelerograms should be used (ES EN 1998-1:2015). Although the case study building is in Ethiopia because of data limitation we use El-centro 1940 and Sierra-madre earth quake in addition to Ankober earth quake.

Table 15 Reference earthquake records.

Earth quake name	**Mag nitude**	**Date**	**Strong motion Duration**	**Station**	**time step**	**peak acceleration**	**Remark**
Ankober	4.6	Dec-4 2016	5.50sec	Furi	0.01	0.00324g	
Imperial Valley,California earthquake	7.1	May 18 1940	30.00 sec	EL Centro	0.02	0.31900g	North south component
Sierra Madre	5.6	Jun 28 1991	7.04sec	Altadena-Eaton Canyon Park	0.02	0.44740g	

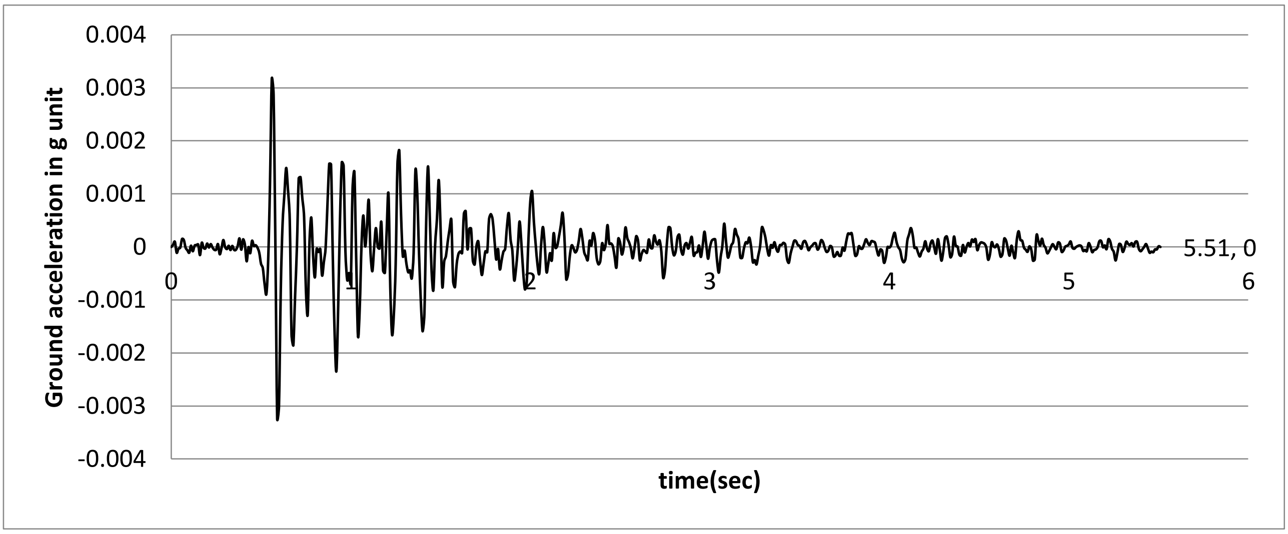

Figure 16 Ankober 2016 ground motion

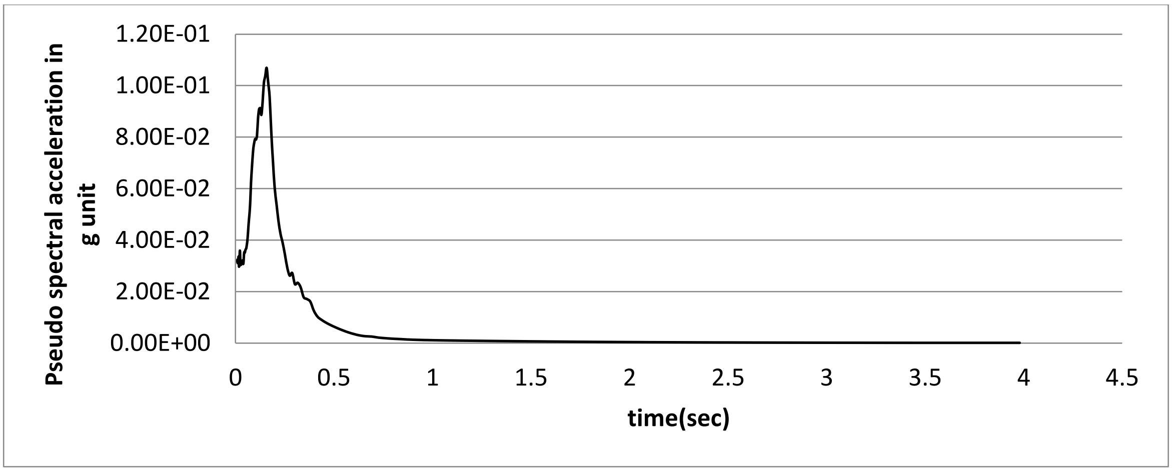

Figure 17 Ankober 2016 Pseudo spectral acceleration Vs time

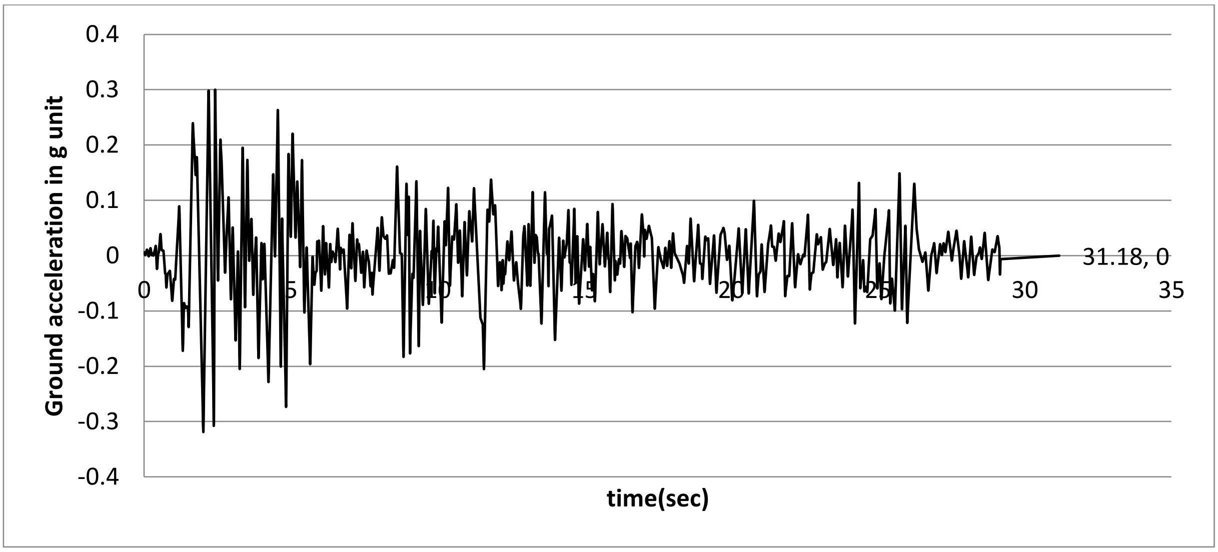

Figure 18 EL-Centro 1940 ground motion

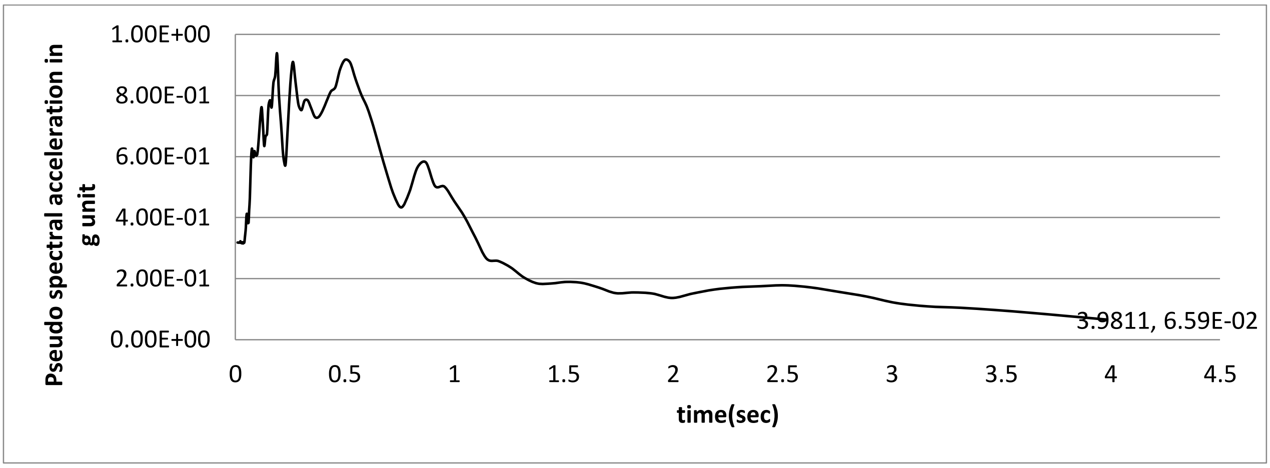

Figure 19 EL-Centro 1940 response spectrum

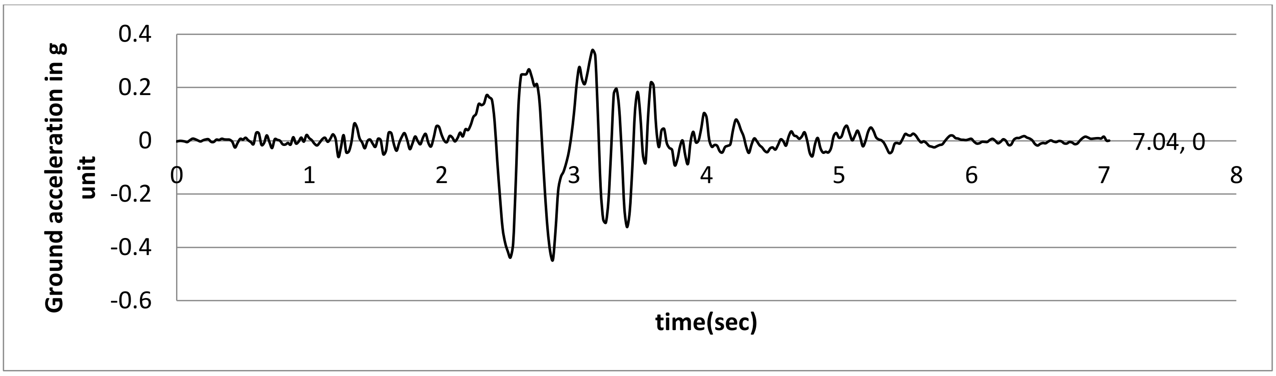

Figure 20 Sierra Madre 1991 ground motion

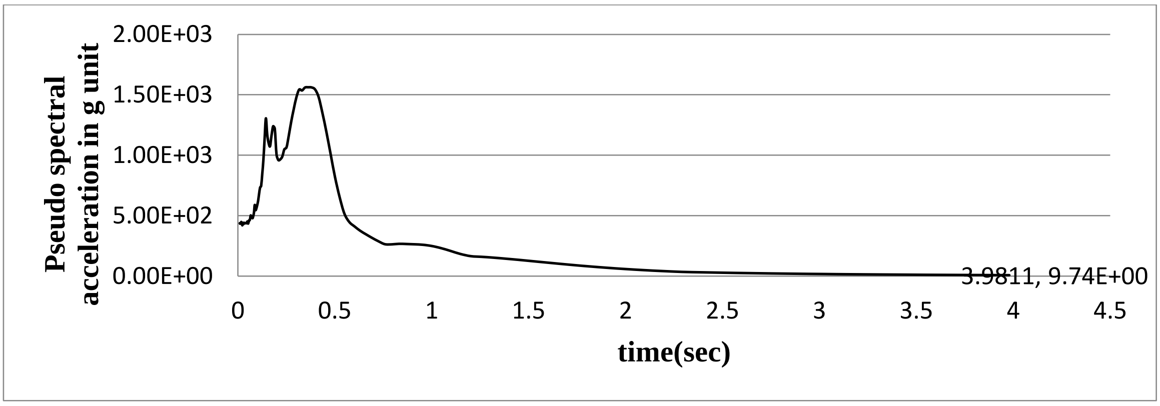

Figure 21 Sierra Madre 1991 response spectrum

3.3.3 TARGET RESPONSE SPECTRUM

Type 1 response spectrum is used for target spectrum. (EBCS-8, 2014) Addis Ababa is zone 3. (EBCS-8, 2014). The bedrock acceleration is given in table 17. (EBCS-8, 2014)

Table 16 Values of the parameters describing the recommended type 1 elastic response spectra.

Ground type	**S**	**TB(s)**	**TC(s)**	**TD(s)**
D	1.8	0.1	0.3	1.2

Table 17 Bedrock acceleration ratio.

Zone	**3**
$\alpha 0 = a_g/g$	0.1

$a_g = 0.1 * 9.81 = 0.981$

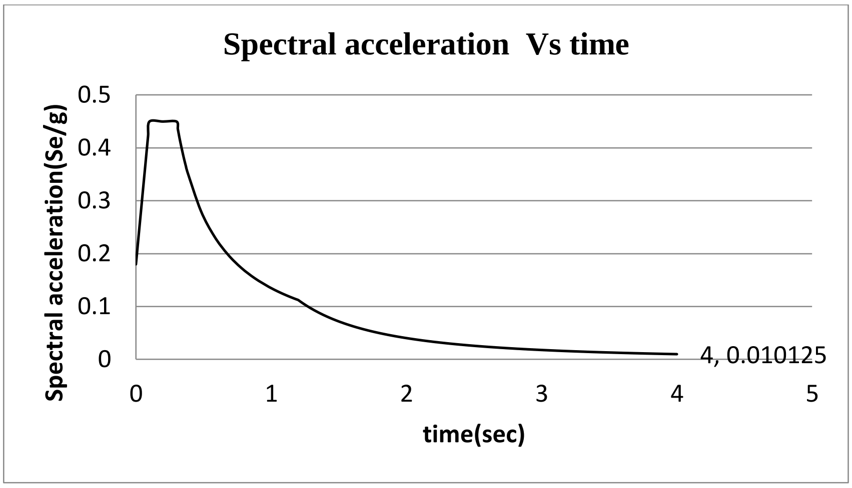

Figure 22 Target horizontal elastic response spectrum

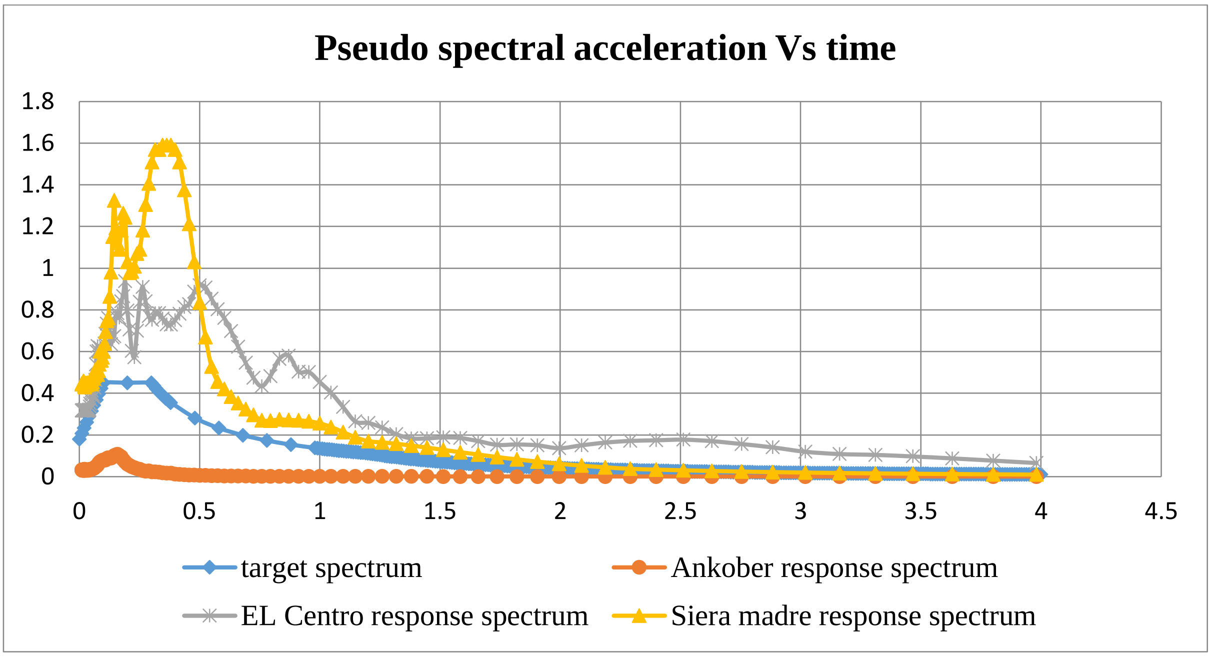

Figure 23 Reference earth quake response spectrum and target response spectrum

3.3.4 SYNTHETIC TIME HISTORY

Matching to target response spectrum is done using time domain method since this method is generally more complicated than the frequency domain approach; it has good convergence properties and in most cases preserves the non-stationary character of the reference time series.

Seismic analysis by direct integration analysis approaches requires a series of time histories as an input. However, in most cases, the possibility of using real earthquake data is limited. Thus, artificial time histories are widely used instead. In many cases, however, response spectra are given. Thus, most of the artificial time histories are generated from the given response spectra. Obtaining the response spectrum from a given time history is straightforward. However, the procedure for generating artificial time histories from a given response spectrum is difficult and complex to understand.

Thus, this work uses a time-domain method for generating a time history from a given response spectrum using ETABS 2016 Analysis and design software.

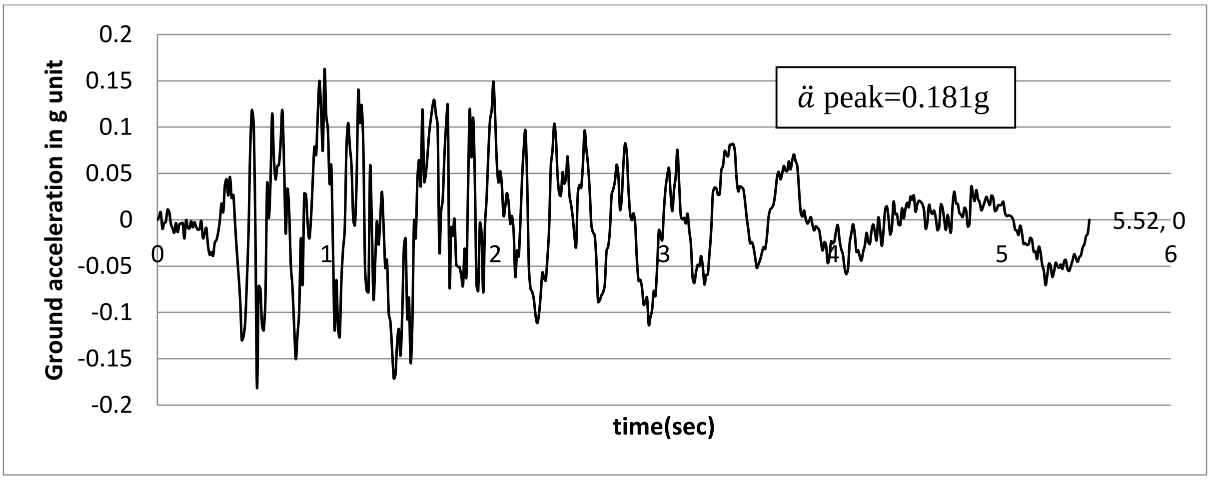

Figure 24 Ankober time history matched to target type 1 RS

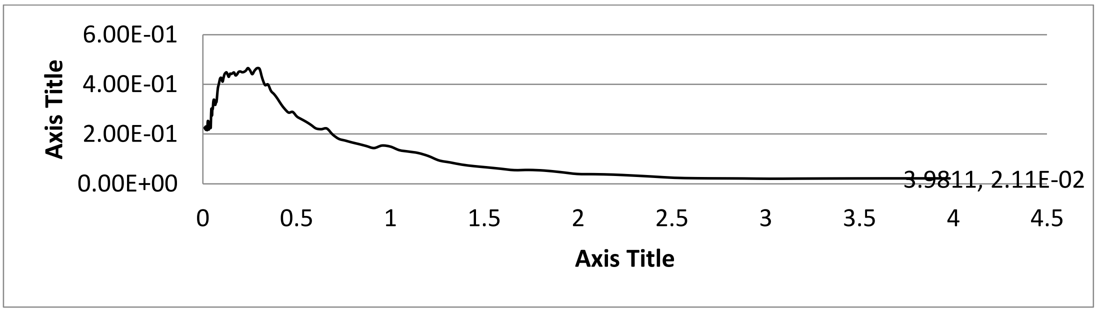

Figure 25 The corresponding response Spectrum

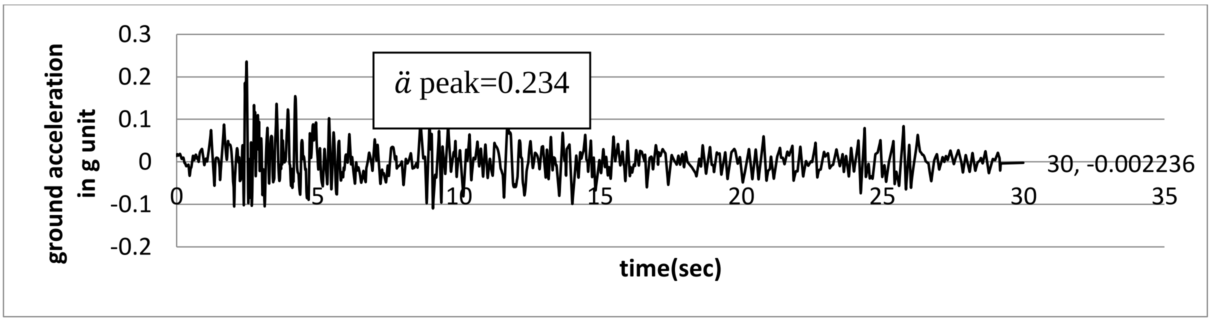

Figure 26 EL-Centro 1940-time history matched to target RS

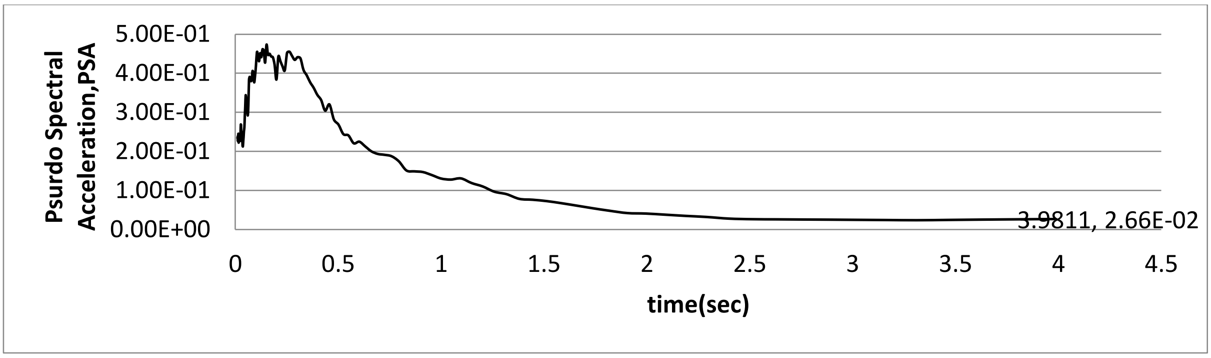

Figure 27 The corresponding response Spectrum

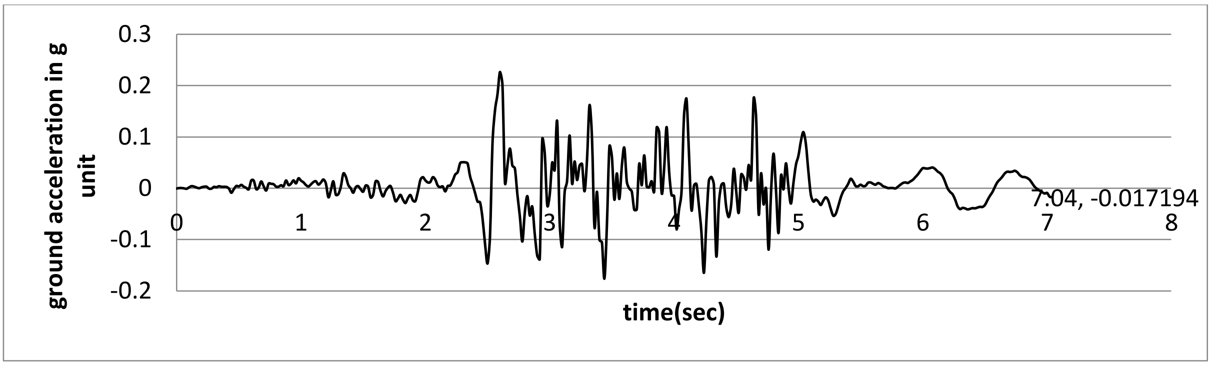

Figure 28 Sierra Madre time history matched to target type 1 RS

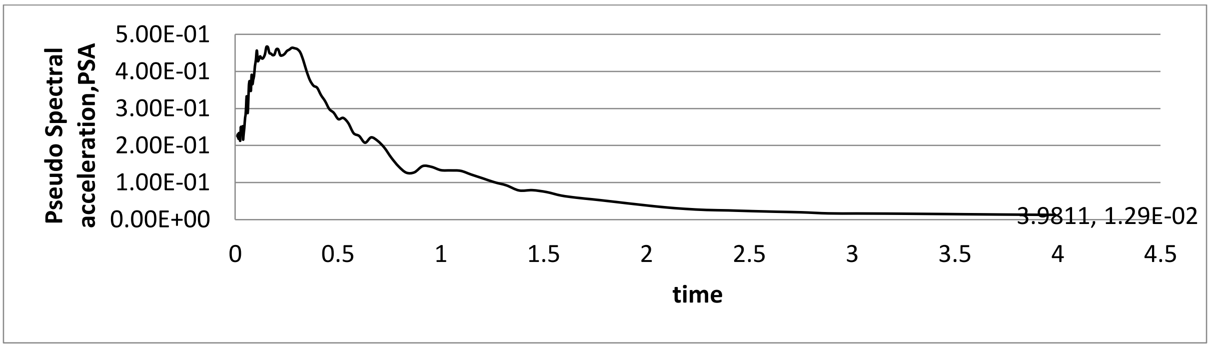

Figure 29 The corresponding response Spectrum

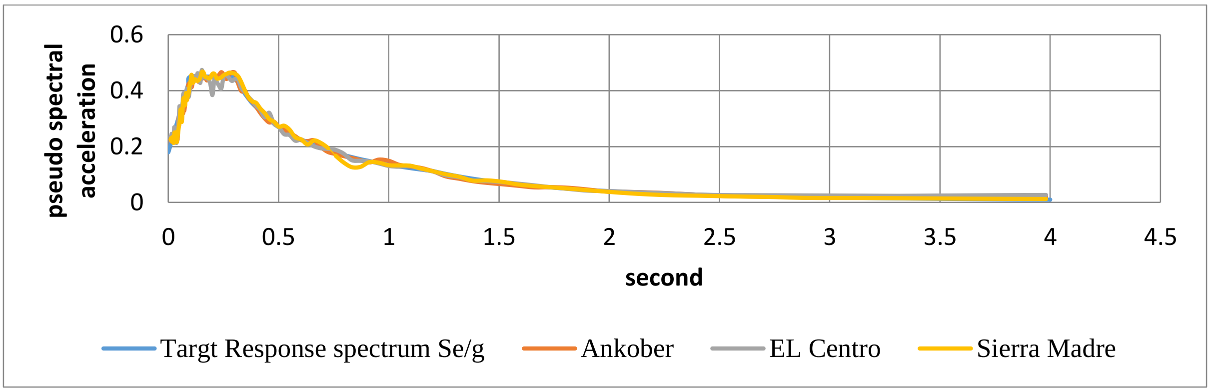

Figure 30 Target and synthetic earth quake Response spectrum

3.4 NONLINEAR TIME-HISTORY ANALYSIS RESULTS

3.4.1 MAXIMUM STORY DISPLACEMENT

From the results, it can be seen that the non-linear displacement response of the structure is smaller than the code response predicted by elastic analysis. The displacement shape is relatively linear and the inter-story drift is less than 0.005. The designed structure satisfied the code requirements on structural deformations.

Table 18 Maximum time history displacement

	Story	Elevation (m)	TH load case		
			Ankober	**EL-Centro**	**Sierra-Madre**
Story displacement(mm)	TTB	51.20	42.90	26.10	22.05
	15TH	48.00	39.80	24.75	20.25
	14TH	44.80	36.60	22.95	18.90
	13TH	41.60	33.50	21.60	17.55
	12TH	38.40	30.30	20.25	16.65
	11TH	35.20	27.00	19.35	16.20
	10TH	32.00	23.60	18.00	15.30
	9TH	28.80	20.20	16.20	14.40
	8TH	25.60	16.80	14.40	12.60
	7TH	22.40	13.50	12.60	10.80
	6TH	19.20	10.30	10.35	8.55
	5TH	16.00	7.40	8.55	6.30
	4TH	12.80	4.90	6.30	4.50
	3RD	9.60	2.80	4.50	2.70
	2ND	6.40	1.30	2.70	1.35
	1ST	3.20	0.30	1.35	0.45
	GR	0.00	0.20	0.45	0.45
	BSMT1	-3.20	0.10	0.20	0.15
	BASE	-6.20	0.00	0.00	0.00

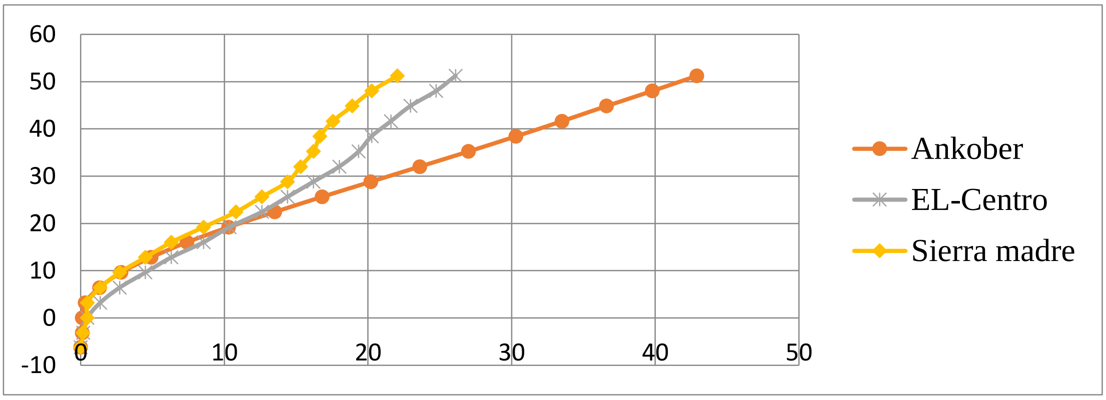

Figure 31 Time history Elevation Vs maximum deformation

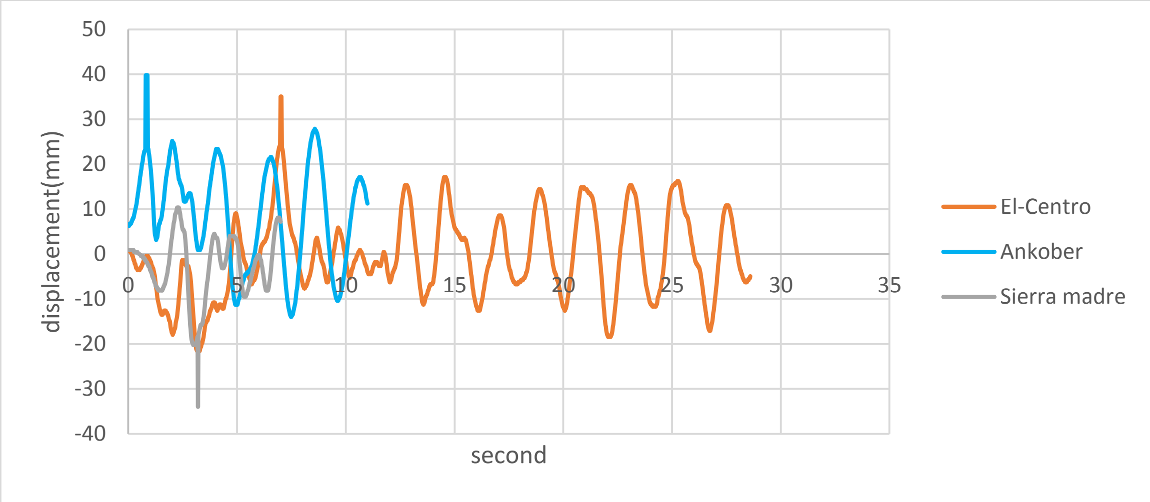

Figure 32 15th story TH story displacement response at joint label 1237

3.4.2 INTER STORY DRIFT

Displacement demand normally be governed by code drift limit. The maximum inter-story drift for different TH cases is presented in table 19.

Table 19 Time history maximum inter-story drift

	Story	Elevation (m)	TH load case		
			Ankober	**EL-Centro**	**Sierra-Madre**
Inter-story drift	TTB	51.20	0.000728	0.000603	0.000513
	15TH	48.00	0.000753	0.00063	0.000513
	14TH	44.80	0.00082	0.000738	0.000531
	13TH	41.60	0.000859	0.000774	0.000621
	12TH	38.40	0.000895	0.00081	0.000657
	11TH	35.20	0.000921	0.000846	0.000702
	10TH	32.00	0.000934	0.000873	0.000729
	9TH	28.80	0.00093	0.000891	0.000765
	8TH	25.60	0.000908	0.000891	0.000792
	7TH	22.40	0.000929	0.00104	0.00081
	6TH	19.20	0.00097	0.00126	0.000846
	5TH	16.00	0.000965	0.00146	0.001053
	4TH	12.80	0.000855	0.00162	0.001287
	3RD	9.60	0.000601	0.00146	0.001512
	2ND	6.40	0.000429	0.000927	0.001413
	1ST	3.20	0.0000429	0.000045	0.00090
	GR	0.00	0.000009	0.000018	0.000045
	BSMT1	-3.20	0.000000	0.000000	0.000002
	BASE	-6.20	0.000000	0.000000	0.000000

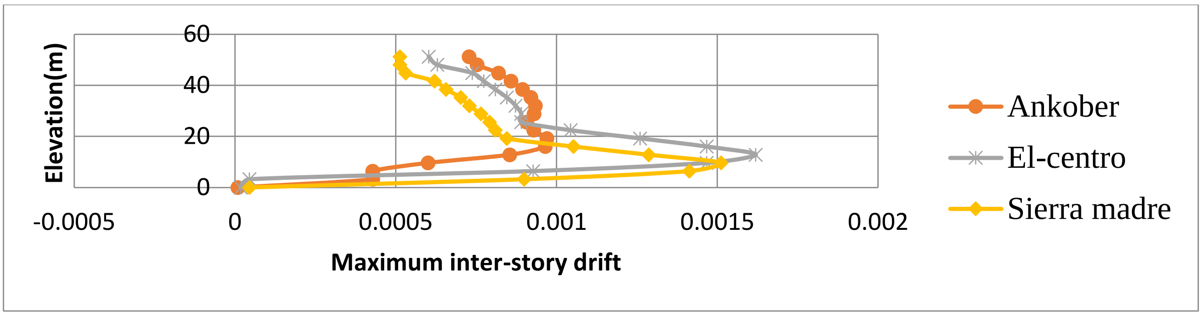

Figure 33 Time history maximum inter-story drift

3.4.3 STORY SHEAR

From the result of time history analysis, the story shear varies with time. Result of typical maximum story shear for different time is shown below in table and graph.

Table 20 Ankober time history for different time

	Story	Elevation(m)	Ankober TH load case		
			1.3 sec	2.6 sec	5.4 sec
Average Story shear(kN)	TTB	51.20	29.4769	-10.6952	2.2340
	15TH	48.00	77.0856	-23.9616	9.64945
	14TH	44.80	110.3976	-28.0508	19.7101
	13TH	41.60	128.4596	-23.4287	32.7259
	12TH	38.40	141.5329	-19.6188	49.9795
	11TH	35.20	150.1419	-21.0473	72.3550
	10TH	32.00	151.8421	-28.6229	100.2150
	9TH	28.80	145.4704	-42.0546	133.4818
	8TH	25.60	130.7281	-60.1761	171.6541
	7TH	22.40	106.0323	-80.7527	213.8003
	6TH	19.20	66.05835	-101.113	258.6221
	5TH	16.00	0.6632	-120.675	304.6325
	4TH	12.80	-102.4290	-145.141	350.3597
	3RD	9.60	-248.5810	-188.347	394.3878
	2ND	6.40	-420.8340	-264.827	433.178
	1ST	3.20	-620.9410	-379.789	473.8117
	GR	0.00	-852.1640	-531.976	520.6190
	BSMT1	-3.20	-1062.600	-673.824	564.2419
	BASE	-6.20	-1569.550	-654.61	700.0000

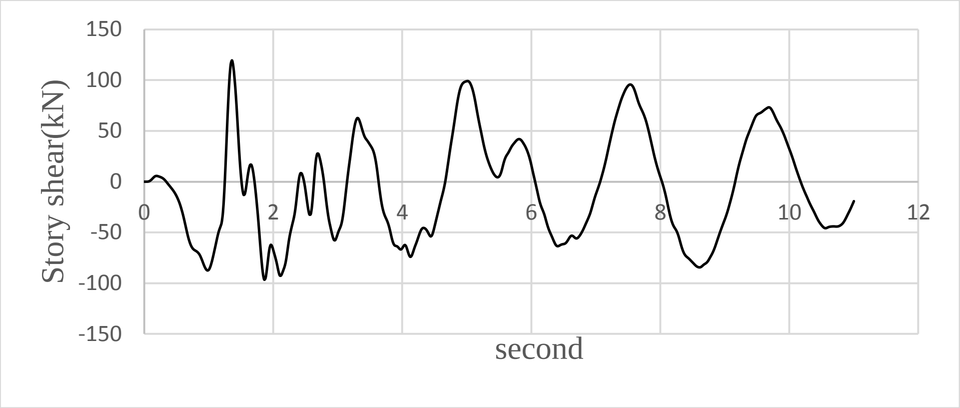

Figure 34 15th Story Ankober TH story shear Vs time

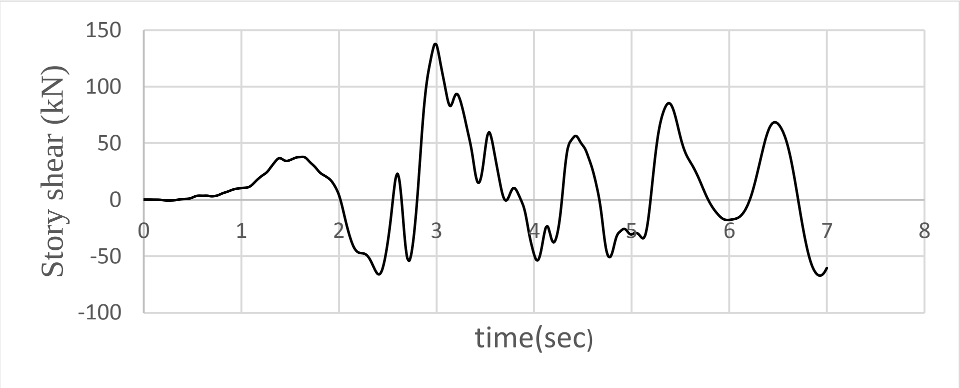

Figure 35 15th Story Sierra Madre TH story shear Vs time

Table 21 Time history maximum story shear

Story shear(kN)			
Elevation(m)	**Ankober**	**EL-Centro**	**Siera-madre**
51.2	408.52	340.814	220.591
48.0	739.57	341.556	221.78
48.0	740.6907	596.418	515.513
44.8	1112.638	597.281	516.19
44.8	1113.896	871.34	641.651
41.6	1523.892	872.361	641.842
41.6	1525.233	1160.51	593.074
38.4	1968.527	1161.525	592.775
38.4	1969.948	1465.198	488.569
35.2	2438.663	1466.235	487.64
35.2	2440.123	1793.687	350.929
32.0	2922.612	1794.745	349.319
32.0	2924.06	2160.386	170.606
28.8	3403.948	2161.464	168.314

Table 21(Continued)

28.8	3405.327	2580.772	46.728
25.6	3860.831	2581.882	49.645
25.6	3862.09	3062.07	289.141
22.4	4266.882	3063.247	292.545
22.4	4267.972	3596.488	571.413
19.2	4594.976	3597.778	575.061
19.2	4595.856	4163.83	961.737
16.0	4824.184	4165.266	965.319
16.0	4824.812	4742.877	1568.806
12.8	4945.802	4744.469	1572.095
12.8	4946.111	5319.047	2471.553
9.6	4971.117	5320.826	2474.74
9.6	4971.165	5858.592	3624.085
6.4	4920.511	5860.556	3627.81
6.4	4920.246	6447.106	4812.255
3.2	4390.452	6449.532	4816.927
3.2	4700.054	7128.467	6078.805
0.0	4700.174	7152.68	6084.954

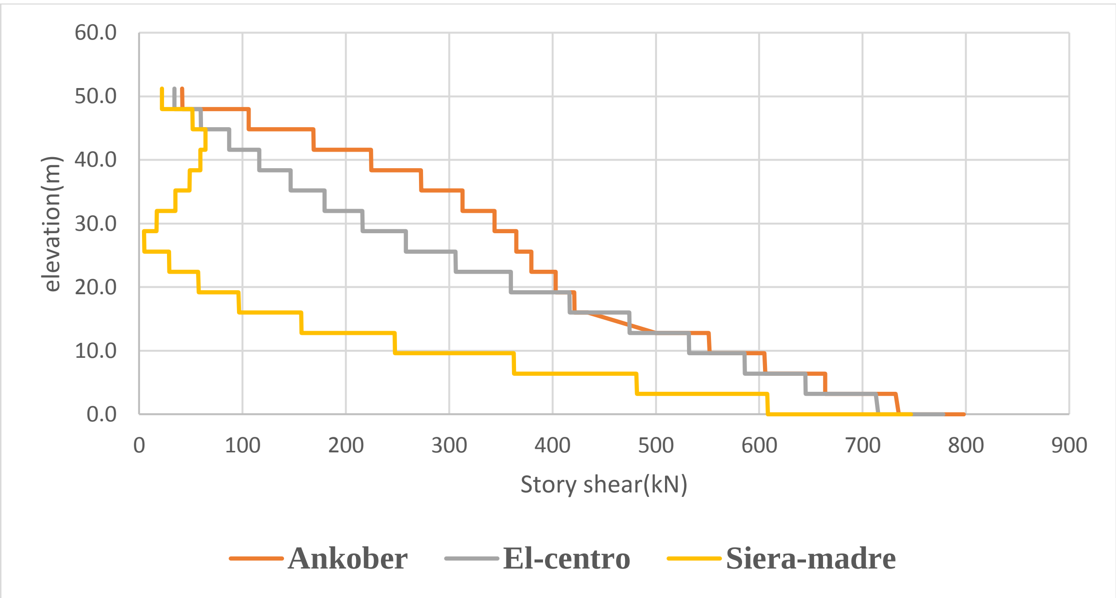

Figure 36 Time history Story Shear Vs elevation for different reference earth quake

3.4.4 STORY MOMENT

Result of column and beam moment at some typical story level is shown in table 22 and table 23.

Table 22 Maximum column moment for time history case

	Time history load case	Level	Column						
			C48	C54	C58	C60	C62	C71	C73
Maximum moment (kNm)	Ankober	ground	-37.3	54.7	-81.6	24.2	26.7	26.1	-17.0
	EL-Centro	ground	19.5	23.6	47.4	28.1	29.2	11.8	13.2
	Sierra madre	ground	19.4	22.1	-48.0	27.8	29.6	11.7	13.1

Table 23 Maximum beam moment for time history case

	Floor	Beam axis	TH load case		
			Ankober	EL-Centro	Sierra-Madre
Maximum moment (kNm)	15th	Axis D/1-8	-164.9	45.8	-45.5
	10th	Axis D/1-8	-161.8	47.6	-46.0
	5th	Axis D/1-8	-142.4	42.0	-40.3
	15th	Axis 5/A-D	-105.6	19.5	-20.6
	10th	Axis 5/A-D	-97.8	19.8	-20.4
	5th	Axis 5/A-D	-68.5	15.43	-15.3

3.4.5 AXIAL FORCES AND TORSION

Axial forces for typical columns and torsion for typical column and beam is shown in table 24 and table 25.

Table 24 Column axial forces and torsion

Column	Ankober TH		EL-Centro TH		Sierra-Madre TH	
	Axial forces(kN)	Torsion (kN-m)	Axial forces(kN)	Torsion (kN-m)	Axial forces(kN)	Torsion (kN-m)
C48	-2303.04	-0.34	830.00	0.20	-827.58	0.08
C54	-2844.28	-0.66	995.10	0.35	-979.10	-0.32
C58	-3526.67	1.87	1126.80	1.42	-1154.44	1.38
C60	-5225.41	0.92	1339.40	0.84	-1329.70	0.81
C62	-5235.99	0.91	1458.70	0.71	-1455.14	0.68
C71	-3128.83	-0.20	1133.00	0.08	-1137.70	0.08
C73	-2591.73	0.70	942.40	0.30	-912.40	0.29

Table 25 Time history maximum beam torsion

Beam Axis	Floor	Maximum Torsion(kN-m)		
		Ankober TH	EL-Centro TH	Sierra-Madre TH
D/1-8	15th	-9.81	4.80	-4.30
D/1-8	10th	9.42	4.80	4.70
D/1-8	5th	-8.98	4.80	4.60

3.4.6 RESULT OF SWAPPING GROUND MOTION

The direction of earthquake is not known exactly. Therefore, time history analysis was performed in different direction to get the maximum effect of earthquake. Table 26 shows axial force and torsion result for 0^0, 30^0, 60^0 and 90^0 ground motion.

Table 26 Different direction time history response

	X-dir TH		30 degree-dir TH		60 degree-dir TH		Y-dir TH	
	Axial forces(kN)	Torsion (kN-m)	Axial force(kN)	Torsion (kN-m)	Axial force(kN)	Torsion (kN-m)	Axial force(kN)	Torsion (kN-m)
C48	-2303	-0.3	-2315	-0.3	-2316	-0.2	-2304	-0.2
C54	-2844	-0.6	-2840	-0.7	-2856	-0.7	-2871	-0.6
C58	-3526	1.8	-3517	1.9	-3538	1.8	-3551	1.7
C60	-5225	0.9	-4775	1.0	-4777	0.9	-4783	0.8
C62	-5235	0.9	-5235	1.0	-5241	0.9	-5245	0.8
C71	-3128	-0.2	-3117	-0.2	-3127	-0.2	-3147	-0.2
C73	-2591	0.7	-2602	0.7	-2598	0.7	-2580	0.7

3.4.7 EFFECT OF FORCE DIRECTION

Due to the randomness of earthquake wave magnitude and direction, and the uncertain direction of strong axis and weak axis in the construction of Engineering structures, the effect of ground motion direction on a structure are studied herein. The characteristics of the ground motion time history and response spectrum of each group were studied.

The seismic response of structures with different directions of ground motion inputs has been analyzed under the same earthquake record, and the results show the difference.

Table 27 Time history maximum story displacement, maximum story drift and maximum story shear

	Direction	TH response
Maximum displacement (mm)	x-direction	6.50000
	30 degree response	5.30000
	60 degree response	4.60000
	y-direction	5.40000
Maximum story drift	x-direction	0.00016
	30 degree response	0.00014
	60 degree response	0.00013
	y-direction	0.00015
Maximum story shear(kN)	x-direction	-1569.55000
	30 degree response	924.44000
	60 degree response	-1039.00000
	y-direction	-929.24000

3.5 DISCUSSION AND COMPARISON

3.5.1 STORY DISPLACEMENT

From the result, when we compare with linear elastic analysis (original design result) the time history case gives smaller result. it can be seen that even though the reference earth quake is different if we generate synthetic earth quake for same target response spectrum the result will be almost the same.

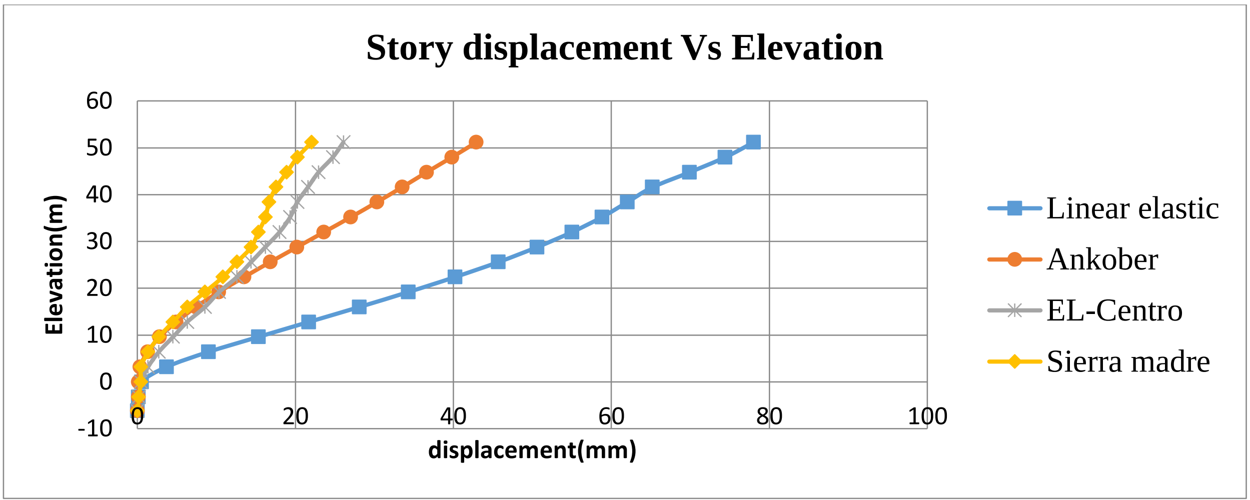

Figure 37 Time history Vs linear elastic story displacement

3.5.2 INTERSTORY DRIFT

The maximum inter-story drift $\frac{d_r v}{h} = 0.00103$ for linear elastic and 0.00018 for time history case (EL-centro) which is less than 0.005(for building having non-structural elements of brittle materials attached to the structure, EBCS-8, Part 1, 2014) which satisfy the code limit for inter story drift. But when we compare original design and time history $=\frac{0.00103}{0.00018} = 5.72$ the linear elastic gives on average 5.72 times higher result.

Figure 37 shows the maximum inter-story drifts for elastic and time history along the height of the building.

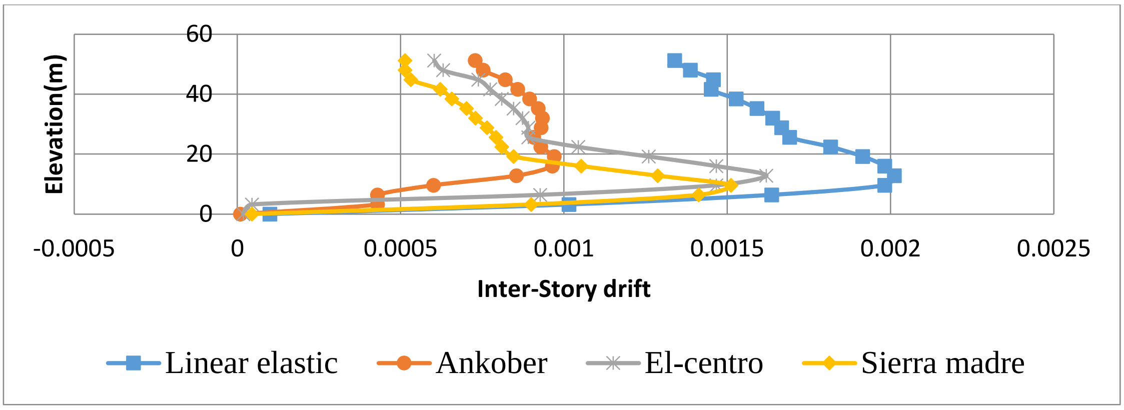

Figure 38 TH Vs linear elastic (original design) maximum inter-story drift

3.5.3 STORY SHEAR

Results from non-linear time history analyses show that the shear force were much smaller than predicted by elastic analysis, about 85%. Comparison with the original design is presented on figure 38.

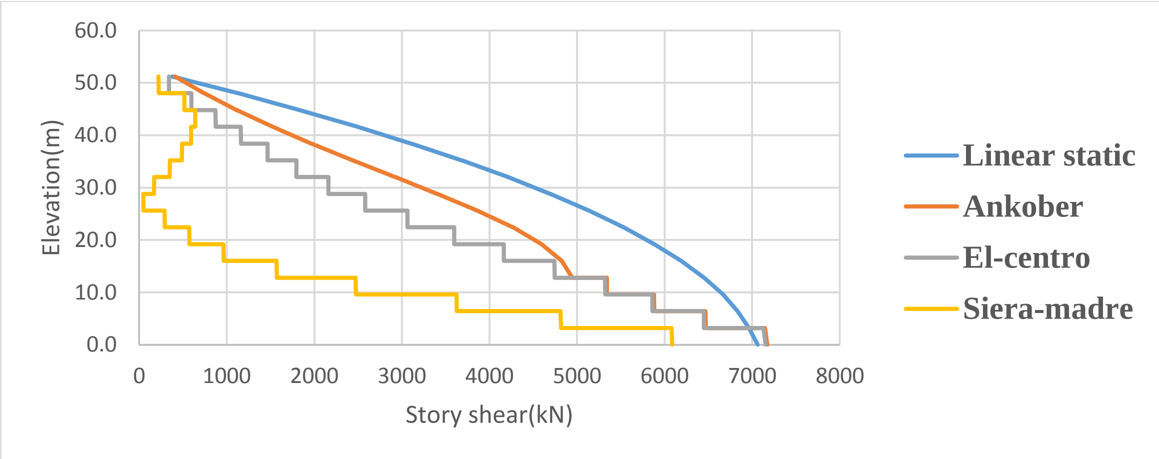

Figure 39 Maximum story shear for TH Vs linear elastic case

3.5.4 COLUMN MOMENT

The linear elastic column moment is on average six times higher than predicted by time history analysis.

Table 28 Column maximum moment for TH Vs linear elastic analysis

Case	Column maximum moment(kNm)						
	C48	**C54**	**C58**	**60**	**C62**	**C71**	**C73**
Linear elastic	102.2	99.6	277.0	273.0	264.0	149.6	93.6
Ankober time history	3.7	3.5	22.8	22.0	21.0	12.3	8.0
El-centro time history	19.5	23.6	47.4	28.2	29.0	11.8	13.2
Sierra madre time history	19.4	22.1	-48.0	28.0	30.0	11.7	13.1

3.5.5 BEAM MOMENT

From the result, it can be seen that even though on some point the non-linear time history analysis beam moment is higher than the linear elastic analysis most of the beam moment of linear elastic case gives higher result.

Table 29 Maximum beam moment for TH Vs linear elastic analysis

Load case	Floor	Maximum beam moment				
		Axis	Linear elastic	Ankober TH	El-centro TH	Sierra-madre TH
Maximum beam moment (kNm)	15th	Axis D/1-8	-40.34	8.40	45.84	-45.45
	10th	Axis D/1-8	-56.99	10.70	47.64	-46.00
	5th	Axis D/1-8	-55.74	9.62	42.00	-40.30
	15th	Axis 5/A-D	54.78	2.40	19.50	-20.60
	10th	Axis 5/A-D	-21.86	3.40	19.80	-20.40
	5th	Axis 5/A-D	21.82	3.35	15.43	-15.30

3.5.6 AXIAL FORCE

The axial load on typical columns is shown in table 30. The non-linear time history analysis result gives higher axial force than linear elastic analysis but it is much lower than the self-weight load case.

Table 30 Column maximum axial force

Ground floor column	Maximum axial force for different load case			
	Linear elastic	Time history load case		
		Ankober	EL-Centro	Sierra madre
C48	553	34	830	827
C54	559	24	995	979
C58	666	17	1126	1154
C60	163	11	1339	1329
C62	178	5	1458	1455
C71	579	27	1133	1137
C73	692	44	942	912

3.5.7 TORSION

The Torsion obtained for all load case is small so that it is not a critical force in the analysis.

Table 31 Column maximum torsion

Column	Maximum torsion for different load case			
	Linear elastic(kNm)	Time history load case torsion(kNm)		
		Ankober	EL-Centro	Sierra madre
C48	-1.80	-0.58	0.20	0.08
C54	-5.00	-0.60	0.35	-0.32
C58	-8.4	-0.93	1.42	1.38
C60	-0.39	-0.97	0.84	0.81
C62	-0.94	-0.95	0.71	0.68
C71	-1.85	0.08	0.08	0.08
C73	-1.09	0.04	0.30	0.29

CHAPTER FOUR

CONCLUSION

4.1 SUMMARY OF FINDING

The major advantage of using the forces obtained from a non-linear time history 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. Compared to original design, the non-linear time-history case gives result smaller than those obtained from original design on story displacement, inter-story drift, story shear, column moment and torsion.

TH analysis was performed in different direction by swapping ground motion data. Since the analysis is computationally expensive only four directions are analyzed. The different direction earthquake gives different result and the maximum of which is used as bases for comparison to the original design.

While nonlinear time history analyses provide more realistic measures of response than other methods, the reliability of nonlinear time history analyses is sensitive to modeling assumptions and parameters.

Damage can be directly related to deformation. And the deformation and drift demand obtained by nonlinear time-history is based on more realistic member stiffness.

The designed structure satisfied the code requirements on structural deformations and inter-story drift. The torsion obtained is small so it has no significant effect.

The case study building is a Mega structure and economical earthquake resistance design is achieved by performing non-linear TH analysis by allowing yielding to take place in some structural members with the provisions that the vertical load-carrying capacity of the structure is maintained even after strong earth-quake. Using non-linear time history have great economic benefit by reduction of cross-section and reinforcement but attention should be given to detailing of members. Appropriately detailed members possess the necessary characteristics to dissipate energy by inelastic deformations.

4.2 RECOMMENDATION

- The case study building is a mega project and if the designer use time history method in the design it will reduce cost and produce more safe design.

- Although the non-linear analysis gives better result of the building response, the analysis is computationally expensive. The performance of ETABS 2016 integrated building design software shall be increased.

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APPENDIX

APPENDIX A-1 STRESS-STRAIN RELATION FOR NON-LINEAR STRUCTURAL ANALYSIS

e_c	f_{ck}	f_{cm}	e_{c1}	e_{cu1}	h	E_{cm}	k	σ_c
0	20	28	1.97	3.5	0.00	30	2.21	0.00
0.1	20	28	1.97	3.5	0.05	30	2.21	3.04
0.2	20	28	1.97	3.5	0.10	30	2.21	5.88
0.3	20	28	1.97	3.5	0.15	30	2.21	8.52
0.4	20	28	1.97	3.5	0.20	30	2.21	10.97
0.5	20	28	1.97	3.5	0.25	30	2.21	13.23
0.6	20	28	1.97	3.5	0.31	30	2.21	15.30
0.7	20	28	1.97	3.5	0.36	30	2.21	17.20
0.8	20	28	1.97	3.5	0.41	30	2.21	18.93
0.9	20	28	1.97	3.5	0.46	30	2.21	20.49
1	20	28	1.97	3.5	0.51	30	2.21	21.90
1.1	20	28	1.97	3.5	0.56	30	2.21	23.14
1.2	20	28	1.97	3.5	0.61	30	2.21	24.23
1.3	20	28	1.97	3.5	0.66	30	2.21	25.18
1.4	20	28	1.97	3.5	0.71	30	2.21	25.98
1.5	20	28	1.97	3.5	0.76	30	2.21	26.64
1.6	20	28	1.97	3.5	0.81	30	2.21	27.17
1.7	20	28	1.97	3.5	0.86	30	2.21	27.57
1.8	20	28	1.97	3.5	0.92	30	2.21	27.83
1.9	20	28	1.97	3.5	0.97	30	2.21	27.97
2	20	28	1.97	3.5	1.02	30	2.21	27.99
2.1	20	28	1.97	3.5	1.07	30	2.21	27.89
2.2	20	28	1.97	3.5	1.12	30	2.21	27.68
2.3	20	28	1.97	3.5	1.17	30	2.21	27.36
2.4	20	28	1.97	3.5	1.22	30	2.21	26.92
2.5	20	28	1.97	3.5	1.27	30	2.21	26.38
2.6	20	28	1.97	3.5	1.32	30	2.21	25.73
2.7	20	28	1.97	3.5	1.37	30	2.21	24.99
2.8	20	28	1.97	3.5	1.42	30	2.21	24.14
2.9	20	28	1.97	3.5	1.47	30	2.21	23.20
3	20	28	1.97	3.5	1.53	30	2.21	22.16
3.1	20	28	1.97	3.5	1.58	30	2.21	21.03
3.2	20	28	1.97	3.5	1.63	30	2.21	19.82
3.3	20	28	1.97	3.5	1.68	30	2.21	18.51
3.4	20	28	1.97	3.5	1.73	30	2.21	17.12
3.5	20	28	1.97	3.5	1.78	30	2.21	15.65

APPENDIX A-2 ANKOBER EARTHQUAKE GROUND ACCELERATION DATA

Ankober earthquake is recorded at furi- mountain station on December 4 2016 is shown in A-2 below. Numerical values for the ground acceleration in units of g, this include 551 data points at equal time spacing's of 0.01sec to be read column by column.; the first value is at t=0. These data are collected from Addis Ababa university Science faculty USGS department.

-0.00010	0.00002	0.00056	-0.00010	0.00053	-0.00009	-0.00017	0.00020
0.00004	0.00016	-0.00080	-0.00050	0.00126	-0.00003	0.00003	0.00010
0.00009	-0.00005	-0.00130	-0.00001	0.00037	0.00038	-0.00002	-0.00024
-0.00010	0.00014	0.00010	0.00036	-0.00080	0.00064	-0.00010	-0.00013
-0.00003	0.00005	0.00056	0.00003	-0.00030	0.00014	-0.00006	-0.00006
-0.00001	-0.00030	-0.00020	-0.00002	-0.00020	-0.00007	0.00011	-0.00008
0.00015	-0.00000	-0.00060	0.00047	0.00009	-0.00060	0.00034	-0.00010
0.00013	-0.00010	-0.00004	-0.00040	0.00033	-0.00040	0.00016	0.00014
-0.00001	0.00012	-0.00001	-0.00050	0.00051	-0.00003	-0.00009	-0.00001
-0.00006	0.00002	-0.00004	0.00031	-0.00060	0.00048	-0.00018	-0.00002
-0.00010	0.00003	-0.00050	0.00100	-0.00080	0.00008	-0.00024	0.00021
0.00002	-0.00001	-0.00030	-0.00080	-0.00030	-0.00050	0.00011	0.00009
-0.00008	-0.00010	0.00000	-0.00170	-0.00010	-0.00080	0.00003	0.00004
0.00003	-0.00030	0.00079	-0.00130	-0.00008	-0.00070	0.00026	0.00012
0.00004	-0.00040	0.00157	-0.00030	-0.00010	0.00017	0.00014	-0.00009
0.00005	-0.00070	0.00156	0.00157	0.00063	0.00086	-0.00022	-0.00058
-0.00020	-0.00090	-0.00020	0.00182	0.00067	0.00105	-0.00033	-0.00044
0.00085	-0.00020	-0.00160	0.00062	-0.00004	0.00047	-0.00027	0.00003
-0.00002	0.00119	-0.00230	-0.00020	0.00034	0.00005	0.00007	0.00037
-0.00003	0.00318	-0.00110	-0.00007	0.00035	-0.00033	-0.00006	0.00035
0.00007	0.00287	0.00053	-0.00030	-0.00030	-0.00050	0.00005	0.00000
-0.00001	0.00004	0.00160	-0.00050	-0.00030	0.00003	0.00041	-0.00015
0.00006	-0.00320	0.00153	-0.00040	0.00002	0.00038	0.00006	-0.00008
-0.00001	-0.00300	-0.00007	-0.00060	0.00011	0.00006	0.00007	0.00018
-0.00005	-0.00080	-0.00060	-0.00010	-0.00020	-0.00047	0.00001	0.00023
-0.00005	0.00042	-0.00050	0.00144	-0.00050	-0.00031	-0.00019	-0.00005
0.00006	0.00098	-0.00070	0.00103	-0.00030	-0.00012	-0.00038	-0.00014
-0.00010	0.00149	0.00111	-0.00010	-0.00008	-0.00025	0.00016	0.00003
-0.00002	0.00108	0.00141	-0.00100	-0.00010	0.00026	0.00008	0.00007
0.00013	0.00051	-0.00020	-0.00160	0.00052	-0.00001	-0.00014	-0.00003
0.00000	-0.00160	-0.00170	-0.00130	0.00062	0.00002	-0.00009	0.00008
-0.00004	-0.00190	-0.00110	0.00047	0.00052	0.00020	0.00037	0.00018
0.00002	-0.00090	5.7E-05	0.00152	0.00015	0.00041	0.00017	-0.00016
-0.00006	0.00000	0.00059	0.00059	-0.00020	0.00064	0.00009	-0.00010
0.00002	0.00129	0.00001	-0.00050	-0.00004	0.00018	-0.00002	-0.00009
-0.00005	0.00132	0.00038	-0.00080	-0.00040	-0.00061	-0.00020	0.00003
-0.00005	0.00092	0.00088	0.00005	-0.00030	-0.00052	-0.00003	-0.00018

Appendix A-2 Ankober Earthquake ground acceleration data(Cont'd)

0.00002	-0.00020	-0.00020	-0.00020	0.00008	-0.00020	0.00011
0.00029	0.00003	-0.00009	-0.00030	0.00001	0.00023	0.00000
0.00004	-0.00001	-0.00020	-0.00020	0.00014	0.00012	0.00003
-0.00020	0.00009	-0.00020	0.00012	0.00009	0.00003	-0.00004
-0.00002	-0.00003	-0.00001	0.00024	0.00015	0.00016	0.00015
0.00011	-0.00009	-0.00007	0.00036	-0.00006	0.00004	0.00011
0.00014	0.00003	0.00011	0.00023	0.00004	-0.00001	-0.00003
0.00014	0.00004	0.00025	-0.00002	0.00009	-0.00010	-0.00009
-0.00030	0.00008	0.00024	-0.00001	0.00008	-0.00008	-0.00030
-0.00050	0.00005	0.00026	-0.00010	0.00006	0.00000	-0.00010
-0.00020	-0.00020	0.00005	-0.00020	0.00000	0.00005	0.00005
0.00003	-0.00030	-0.00020	-0.00009	-0.00020	0.00000	0.00004
0.00044	-0.00010	-0.00010	-0.00003	-0.00010	-0.00004	-0.00009
0.00011	-0.00000	0.00001	0.00011	0.00009	0.00009	-0.00002
-0.00020	0.00012	-0.00003	-0.00003	0.00002	0.00007	0.00010
-0.00010	0.00011	-0.00003	-0.00008	0.00000	-0.00009	0.00006
0.00003	0.00004	0.00006	0.00007	0.00005	-0.00007	0.00004
0.00009	0.00002	0.00015	0.00012	-0.00004	0.00000	0.00007
0.00034	-0.00003	0.00008	0.00000	-0.00020	0.00002	0.00000
0.00023	0.00009	0.00004	-0.00010	-0.00010	0.00002	0.00009
0.00014	0.00011	0.00011	0.00015	0.00016	0.00010	0.00009
-0.00020	0.00004	0.00010	-0.00002	0.00012	-0.00002	-0.00004
-0.00020	-0.00006	0.00009	-0.00030	-0.00000	-0.00003	-0.00008
-0.00003	0.00000	-0.00005	-0.00010	-0.00020	0.00000	-0.00004
0.00009	0.00002	-0.00020	0.00017	0.00003	0.00002	0.00000
0.00007	0.00007	-0.00005	0.00020	-0.00010	0.00003	0.00005
0.00018	0.00007	-0.00007	0.00004	-0.00020	0.00001	-0.00002
0.00000	-0.00005	-0.00030	-0.00020	0.00014	-0.00008	-0.00010
-0.00030	0.00000	-0.00020	-0.00010	0.00030	-0.00005	-0.00009
-0.00020	0.00013	0.00000	0.00020	0.00013	-0.00008	-0.00010
-0.00030	0.00009	-0.00001	0.00002	0.00012	0.00007	-0.00040
-0.00020	-0.00002	0.00003	0.00000	-0.00001	0.00001	-0.00005
0.00001	-0.00009	0.00018	-0.00010	-0.00005	-0.00010	0.00001
0.00037	-0.00007	0.00027	-0.00020	-0.00009	-0.00006	
0.00031	0.00002	0.00009	0.00004	0.00001	-0.00005	
0.00017	0.00000	0.00000	-0.00010	-0.00004	0.00000	
-0.00004	-0.00020	-0.00006	0.00000	-0.00030	0.00011	

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