



Addis Ababa Institute of Technology
School of Civil and Environmental Engineering

Performance of reinforced concrete wall frame structures subjected to
high story eccentricity

A Project Submitted to School of Civil and Environmental Engineering in Partial Fulfillment of
the Requirements for the Degree of Master of Engineering in Structural Engineering

By

Biruk Kefelegn Tesfaye

Advisor: Dr.-Ing. Girma Zerayohannes

June, 2019



Addis Ababa Institute of Technology
School of Civil and Environmental Engineering

Performance of reinforced concrete wall frame structures subjected to
high story eccentricity

A Project Submitted to School of Civil and Environmental Engineering in Partial Fulfillment of
the Requirements for the Degree of Master of Engineering in Structural Engineering

By

Biruk Kefelegn Tesfaye
June, 2019

Approved by Board of Examiners

Dr.-Ing. Girma Zerayohannes
Advisor

Signature

Date

Dr. Shifferaw Taye
Examiner

Signature

Date

Chairman

Signature

Date

Declaration

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

Name: Biruk Kefelegn

Signature: _____

Place: Addis Ababa Institute of Technology

Date of Submission: June, 2019

Acknowledgement

Thanks to God for each and every success in my life and satisfactory accomplishment of this project.

My deepest gratitude goes to my project advisor Dr.-Ing. Girma Zerayohannes for his genuine guidance and valuable advice to accomplish the project on time.

At last, but not least, I would like to express my profound heartfelt thanks to my family and friends.

Table of Contents

Declaration	i
Acknowledgement	ii
List of Tables	v
List of Figures	vii
Notations	viii
Abstract	ix
Chapter One	1
Introduction.....	1
1.1 Background	1
1.2 Objective of Study	2
Chapter Two.....	3
Literature Review.....	3
2.1 Structural Systems	3
2.1.1 Moment Resisting Frames	4
2.1.2 Wall-Frame Structural Systems	5
2.1.3 Framed-Tubes	8
2.2 Center of mass and Center of Rigidity	8
2.3 Eccentricity.....	8
2.4 Earth Quake Analysis Method	9
2.4.1 Response Spectrum Method	11
Chapter Three.....	13
Analysis and Results	13
3.1 Material Properties	13
3.1.1 Concrete	13
3.1.2 Reinforcement Steel.....	14
3.2 Reinforced Concrete Section Properties	15
3.2.1 Section Properties	15
3.2.2 Effective Stiffness factors	16
3.3 Loading.....	17
3.3.1 Loads on Structure	17
3.3.2 Load Combinations	22

3.4	Building Models	22
3.5	Analysis Results and Discussion.....	25
3.5.1	Case 1: G+5 Building (Model 1)	25
3.5.2	Case 2: G+10 Building (Model 2)	30
3.5.3	Case 3: G+15 Building (Model 3)	35
3.5.4	Case 4: G+20 Building (Model 4)	41
3.5.5	Case 5: G+30 Building (Model 5)	49
3.5.6	Case 6: G+40 Building (Model 6)	57
	Conclusion and Recommendation	66
	References.....	68

List of Tables

Table 1: Structural Elements Cross-sectional Properties.....	15
Table 2: Property/Stiffness Modifiers.....	16
Table 3: Load case/ modal case	20
Table 4: Response spectrum function.....	21
Table 5: Load Combinations.....	22
Table 6: General model details for models without eccentricity.....	24
Table 7: General model details for models with eccentricity	24
Table 8: The member cross sections used in G+5 models	25
Table 9: Center of mass and Center of rigidity for G+5 model without eccentricity	26
Table 10: Center of mass and Center of rigidity for G+5 model with eccentricity	26
Table 11: Eccentricity at each floor level for G+5 model.....	27
Table 12: Period and Frequencies at each floor level for G+5 model.....	27
Table 13: Story Diaphragm Displacements at each floor level for G+5 model ...	28
Table 14: Story Drift at each floor level for G+5 model	28
Table 15: The member cross sections used in G+10 models	30
Table 16: Center of mass and Center of rigidity for G+10 model without eccentricity	31
Table 17: Center of mass and Center of rigidity for G+10 model with eccentricity	31
Table 18: Eccentricity at each floor level for G+10 model	32
Table 19: Period and Frequencies at each floor level for G+10 model	32
Table 20: Story Diaphragm Displacements at each floor level for G+10 model.	33
Table 21: Story Drift at each floor level for G+10 model.....	34
Table 22: The member cross sections used in G+15 models	36
Table 23: Center of mass and Center of rigidity for G+15 model without eccentricity	37
Table 24: Center of mass and Center of rigidity for G+15 model with eccentricity	37
Table 25: Eccentricity at each floor level for G+15 model	38
Table 26: Period and Frequencies at each floor level for G+15 model	39
Table 27: Story Diaphragm Displacements at each floor level for G+15 model.	40
Table 28: Story Drift at each floor level for G+15 model.....	40
Table 29: The member cross sections used in G+20 models	42
Table 30: Center of mass and Center of rigidity for G+20 model without eccentricity	43
Table 31: Center of mass and Center of rigidity for G+20 model with eccentricity	44
Table 32: Eccentricity at each floor level for G+20 model	45
Table 33: Period and Frequencies at each floor level for G+20 model	45
Table 34: Story Diaphragm Displacements at each floor level for G+20 model.	46
Table 35: Story Drift at each floor level for G+20 model.....	47

Table 36: The member cross sections used in G+30 models	49
Table 37: Center of mass and Center of rigidity for G+30 model without eccentricity	50
Table 38: Center of mass and Center of rigidity for G+30 model with eccentricity	51
Table 39: Eccentricity at each floor level for G+30 model	52
Table 40: Period and Frequencies at each floor level for G+30 model	53
Table 41: Story Diaphragm Displacements at each floor level for G+30 model.	54
Table 42: Story Drift at each floor level for G+30 model.....	55
Table 43: The member cross sections used in G+40 models	57
Table 44: Center of mass and Center of rigidity for G+40 model without eccentricity	58
Table 45: Center of mass and Center of rigidity for G+40 model with eccentricity	59
Table 46: Eccentricity at each floor level for G+40 model	60
Table 47: Period and Frequencies at each floor level for G+40 model	62
Table 48: Story Diaphragm Displacements at each floor level for G+40 model.	62
Table 49: Story Drift at each floor level for G+40 model.....	64

List of Figures

Figure 1: Number of story Vs. Structural System.....	4
Figure 2: Shear Wall Arrangement Examples	7
Figure 3: Response spectrum function graph	21
Figure 4: Typical Floor layout 1: Models without eccentricity	23
Figure 5: Typical Floor layout 2: Models with eccentricity.....	23

Notations

a_{gR} - reference peak ground acceleration on type A ground

a_g - design ground acceleration on type A ground

g - Acceleration due to gravity

q - Behaviour factor

q_0 - Basic value of behaviour factor

K_w - Factor reflecting the prevailing failure mode in structural systems with walls.

α_1 - multiplier of horizontal design seismic action at formation of first plastic hinge in the system

α_u - multiplier of horizontal seismic design action at formation of global plastic mechanism

T_1 - fundamental period of the building in the horizontal direction of interest

γ_I - importance factor

ξ - viscous damping ratio (in percent)

η - damping correction factor

H - Total height of the building

$S_d(T)$ - design spectrum (for elastic analysis)

Abstract

The main objective of this project is to study the performance of wall frame structures subjected to high eccentric loads. This project aims to investigate the effect of eccentricity on the lateral force resistance capacity of wall frame structural systems. It also identifies the members that are highly affected by this high story eccentricity and propose ways to improve their strength. This project aims to answer questions like; what is story eccentricity in wall frame structures? What is the response of wall frame structures with high eccentricity when they are subjected to lateral loads? what are the parameters that are mainly affected by story eccentricity? What are the effective measures that can be taken to avoid eccentricity on the structures?

The performance of twelve different buildings with different story levels and shear wall locations has been investigated using ETABS finite element analysis software. The building models has been selected to represent wall frame structures with and without story eccentricity. The analysis and design of each individual model has been conducted according to Ethiopian building code and standards. Then the analysis and design output results has been compared, briefly interpreted and discussed in order understand the effect of eccentricity on the performance of wall frame structures.

The project found that the lateral load resistance capacity of wall frame structures is highly dependent on story eccentricity and may cause failure if not considered during building design. The increase in eccentricity, increased both the period and inter story drift of the wall frame structure. The structures with high story eccentricity required bigger lateral load resisting structures than the structure with minimal or without eccentricity. According to its findings this project recommended to avoid story eccentricities in building design.

Chapter One

Introduction

1.1 Background

Reinforced concrete wall frame structures are the most widely used and preferable structural systems from medium rise and high-rise buildings. This structures in their life time may be exposed to different types of lateral loads including wind load and seismic loads. The performance of this structures to these applied lateral loads depends on many parameters. The distance between their center of mass and center of rigidity which is also referred to as eccentricity of the structure is one of them.

Eccentricity is the main source of torsional forces when lateral loads applied on the structure. They are two types of torsional loads according to many building design codes. This are accidental torsion and intrinsic torsion. Accidental torsion is mainly due to random variation of mass position and stiffness of lateral force resisting systems. Intrinsic torsion is caused by the difference between the natural load application and resistance points.

The interest to study the performance of reinforced concrete wall frame structures with high eccentricity subjected to lateral loads arise from the need to investigate the effect of this distance between the center of mass and center of rigidity on the performance of lateral force resisting structural elements and the building frame as a whole.

During the design process of a structure incorrect determination and application of eccentricity or building that has high story eccentricity may lead us to acquire unstable structure which may lead to catastrophic failure by causing some structural members to carry loads that are higher than their design load strength. Hence, the effect of high eccentricity in the performance of wall frame structural systems has to be investigated.

with safety requirement for the performance of structure with high eccentricity subjected to lateral loads like wind load and seismic loads and also correct way of accounting for these actions during the design of the structure.

Revision on different building design codes indicates that there is a variation on the specified accidental eccentricities. Ethiopian, Europe, USA, Romanian and Greece codes use 5%, Bulgarian code uses 10% accidental eccentricities and also

Mexican and Canadian codes use different approach to determine accidental eccentricity.

The major hypotheses drawn before conducting this research are eccentricity on a building structure greatly affect the overall response of a structure to the subjected lateral loads. This effect can reduce the strength of the structure when exposed to lateral loads like wind and seismic actions. The strength of the structure reduces with the increase in story eccentricities. The performance of building structures with high eccentricity at each floor level may be affected by the number of floors above the floor of interest.

1.2 Objective of Study

The General objective of this study is to investigate the performance of wall frame structures when they are subjected to high story eccentricities.

This project aims to investigate the effect of eccentricity on the lateral force resistance capacity of wall frame structural systems. It also identifies the members that are highly affected by this high story eccentricity and propose ways to improve their strength.

The specific objectives of this study are:

- Determine the main parameters that affect the resistance of wall frame structures when they are subjected to high eccentricity.
- Determine how these eccentric structures respond/perform when they subjected to lateral loads.
- Determine the main structural members that are critically affected by this eccentricity and how they respond during lateral loading.
- Determine how the number of floors above and below the story of interest affect its performance during lateral loading.
- Propose mitigation measures to reduce the effect of eccentricity and help the structure improve its performance during application of lateral loading.

Chapter Two

Literature Review

2.1 Structural Systems

The design of medium and high-rise buildings essentially involves a conceptual design (by architects) approximation analysis, primary design and optimization, to safely carry gravity and lateral loads. The design criteria are strength, serviceability, stability and human comfort. The strength can be satisfied by limit stresses, while the serviceability is satisfied by drift limit and stability is satisfied by sufficient factor of safety against buckling and $p-\Delta$ effect. [1]

The main aim of this project is to investigate the performance of wall frame structures subjected to high eccentric forces. However, first we have to have an understanding about the different forms of structural systems and what specifically made wall frame structures preferable form of structural systems for this project.

The most commonly used structural systems can be classified as follows; [1]

- (a) Moment resisting frames
- (b) Wall frame systems
- (c) Shear truss-outrigger braced systems
- (d) Framed-tubes
- (e) Tube-in-tube systems with interior columns
- (f) Bundled tubes
- (g) Truss tubes without interior columns
- (h) Modular tubes

The selected structural system should be able to carry different types of loads such as gravity, lateral, temperature, blast and impact loads. Optimizing the different type of structural systems for multi-story buildings, Khan, Lyengar and Colaco prepare the graph below which shows number of stories versus structural system.

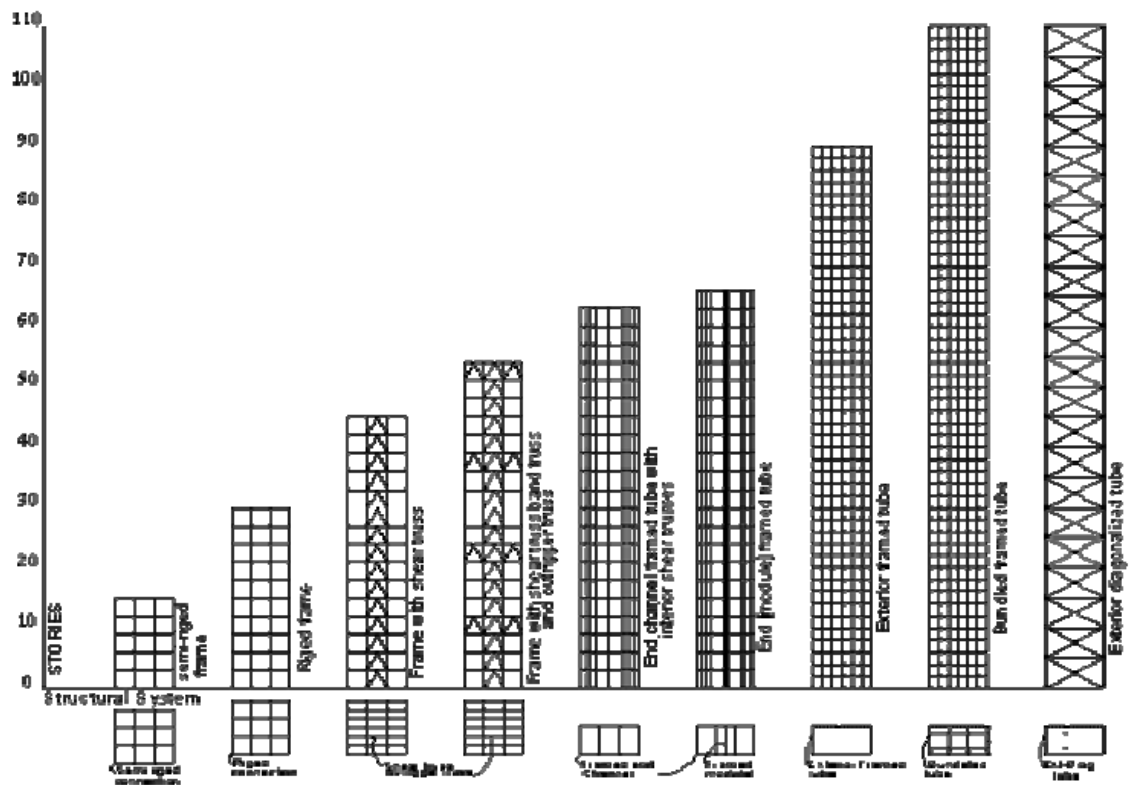


Figure 2.1 Number of story Vs Structural system [12]

Activ
Go to

Figure 1: Number of stories Vs. Structural System

From the graph we can see that the moment resisting frames (deep beam-column, rigid frame system) can be stable until 15 stories. When we are supposed to increase the story above it, we can use other structural systems. The use of rigid frame generally restricts columns' placements too regular, orthogonal layouts, often requires deeper beams and more closely spaced larger columns. These frame systems are normally not well suited for structures with usually long spans or tall columns. Rigid frames depend on extra stiff connections in the structural frame work to resist the effect of lateral forces. However, rigid frame is the most structurally inefficient of lateral load resisting systems. [1]

2.1.1 Moment Resisting Frames

The moment resisting frame structural systems are simply beam-column system in which the beams and columns are rigidly connected to provide moment resistance at joints and provide sufficient resistance against lateral loads. [1]

Individual frame systems are required to resist proportion of the lateral forces applied on the structure. The resistance to lateral loads of moment resisting frames are dependent on the bay span, number of bays in the frame, number of frames, individual stiffness of beam column elements and rigidity of beam column connection. The design of these frames is controlled by the bending stiffness of individual members. The deeper the member, the more efficiently its bending resistance can be. [1]

Buildings with moment resisting frame structural types are generally restricted to only a few stories in height in current practices, since other more efficient structural forms are available. [1]

2.1.2 Wall-Frame Structural Systems

Medium rise and high-rise building structure which are mainly affected by lateral forces due to wind or earthquake actions to an extent that they play an important role of their structural design. Over the last decade's various types of medium and high-rise structure systems have been developed to be efficient in resisting both gravity and lateral loading the structure encounter in its life time. one of the most common of these structural systems is the wall-frame structural system. This structural system comprises of a sets of shear walls and moment resisting frames, in which the walls and frames interact horizontally to share in carrying the lateral loading, where both carry their share of gravity loading. [2]

Walls and moment resisting frames will be arranged in parallel planes, so that they interact horizontally through the floor slab, or in the same plane and interact horizontally through the connecting beams. In both cases, the horizontal interaction results from the change in relative stiffness's between the walls and frames over the height of the structure, the walls restraining the frame from deflecting in the lower region and the frames restraining the walls in the upper region. Walls behave as flexural cantilever, while frames behave as shear cantilever.[3]

Difference in deflection forms of walls and frames cause them to interact horizontally (shear wall-frame interaction). Because the horizontal interaction between the walls and frames, the distributions of loading on the walls and the frames may be very different from that of the external loading, with the result that the lateral stiffness of the structure is increased, making wall-frame a moderately efficient type of medium and high-rise building structure. [2]

Wall-frame structures are suited for buildings of up to 50 stories or more. It is possible to design shear walls as cores that surround elevator shafts and

stairwells. Closed and partially closed cores can increase the stiffness of the building laterally and torsionally).

The minimum thickness for wall to be provided is 150 mm. Shear wall in its plane has tremendous load carrying capacity but for out of its plane loads its capacity decreases i.e. its moment resistance capacity decreases. Moments that are resisted by wall are ultimately transferred to foundation and as a large amount of moment has to be resisted by shear wall, foundation of shear wall becomes heavy.[3]

As all the load that comes on beam is transfer to wall through junction and that junction has a crucial point of this system is transfer of load. So, this connection needs special attention and technique.[4]

Mostly in walls openings are not provided but if it is needed to be provided than skill and technique are required. If openings are provided at critical point then it will lead to failure in transfer of load than ultimately leads to failure in structural system.

Advantages of Reinforced Frame braced with Shear wall

- The size of the column gets reduced considerable with the use of shear wall in frame.
- Size of column can be changed to a large extent at different floors with the use of shear wall in frame.
- More carpet area is available in building as size of column is reduced with the use of shear wall in frame.
- Cost of construction is less as compared to frame without shear wall.
- It is more resistant to Earthquake.
- Speed of construction is fast.

The most convenient place to locate shear wall is an external blank wall on edges or on two parallel edges so that stiffness of structure is maintained in best possible way. It should be spaced symmetrically so that center of gravity (of structure remains at center and there is not much eccentricity on application of lateral loads like Seismic, wind etc. So, its placement needs special skills and experience because if not placed at proper location it would lead to adverse behavior.

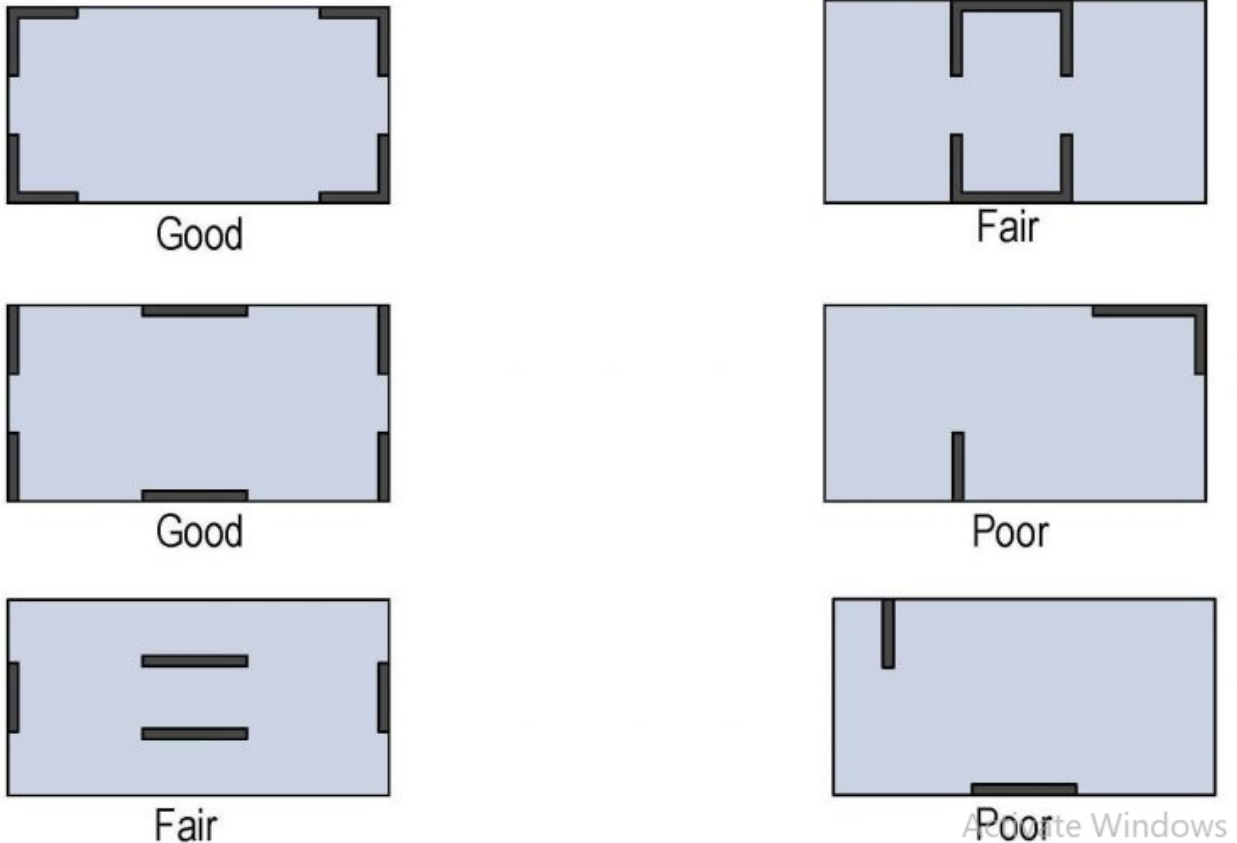


Figure 2: Shear Wall Arrangement Examples

An understanding of the interaction in a wall-frame structure is given by the deflected shapes of a shear wall and a moment resisting frame subjected independently to horizontal loading. The wall deflects in a flexural mode, with concavity downwind and a maximum slope at the top, while the frame deflects in a shear mode, with concavity upwind and a maximum slope at the base. The walls and frames of a structure that is symmetrical about the axis of loading are constrained to deflect identically by the in-plane rigidity of the floor slabs. Consequently, the deflected shape of the dual system has a flexural profile in the lower part and a shear profile in the upper. [2]

The wall and the frame constrain each other from adopting their free deflected shapes, and cause a redistribution of forces between the two. The redistribution of forces is achieved through the presence of horizontal interaction forces. The wall restrains or holds back the frame in the lower story's, while the frame restrains the wall in the upper stories. The significantly different top slopes of the freely deflecting separate wall and frame cause them, when combined in a wall-frame structure, to push on each other with a concentrated interaction force at the top, so that they adopt the same slope. The resultant effect of the

horizontal interaction contributes to the wall-frame structure's lateral stiffness, making it greater than the summation of the individual wall's and frame's lateral stiffness's. The degree of horizontal interaction is governed by the relative stiffness's of the walls and frames, and the height of the structure. [2]

Height is a major factor in determining the influence of the frame on the lateral stiffness of the wall-frame. In low-rise buildings the contribution of the frame to the lateral resistance is often negligible, but in medium to high-rise buildings its contribution can be significant and should be considered in the design for lateral loading. [4]

2.1.3 Framed-Tubes

The framed tube system is a special case of the moment resisting frame, which usually consists of closely spaced wide steel columns combined with relatively deep beams. These frames are usually, but not only, located on the perimeter of the structure, to introduce more stiffness to overcome the problem of excessive horizontal deflection of orthodox moment resisting frames, as the expense of reduction in ductility. [1]

2.2 Center of mass and Center of Rigidity

Center of mass is a position defined relative to an object or system of objects. It refers to a unique point by which the weighted relative position of the distributed mass sums to zero, or a point at which if a force is applied on the structure at that point the structure moves in the direction of force without rotating. Center of mass is the average position of all parts of the system weighted according to their masses. For simple objects with uniform density their center of mass is located at their centroid. Therefore, we consider center of mass as a point in which mass of the object is concentrated for calculation reasons. The weighted relative position of the distributed mass sums to zero at the center of mass. [3]

Center of rigidity is the stiffness centroid within a floor-diaphragm plan. Center of rigidity is also a point at a particular story as the application of lateral load at that point will not produce rotation of that story if a slab modeled as rigid diaphragm. [4]

2.3 Eccentricity

Eccentricity is defined as the distance between the center of rigidity and center of mass of a respective floor of a multi-story building while accidental eccentricity generally accounts for factors such as the rotational component of the ground

motion about the vertical axis however most of the time buildings are too short to be affected by this phenomenon accidental eccentricities are due to random variations of mass position and stiffness of lateral resisting elements. [4]

Eccentricities are used in several design building codes in order to consider the torsional behavior of lateral loads such as earth quake. They are two basic types of design eccentricities; this are intrinsic and accidental eccentricities. Intrinsic eccentricities are caused by the unbalance of the theoretical positions of load and resistance centers; or in other words, as a consequence of non-coincident mass and rigidity centers which will give rise to intrinsic torsion. On the other hand, accidental eccentricities are due to the random variations of these centers relative to the theoretical positions and to possible ground motion rotational components (i.e., spatial variation of the ground record). In most cases, however, plan dimensions of buildings are small allowing ground rotation effects to be neglected and, therefore, accidental eccentricity is mainly due to random variations of mass position or stiffness of lateral resisting elements. These accidental eccentricities generate accidental torsional forces in the building. [3]

Seismic design building codes used worldwide considers the effect of accidental eccentricity which will give rise to accidental torsional moment in design of buildings with rigid diaphragms. This additional torsion moment shall be determined by simply assuming the seismic mass to be displaced from the calculated center of mass in each direction a distance equal to the accidental eccentricity value (i.e. eccentricity percentage of the building dimension at each story perpendicular to the direction of the seismic design force under consideration). Many building codes that has been used worldwide uses the value of 5% accidental eccentricity for multi-story buildings. For completeness, it is worth stating that some other few international design codes too conservatively assume the accidental eccentricity to be $\pm 10\%$ of the applicable building dimension. [5]

2.4 Earth Quake Analysis Method

The engineering community involved with the development of seismic design procedures, there is a general belief that the conventional elastic design and analysis methods cannot capture many important aspects that control the seismic performance of structures in severe earthquakes. Moreover, another powerful tool, inelastic time-history analysis, is computationally expensive and not feasible for many cases. [6]

Design has always been a compromise between simplicity and reality. The latter term, reality, seems to be very complex due to big uncertainties in imposed

demands and available capacities. The first term, simplicity, is a necessity driven by computational cost and at the same time the limited ability to implement complexity with available knowledge and tools. [6]

Performance based design and assessment in structural engineering is an important factor. The decision of the analysis method for performance-based assessment is being a new topic and linear elastic methods of analysis have been used for a long time. The force demand on each component of the structure is obtained and compared with available capacities by performing an elastic analysis. [6]

Elastic analysis methods include code static lateral force procedure, code dynamic procedure and elastic procedure using demand-capacity ratios. These methods are also known as force-based procedures which assume that structures respond elastically to earthquakes. [5]

In code static lateral force procedure, a static analysis is performed by subjecting the structure to lateral forces obtained by scaling down the smoothened soil dependent elastic response spectrum by a structural system dependent force reduction factor, "R". In this approach, it is assumed that the actual strength of structure is higher than the design strength and the structure is able to dissipate energy through yielding. [6]

In code linear dynamic procedure, force demands on various components are determined by an elastic dynamic analysis. Linear dynamic analysis procedures include response spectrum analysis and time history analysis. The former is a method for obtaining an approximate solution of the coupled, second-order, linear differential equations of motion under forced vibration. [7]

The response spectrum analysis begins with determining the natural frequencies and mode shapes via an eigenvalue analysis. The coupled equations of motion are then decoupled via a modal transformation wherein the principle of orthogonality of the mode shapes with respect to the mass, damping, and stiffness matrices is applied. Each decoupled equation corresponds to the equation of motion of a single-degree-of-freedom (SDOF) system associated with a mode of vibration. The peak response of each single degree-of-freedom system is obtained through the use of elastic-response spectra. Since the peak response in each mode does not occur at the same time, the peak response with contributions from all modes is estimated via the application of modal combination rules that are based on random vibration theory. [6]

Time history analysis provides a method for obtaining the “exact” response of a structure as a function of time, whereas response spectrum analysis results in estimates of peak response. [6]

The response-history is normally determined using step-by-step numerical integration of the equation of motion. Consequently, time history analysis is performed using computer software where the ground acceleration is divided into small time steps and the response is calculated at the end of each time step while satisfying dynamic equilibrium. In general, the ground acceleration is only available at discrete points in time separated by a fixed time step while the solution may be sought at points in time other than at integer multiples of the time step. Thus, the ground acceleration must be interpolated, with linear interpolation often being adequate. [6]

2.4.1 Response Spectrum Method

The seismic analysis and design of a structure to be built at a particular location to be performed, the actual time history record is required. However, it is not possible to have such records at each and every location. Further, the seismic analysis of structures cannot be carried out simply based on the peak value of the ground acceleration as the response of the structure depend upon the frequency content of ground motion and its own dynamic properties. To overcome the above difficulties, earthquake response spectrum is the most popular tool in the seismic analysis of structures. [7]

The response spectrum is simply a plot of the peak or steady-state response (displacement, velocity or acceleration) of a series of oscillators of varying natural frequency, which are forced into motion by the same base vibration or shock. The resulting plot can then be used to pick off the response of any linear system, given its natural frequency of oscillation. One such use is in assessing the peak response of buildings to earthquakes. Response spectra can also be used in assessing the response of linear systems with multiple modes of oscillation (multi-degree of freedom systems), although they are only accurate for low levels of damping. Model analysis is performed to identify the modes, and the response in that mode can be picked from the response spectrum. This peak response is then combined to estimate a total response. The earthquake spectrum, on the other hand, is an average of a number of earthquake records modified for site specific conditions and then smoothed out for design purposes which are specified by the appropriate building code. [7]

Response spectra are curves plotted between maximum response of SDOF system subjected to specified earthquake ground motion and its time period (or

frequency). Response spectrum can be interpreted as the locus of maximum response of a single degree of freedom system (SDOF) for given damping ratio. Response spectra thus helps in obtaining the peak structural responses under linear range, which can be used for obtaining lateral forces developed in structure due to earthquake thus facilitates in earthquake-resistant design of structures. [8]

Usually response of a SDOF system is determined by time domain or frequency domain analysis, and for a given time period of system, maximum response is picked. This process is continued for all range of possible time periods of SDOF system. Final plot with system time period on x-axis and response quantity on y-axis is the required response spectra pertaining to specified damping ratio and input ground motion. Same process is carried out with different damping ratios to obtain overall response spectra. [7]

The three spectra i.e. displacement, pseudo velocity and pseudo acceleration provide the same information on the structural response. However, each one of them provides a physically meaningful quantity and therefore, all three spectra are useful in understanding the nature of an earthquake and its influence on the design. [8]

According to Euro code 8, 2004 [56], this type of analysis shall be applied to buildings whose response is significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction. The response of all modes of vibration contributing significantly to the global response shall be taken into account. These requirements may be deemed to be satisfied if: [7]

- The sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure; [7]
- All modes with effective modal masses greater than 5% of the total mass are taken into account. [7]

Static method specified in building codes are based on single-mode response with simple corrections for including higher mode effects. While appropriate for simple regular structures, the simplified procedures do not take into account the full range of seismic behavior of complex structures. Therefore, dynamic analysis is the preferred method for the design of buildings with unusual or irregular geometry. [9]

Chapter Three

Analysis and Results

Finite element method is so far the well-developed numerical method to deal with the analysis and design of wall frame structures. It is very tedious, difficult and time consuming to deal with the analysis and design of multistory, multi-span wall-frame structures using finite element method without applying any kind of finite element software.

For the purpose of this project, it has been investigated the performance of wall-frame structures subjected to different eccentricities using a total no. of 12 wall-frame building structures with variable shear wall locations and number of stories using ETABS finite element software.

3.1 Material Properties

Reinforced concrete is a dominant structural material throughout the world because of the wide availability of constitutions of concrete and reinforcing steel bars, the relatively simple skills required for its construction and the economy of reinforced concrete compared to other forms of construction. Other advantages include mouldability to any desired shape in forms while plastic, fire resistance, suitability of material for architectural shapes, rigidity and low maintenance.

3.1.1 Concrete

Concrete is a conglomerate, artificial stone like material obtained by hardening and curing a mixture of mainly cement, water and aggregates and sometimes admixtures.

The main measure of the structural quality of concrete is its compression strength. Tests for this property are made on cylindrical specimen of height equal to twice the diameter (usually 6x12 inches, i.e. 150x300mm) originally as specified by American society for Testing and materials (ASTM). According, the cylinder specimens are moist cured at about 70±50F, generally for 28 days and then tested in the laboratory at a specified rate of loading usually to reach the maximum stress in 2 to 3 minutes. The compression strength obtained from such test is known as the cylinder strength f_c or f_{ck} and this is the main property specified for design purpose.

Depending up on the mix (especially the water cement ratio) and the time and quality of curing, compressive strength of concrete can be obtained up to

100MPa. For most practical and ordinary use cylindrical strength (f_{ck}) available ranges between 20 to 50 MPa.

The concrete strength class used for the purpose of this project are C25 concrete and C30 concrete with cylindrical strength (f_{ck}) equivalent to 20MPa and 25MPa respectively

According to EBCS EN 1991-1-1:2014, Annex A: Table A1 the density (mass/volume), of normal weight of concrete is stipulated as 24 kN/m³.

The modulus of elasticity (E_{cm}) of normal weight concrete is given as 30GPa for cylindrical strength (f_{ck}) of 20MPa and 31GPa for cylindrical strength (f_{ck}) of 25MPa according to EBCS EN 1992-1-1:2014, Table 3.1. Damping coefficient of 0.05 and poissonous ratio of 0.2 are used for concrete in this project.

Concrete has also disadvantages; an important one is that concrete is a relatively brittle material i.e. it easily breaks in tension while it is very strong in compression. This disadvantage can however be offset by reinforcing concrete with steel in the tension zone.

3.1.2 Reinforcement Steel

It is a high-strength and high cost steel bar used in the construction of reinforced concrete structures to provide additional strength. When reinforcing steel is used with concrete, the concrete is made to resist compression stress and the steel is made to resist tensile stress with or without additional compressive stress.

Characteristic properties of reinforcing bars are expressed using its yield strength f_y (f_{yk}) and modulus of elasticity E_s . Ethiopian building code and standards EBCS EN 1992-1-1:2014, section 3.2.2. (3) recommends the lowest value of yield strength to be 400MPa. The reinforcement steel yield strength of f_{yk} 400MPa and modulus of elasticity (E_s) 200GPa is used for the purpose of this project.

According to EBCS EN 1991-1-1:2014, Annex A: Table A4 the density (mass/volume), of reinforcement steel bar is stipulated as 77 kN/m³.

The yield strength (f_y), tensile strength (f_u), expected yield strength (f_{ye}) and expected tensile strength (f_{ue}) of the reinforcement bars are taken equivalent with the yield strength of steel 400MPa for the purpose of this project.

When reinforced concrete structural elements are used, sufficient bond between the two materials must be developed to ensure that there is no relative movement between the steel bars and the surrounding concrete. This bond may be

developed by, chemical adhesion natural roughness closely spaced rib-shaped surface deformation of reinforcement bars.

3.2 Reinforced Concrete Section Properties

3.2.1 Section Properties

Design of concrete sections involves determining the cross-sectional dimensions of concrete structural members and the required quantity of reinforcement. A large number of parameters have to be dealt with in design of concrete sections such as geometrical width, depth, area of reinforcement, steel strain, concrete strain and steel stress. Consequently, trial and adjustment are necessary in the choice of concrete sections, with assumptions based on conditions at site, availability of the constituent materials, particular demands of the owners, architectural and headroom requirements, applicable codes and environmental conditions.

Table 1: Structural Elements Cross-sectional Properties

Item No.	Structural Element	Section Name	Width (mm)	Height (mm)	Material	Reinforcement Cover (mm)
1	Column	CL 40X40	400	400	C30 Concrete	50
		CL 50X50	500	500	C30 Concrete	50
		CL 60X60	600	600	C30 Concrete	50
		CL 70X70	700	700	C30 Concrete	50
		CL 80X80	800	800	C30 Concrete	50
		CL 90X90	900	900	C30 Concrete	50
		CL 100X100	1000	1000	C30 Concrete	50
		CL 110X110	1100	1100	C30 Concrete	50
		CL 120X120	1200	1200	C30 Concrete	50
		CL 130X130	1300	1300	C30 Concrete	50
		CL 140X140	1400	1400	C30 Concrete	50
		CL 150X150	1500	1500	C30 Concrete	50
2	Beam	BM 30X50	300	500	C25 Concrete	50
		BM 50X60	500	600	C25 Concrete	50
		BM 70X40	400	700	C25 Concrete	50
		BM 60X50	500	600	C25 Concrete	50
		BM 40x60	400	600	C25 Concrete	50
Item No.	Structural Element	Section Name	Thickness (mm)		Material	Reinforcement Cover (mm)
3	Wall	Wall 300mm	300		C30 Concrete	20

		Wall 400mm	400	C30 Concrete	20
		Wall 500mm	500	C30 Concrete	20
4	Slab	Slab 200mm	200	C25 Concrete	20

The structural models used to investigate the performance of buildings subjected to variable eccentricity have used different sizes of walls, columns and beams that vary realistically with the location of concrete shear wall and the number of stories at the given mode.

3.2.2 Effective Stiffness factors

It is necessary to define effective elastic properties to define the elastic part of the lumped plasticity model since the elastic part of the member cracks when the member enters in to inelastic range. Hence, the stiffness reduction factors were used to reduce the gross stiffness properties of the beams and the columns and the reduced stiffness were used to define the effective stiffness model and the lumped-plasticity model. The effective rigidity assumed for each member of the structure should be consistent with its intended behavior and the models used. The approximate component initial effective stiffness values according to EBCS EN is used for beams, columns, shear walls and slabs.

EBCS EN 1998:2014 stipulates that unless a more accurate analysis of cracked elements is formed the elastic flexural and shear stiffness properties of concrete and masonry elements shall be taken one half of the stiffness of the corresponding uncracked element.

Table 2: Property/ Stiffness Modifiers

Item	Property/ Stiffness	Property/ Stiffness modifiers for analysis
1	Cross-section (axial) area	1
2	Shear area in 2 direction	0.5
3	Shear area in 3 direction	0.5
4	Torsional constant	0.1
5	Moment of inertia about 2 axis	0.5
6	Moment of inertia about 3 axis	0.5
7	Mass	1
8	Weight	1

3.3 Loading

3.3.1 Loads on Structure

3.3.1.1 Dead Load

The dead load on wall frame structure is the aggregate weight of all structural elements and also finishing and partition loads.

$$\text{Cement Screed} = 0.03 \times 23 = 0.69 \text{ kN/m}^2$$

$$\text{Plastering} = 0.03 \times 23 = 0.69 \text{ kN/m}^2$$

$$\text{Floor Finish} = 0.01 \times 27 = 0.27 \text{ kN/m}^2$$

$$\text{Partition Load} = 2 \text{ kN/m}^2$$

Therefore, the total unfactored dead load is summation of the above and additional self-weight of the structure is included using the software:

$$\text{Unfactored Dead Load} = 3.65 \text{ kN/m}^2 + \text{Self Weight of Structure}$$

Unit weight of cement screed and plastering is 23 kN/m^3 according to EBCS EN 1991-1-1:2014, Annex A: Table A1 and Granite (Marble) floor finishing is assumed to be used in the entire floor area of the building and its Unit weight is 27 kN/m^3 according to EBCS EN 1991-1-1:2014, Annex A: Table A2.

3.3.1.2 Live Load

Assuming the purpose of the building: Office building.

Category of use: Category B According to EBCS EN 1991-1-1: 2014, Table 6.1

The induced unfactored live loads on the structure for category of use B according to EBCS EN 1991-1-1: 2014, Table 6.2 is;

$$Q_k = 4.5 \text{ kN}$$

$$q_k = 3.0 \text{ kN/m}^2$$

3.3.1.3 Earth Quake Load

Earth quake loading is a product of natural loading which depend on geometric location of the structure. The earth quake forces can be described as a function of the acceleration coefficient, soil type, behavior factor and the fundamental period.

Method of calculating lateral (seismic) loads and combinations according to EBCS EN 1990:2014. The earth quake effect of a structure can be analyzed either, using response spectrum analysis or equivalent static analysis procedure.

The response spectrum method or the mode superposition method is generally applicable to analysis of dynamic response of complex structures in their linear range of behavior. This particular applies to analysis of forces and deformation in multistory buildings due to medium intensity ground shaking causing moderately large but essentially linear response of the structure. The method is based on the fact that for certain forms of damping which are reasonable model for many buildings the response in each natural mode of vibration can be computed independently of the others and the modal responses can be combined to determine the total response.

The response spectrum is simply a plot of the peak or steady-state response (displacement, velocity or acceleration) of a series of oscillators of varying natural frequency, which are forced into motion by the same base vibration or shock. The resulting plot can then be used to pick off the response of any linear system, given its natural frequency of oscillation.

The equivalent lateral load analysis procedure requires less effort because, except for the fundamental period, the periods and shapes of the higher natural modes are not needed. The magnitude of lateral forces is based on an estimate of the fundamental period of vibration and their distribution on simple formula appropriate for buildings with regular distribution of mass and stiffness over height.

The main difference between the two procedures lies in the magnitude of the base shear and distribution of the lateral forces. Where as in the response spectrum method the force calculations are based on computed periods and mode shapes of several modes of vibration. In the equivalent lateral load, they are based on an estimate of the fundamental period and simple formula for distribution of forces which are appropriate for buildings with regular distribution of mass and stiffness over height.

According to EBCS EN 1998:2014, we can decide which method of calculating lateral (seismic) load effects is appropriate base on structural model, structural analysis, the value of behavior factor γ , geometric non-regularity limit, non-regularity distribution of over strength in elevation exceeding the limits can be most appropriate for our structure system.

Response spectrum method of lateral (seismic) load calculation according to EBCS EN 1998-1: 2014 is used for earth quake loading determination in this project.

The soil formation in Addis Ababa varies from site to site and sometimes it even shows great variation in the same site. Sites where deep deposits of dense or medium dense sand, gravel, stiff clay with thickness from several tens to many hundreds of meters are assumed for this study. According to EBCS EN 1998-1: 2014 Table 3 this ground condition is of Ground Type C with a bearing capacity (C_u) ranging from 70kPa to 250kPa.

Ground acceleration a_g/g ;

Addis Ababa: is Zone 3 according to EBCS EN 1998-1:2014, Annex D: Table D2
 $a_g/g = 0.10$ for zone 3 according to EBCS EN 1998-1:2014, Annex D: Table D1

Behavior factor q ;

$q = q_0 K \geq 1.50$ According to EBCS EN 1998-1:2014, equation 5.1

$q_0 = 3.0 \alpha_u/\alpha_1$ According to EBCS EN 1998-1:2014, Table 5.1 for dual systems with medium ductility class.

For frame or frame equivalent dual systems with multistory, multibay frames or equivalent dual structures with plan irregularity

$\alpha_u/\alpha_1 = (1.3 + 1.0)/2 = 1.15$... According to EBCS EN 1998-1: 2014 section 5.2.2.2 (5) (a) and section 5.2.2.2 (6)

Hence, $q_0 = 3.0 \alpha_u/\alpha_1$

$$q_0 = 3.0 \times 1.15 = 3.45$$

$K_w = 1.0$ for frame or frame equivalent dual systems according to EBCS EN 1998-1: 2014 section 5.2.2.2. (11)P

Hence, $q = q_0 * K_w \geq 1.50$

$$q = 3.45$$

Correction factor is 1.0 for 5% viscous damping.

Summary on properties used in response spectrum analysis

H	74.3 m	Framing Type	moment resistant space concrete frame				
C _t	0.075	Ground Type	C				
T ₁	1.898	Ductility Class	DCM				
μ _ø	5.9	Structure Type	Frame System (Dual System or Coupled wall)				
ZONE	3	building type	multi-bay and multistory				
a _g	0.1	a _u /a ₁	1.3				
IC	II	(a _u /a ₁) for plan irregularity	1.15				
γ _I	1	q _o	3.45				
a _{gr}	0.1	Regular in Plan	NO				
ξ	0.05	Regular in Elevation	YES				
η	1.407	q	3.45				
Sd(T)	0.026	Type 1 elastic response parameters	S	T _B	T _c	T _D	
			1.15	0.2	0.6	2	

Load Case/Modal Cases

Table 3: Load case/ modal case

Load Case Name	Load Case Type	Function	Load Pattern Type	Scale Factor
Dead	Dead		Dead	1
Live	Live		Live	1
RSU1		Response Spectrum	Acceleration U1	9806.65
RSU2		Response Spectrum	Acceleration U2	9806.65

Earth quake loads are considered using response spectrum functions that consists of two load cases that are defined as accelerations in U1 and U2 directions of the corresponding response spectrum function. CQC method of modal combination is used to combine modal responses for this project. The maximum number of modal cases to run is limited to 12 modes for this project.

Response Spectrum

Table 4: Response spectrum function

Name	Period (Sec)	Acceleration
Response Spectrum 1	0	0.076667
Response Spectrum 1	0.067	0.078889
Response Spectrum 1	0.133	0.081111
Response Spectrum 1	0.2	0.083333
Response Spectrum 1	0.6	0.083333
Response Spectrum 1	0.833	0.06
Response Spectrum 1	1.067	0.046875
Response Spectrum 1	1.3	0.038462
Response Spectrum 1	1.533	0.032609
Response Spectrum 1	1.767	0.028302
Response Spectrum 1	2	0.025
Response Spectrum 1	3.333	0.02
Response Spectrum 1	4.667	0.02
Response Spectrum 1	6	0.02
Response Spectrum 1	7.333	0.02
Response Spectrum 1	8.667	0.02
Response Spectrum 1	10	0.02



Figure 3: Response spectrum function graph

3.3.2 Load Combinations

Table 5: Load Combinations

Combo No.	Combination Type	Factors			
		Gk	Qk	RSu1	RSu2
1	ULS combo	1.35	1.5		
2	SLS combo	1	1		
3	Comb EQ in +ve X dxn accompanied by -ve Y dxn	1	0.3	1	0.3
4	Comb EQ in +ve X dxn accompanied by +ve Y dxn	1	0.3	1	-0.3
5	Comb EQ in -ve X dxn accompanied by +ve Y dxn	1	0.3	-1	0.3
6	Comb EQ in -ve X dxn accompanied by -ve Y dxn	1	0.3	-1	-0.3
7	Comb EQ in +ve Y dxn accompanied by +ve X dxn	1	0.3	0.3	1
8	Comb EQ in +ve Y dxn accompanied by -ve X dxn	1	0.3	0.3	-1
9	Comb EQ in -ve Y dxn accompanied by +ve X dxn	1	0.3	-0.3	1
10	Comb EQ in -ve Y dxn accompanied by -ve X dxn	1	0.3	-0.3	-1

After completion of the assignment of wall frame sections and corresponding load assignments, automatic wall and slab meshing is conducted to discretize this thine shell elements. The rigid floor diaphragm is assigned at each floor level.

3.4 Building Models

Performance of buildings subjected to high eccentricity is investigated by considering two floor layouts. The first-floor layout having zero eccentricity and the second one with maximum eccentricity.

The investigation is future continued by keeping these two typical floor levels and varying the number of stories incrementally. The parametric study is conducted on twelve types of buildings which have two different types of floor arrangements arrangement but variable stories.

The buildings are assumed to cover the same plan area, with similar columns and concrete wall arrangements. As shown in the figure below;

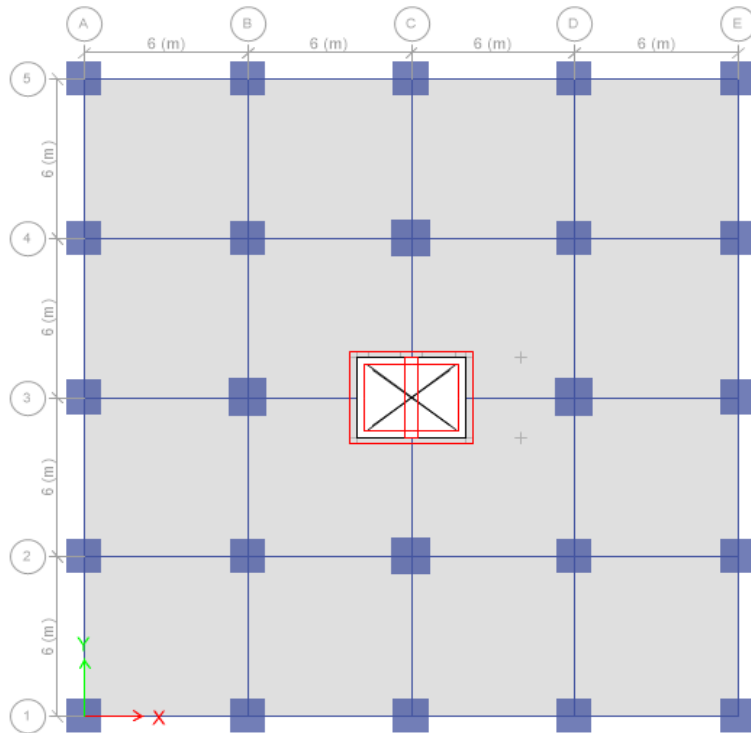


Figure 4: Typical Floor layout 1: Models without eccentricity

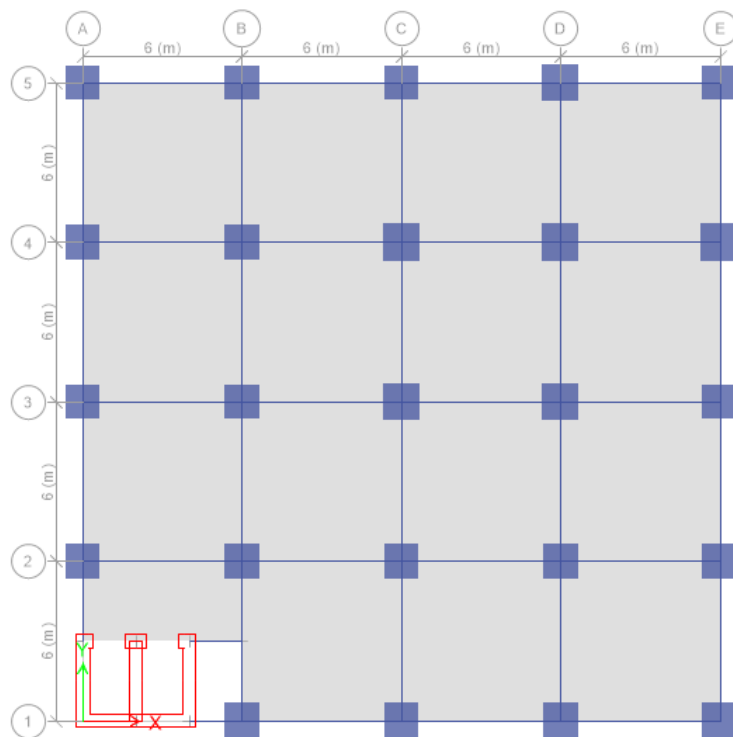


Figure 5: Typical Floor layout 2: Models with eccentricity

The size of columns, beams and shear walls for individual models increased realistically as the number of stories is increased and arrangement and location of shear wall varied. The building models have constant story height of 3.5m throughout, the beam column connections are rigid, and rigid diaphragms are applied for individual floors. The general details of each building model used in this project are shown in the table below.

Table 6: General model details for models without eccentricity

Model	Panel Dimension Lx(m)	Panel Dimension Ly (m)	No of Panels in X Direction	Number of Panels in Y Direction	No. Of Stories	Shear wall Location	
						X-coordinate	Y-Coordinate
Model 1-0	6	6	4	4	5	10	10.5
Model 2-0	6	6	4	4	10	10	10.5
Model 3-0	6	6	4	4	15	10	10.5
Model 4-0	6	6	4	4	20	10	10.5
Model 5-0	6	6	4	4	30	10	10.5
Model 6-0	6	6	4	4	40	10	10.5

Table 7: General model details for models with eccentricity

Model	Panel Dimension Lx(m)	Panel Dimension Ly (m)	No of Panels in X Direction	Number of Panels in Y Direction	No. Of Stories	Shear wall Location	
						X-coordinate	Y-Coordinate
Model 1-1	6	6	4	4	5	0	0
Model 2-1	6	6	4	4	10	0	0
Model 3-1	6	6	4	4	15	0	0
Model 4-1	6	6	4	4	20	0	0
Model 5-1	6	6	4	4	30	0	0
Model 6-1	6	6	4	4	40	0	0

3.5 Analysis Results and Discussion

3.5.1 Case 1: G+5 Building (Model 1)

The effect of eccentricity on the performance of wall frame structures has been investigated in this section using a six story G+5 building model. The effect of eccentricity on the performance of this short story structures is clearly shown using two six story building models with different wall frame structural layout. The first building model has a structural wall frame arrangement that will produce zero eccentricity and the second building the wall and frames are arranged in such a way to produce high story eccentricity. The loading and material properties of all structural elements used is briefly shown in the previous section.

The buildings laid out on a rectangular grid with a typical 6m width bays in the X and Y directions. For purpose of simplicity the beams, slab and shear walls have constant cross section throughout the building and the columns have been designed and the optimum section is used. The structures are hypothetical and have been chosen for the illustrative purpose. The member cross sections selected for this analysis were;

Table 8: The member cross sections used in G+5 models

Item No.	Structural Element	Section Name		Width (mm)	Height (mm)
		Model 1-0 Without Eccentricity	Model 1-1 With Eccentricity		
1	Column	CL 40X40	CL 40X40	400	400
2	Beam	BM 30X50		300	500
				Thickness (mm)	
3	Wall	Wall 300mm		300	
4	Slab	Slab 200mm		200	

The structural models have then been modeled, analyzed and designed according to Ethiopian building code and standards EBCS EN:2014. The analyses and design results are compared with each other and briefly discussed to investigate the effect of eccentricity on the performance of building structure.

Table 9: Center of mass and Center of rigidity for G+5 model without eccentricity

Model	Story	Mass (kg)	Center of Mass (m)		Center of Rigidity (m)		Eccentricity (m)	
			CMX	CMY	CRX	CRY	ex	ey
Model 1-0	GR	653,611.57	11.99	11.99	12.00	12.00	0.00	0.00
	1	650,527.94	11.99	11.99	12.00	12.00	0.00	0.00
	2	648,031.68	11.99	11.99	12.00	12.00	0.00	0.00
	3	648,031.68	11.99	11.99	12.00	12.00	0.00	0.00
	4	648,031.68	11.99	11.99	12.00	12.00	0.00	0.00
	5	628,052.38	11.99	11.99	12.00	12.00	0.00	0.00

Table 10: Center of mass and Center of rigidity for G+5 model with eccentricity

Model	Story	Mass (kg)	Center of Mass (m)		Center of Rigidity (m)		Eccentricity (m)	
			CMX	CMY	CRX	CRY	ex	ey
Model 1-1	GR	643,146.22	12.27	12.31	2.96	2.96	9.31	9.35
	1	639,677.14	12.27	12.31	3.16	1.62	9.11	10.70
	2	636,868.85	12.27	12.31	3.57	1.39	8.70	10.93
	3	636,868.85	12.27	12.31	4.12	1.63	8.15	10.68
	4	636,868.85	12.27	12.31	4.76	2.08	7.51	10.23
	5	619,427.11	12.28	12.32	5.50	2.63	6.78	9.69

The location of center of mass and center of rigidity of the buildings used to investigate the performance of structure subjected to high eccentricity in this section of case study is shown in the tables above. The eccentricity of both buildings has been quantified and clearly compared in the table below to show that the first building has no eccentricity while the second one is with high eccentricity.

Table 11: Eccentricity at each floor level for G+5 model

Story	Model 1-0 Without Eccentricity		Model 1-1 With Eccentricity	
	Eccentricity (m)			
	ex	ey	ex	ey
GR	0.00	0.00	9.31	9.35
1	0.00	0.00	9.11	10.70
2	0.00	0.00	8.70	10.93
3	0.00	0.00	8.15	10.68
4	0.00	0.00	7.51	10.23
5	0.00	0.00	6.78	9.69

Table 12: Period and Frequencies at each floor level for G+5 model

Mode	Model 1-0 Without Eccentricity		Model 1-1 With Eccentricity	
	Period (sec)	Frequency (cyc/sec)	Period (sec)	Frequency (cyc/sec)
1	0.741	1.350	1.27	0.79
2	0.648	1.544	0.71	1.42
3	0.585	1.708	0.41	2.41
4	0.248	4.031	0.39	2.57
5	0.151	6.633	0.24	4.20
6	0.146	6.828	0.18	5.63
7	0.136	7.330	0.17	6.07
8	0.110	9.097	0.13	7.86
9	0.089	11.224	0.11	9.06
10	0.079	12.608	0.11	9.36
11	0.066	15.083	0.10	10.11
12	0.064	15.678	0.09	11.71

The lowest natural frequency (f) of vibration of a structure corresponds to the longest time period (T) of vibration, as frequency and time period are inversely proportional ($T=1/f$). This is also referred to as the first mode of vibration or fundamental period of vibration. Structure will have multiple natural modes of vibration for which frequency will be higher and time period will be shorter than the fundamental period as can be observed from the table above. The analysis result on the G+5 model has shown that the fundamental period of the wall-frame structure without eccentricity (0.741sec) is lower than the fundamental period of the wall frame structure (1.27sec) with eccentricity.

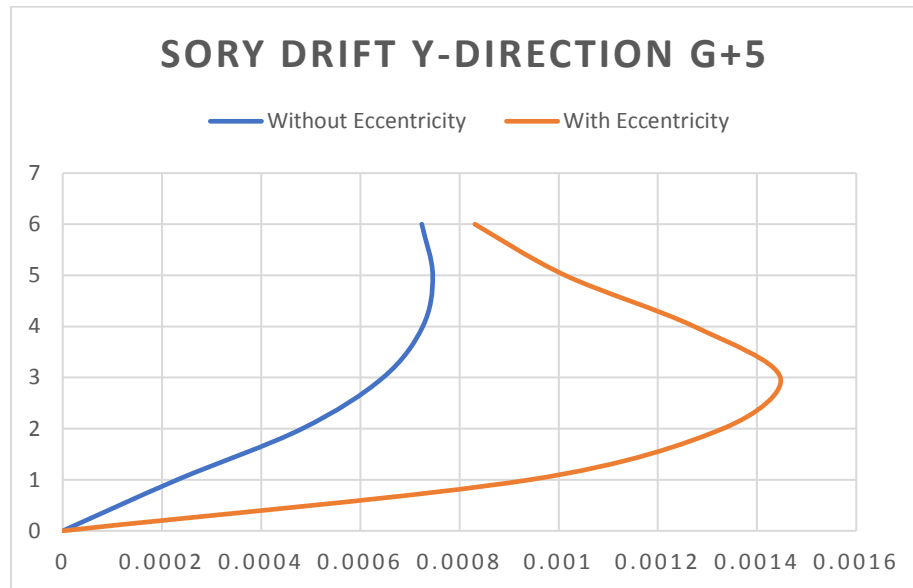
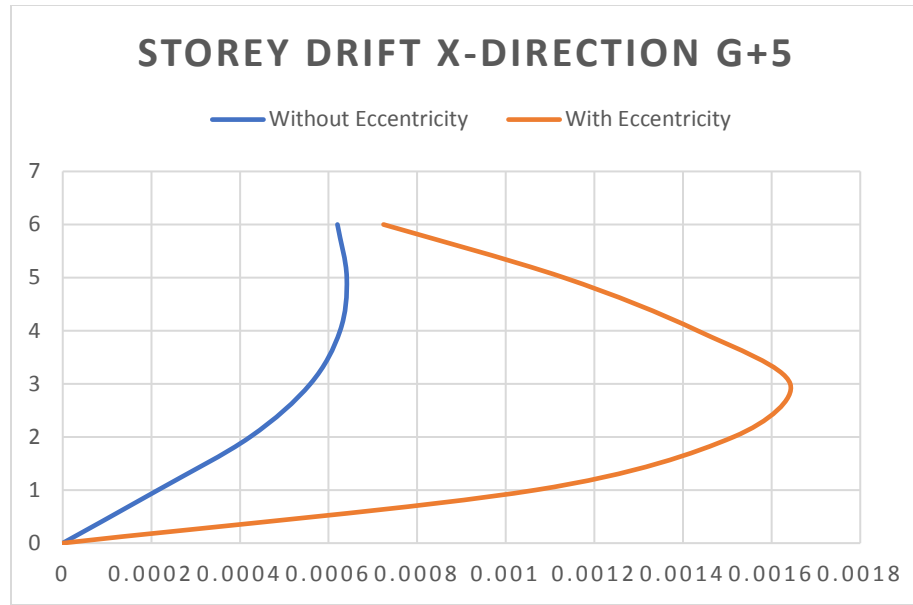
The modal diaphragm center of mass displacements has been investigated and compared from the analysis result of both models and the story drift of each model is calculated in the tables below.

Table 13: Story Diaphragm Displacements at each floor level for G+5 model

Story/ Diaphragm	Model 1-0 Without Eccentricity		Model 1-1 With Eccentricity	
	UX (mm)	UY (mm)	UX (mm)	UY (mm)
5	10.749	12.404	14.083	12.62
4	8.587	9.881	12.806	10.676
5	6.353	7.282	10.796	8.738
2	4.173	4.751	8.138	6.486
1	2.225	2.495	4.989	3.943
GR	0.749	0.809	2.049	1.614

Table 14: Story Drift at each floor level for G+5 model

Story/ Diaphragm	Model 1-0 Without Eccentricity		Model 1-1 With Eccentricity	
	Drift X (mm)	Drift Y (mm)	Drift X (mm)	Drift Y (mm)
5	0.00062	0.000724	0.000724	0.000831
4	0.000641	0.000746	0.001131	0.001013
3	0.000626	0.000726	0.001433	0.001273
2	0.000559	0.000646	0.001642	0.001447
1	0.000423	0.000483	0.001514	0.001331
GR	0.000215	0.000231	0.001067	0.000942



3.5.2 Case 2: G+10 Building (Model 2)

The effect of eccentricity on the performance of wall frame structures has been investigated in this section using eleven story G+10 building model. The effect of eccentricity on the performance of this short story structures is clearly shown using two eleven story building models with different wall frame structural layout. The first building model has a structural wall frame arrangement that will produce zero eccentricity and the second building the wall and frames are arranged in such a way to produce high story eccentricity. The loading and material properties of all structural elements used is briefly shown in the previous section.

The buildings laid out on a rectangular grid with a typical 6m width bays in the X and Y directions. For purpose of simplicity the beams, slab and shear walls have constant cross section throughout the building and the columns have been designed and the optimum section is used. The structures are hypothetical and have been chosen for the illustrative purpose. The member cross sections selected for this analysis were;

Table 15: The member cross sections used in G+10 models

Item No.	Structural Element	Section Name		Width (mm)	Height (mm)
		Model 1-0 Without Eccentricity	Model 1-1 With Eccentricity		
1	Column	CL 40X40	CL 40X40	400	400
		CL 50X50	CL 50X50	500	500
		CL 60X60	CL 60X60	600	600
		CL 70X70	CL 70X70	700	700
2	Beam	BM 30X50		300	500
				Thickness (mm)	
3	Wall	Wall 300mm		300	
4	Slab	Slab 200mm		200	

The structural models have then been modeled, analyzed and designed according to Ethiopian building code and standards EBCS EN:2014. The analyses and design results are compared with each other and briefly discussed to investigate the effect of eccentricity on the performance of building structure.

The cross-section design of the two building models has revealed that;

- The column immediately near the shear wall require a slightly bigger sections at the top story levels in both wall frame structures with eccentricity and without eccentricity.
- The other column, beam, slab and shear wall cross sections remain similar for both cases.
- The calculated maximum bending moments in the walls and core are less.

Table 16: Center of mass and Center of rigidity for G+10 model without eccentricity

Model	Story	Mass (kg)	Center of Mass (m)		Center of Rigidity (m)		Eccentricity (m)	
			CMX	CMY	CRX	CRY	ex	ey
Model 2-0	GR	693,943.38	12.00	12.00	12.00	12.00	0.00	0.00
	1	689,489.26	12.00	12.00	12.00	12.00	0.00	0.00
	2	678,084.76	12.00	12.00	12.00	12.00	0.00	0.00
	3	671,354.63	12.00	12.00	12.00	12.00	0.00	0.00
	4	667,585.76	12.00	12.00	12.00	12.00	0.00	0.00
	5	658,237.00	12.00	12.00	12.00	12.00	0.00	0.00
	6	651,335.56	12.00	12.00	12.00	12.00	0.00	0.00
	7	650,087.43	12.00	12.00	12.00	12.00	0.00	0.00
	8	650,087.43	12.00	12.00	12.00	12.00	0.00	0.00
	9	650,087.43	12.00	12.00	12.00	12.00	0.00	0.00
	10	628,566.32	12.00	12.00	12.00	12.00	0.00	0.00

Table 17: Center of mass and Center of rigidity for G+10 model with eccentricity

Model	Story	Mass (kg)	Center of Mass (m)		Center of Rigidity (m)		Eccentricity (m)	
			CMX	CMY	CRX	CRY	ex	ey
Model 2-1	GR	683,943.02	12.28	12.33	4.46	5.46	7.83	6.86
	1	679,005.56	12.29	12.33	4.13	3.17	8.15	9.15
	2	668,072.16	12.28	12.32	4.46	2.66	7.82	9.66
	3	661,296.15	12.28	12.32	4.92	2.76	7.35	9.56
	4	657,129.59	12.28	12.32	5.44	3.10	6.84	9.22
	5	647,780.83	12.28	12.32	5.92	3.51	6.36	8.81
	6	639,365.11	12.27	12.32	6.32	3.94	5.95	8.38
	7	636,868.85	12.27	12.31	6.67	4.32	5.60	7.99
	8	636,868.85	12.27	12.31	7.06	4.75	5.21	7.56
	9	636,868.85	12.27	12.31	7.49	5.23	4.78	7.08
	10	619,427.11	12.28	12.32	7.98	5.68	4.30	6.64

The location of center of mass and center of rigidity of the buildings used to investigate the performance of structure subjected to high eccentricity in this section of case study is shown in the tables above. The eccentricity of both buildings has been quantified and clearly compared in the table below to show that the first building has no eccentricity while the second one is with high eccentricity.

Table 18: Eccentricity at each floor level for G+10 model

Story	Model 2-0 Without Eccentricity		Model 2-1 With Eccentricity	
	Eccentricity (m)			
	ex	ey	ex	ey
GR	0.00	0.00	7.83	6.86
1	0.00	0.00	8.15	9.15
2	0.00	0.00	7.82	9.66
3	0.00	0.00	7.35	9.56
4	0.00	0.00	6.84	9.22
5	0.00	0.00	6.36	8.81
6	0.00	0.00	5.95	8.38
7	0.00	0.00	5.60	7.99
8	0.00	0.00	5.21	7.56
9	0.00	0.00	4.78	7.08
10	0.00	0.00	4.30	6.64

Table 19: Period and Frequencies at each floor level for G+10 model

Mode	Model 2-0 Without Eccentricity		Model 2-1 With Eccentricity	
	Period (sec)	Frequency (cyc/sec)	Period (sec)	Frequency (cyc/sec)
1	1.390	0.719	1.99	0.50
2	1.283	0.779	1.53	0.65
3	1.203	0.831	0.94	1.06
4	0.417	2.401	0.69	1.44
5	0.360	2.779	0.40	2.50
6	0.327	3.061	0.39	2.54
7	0.246	4.069	0.27	3.67
8	0.176	5.684	0.23	4.38
9	0.157	6.360	0.20	4.94
10	0.145	6.915	0.19	5.39

11	0.135	7.404	0.16	6.23
12	0.110	9.074	0.13	7.53

The lowest natural frequency (f) of vibration of a structure corresponds to the longest time period (T) of vibration, as frequency and time period are inversely proportional ($T=1/f$). This is also referred to as the first mode of vibration or fundamental period of vibration. Structure will have multiple natural modes of vibration for which frequency will be higher and time period will be shorter than the fundamental period as can be observed from the table above. The analysis result on the G+10 model has shown that the fundamental period of the wall-frame structure without eccentricity (1.39sec) is lower than the fundamental period of the wall frame structure (1.99sec) with eccentricity.

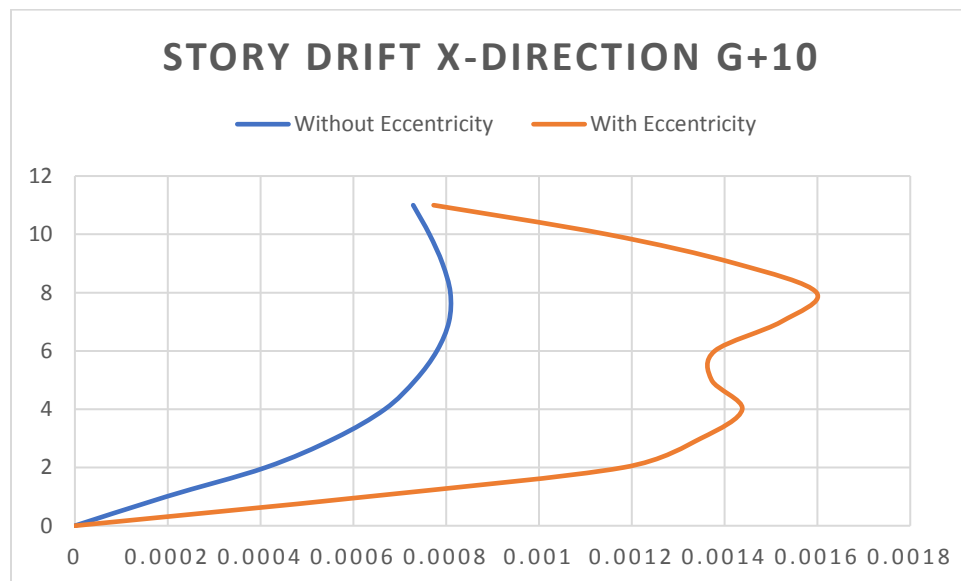
The modal diaphragm center of mass displacements has been investigated and compared from the analysis result of both models and the story drift of each model is calculated in the tables below.

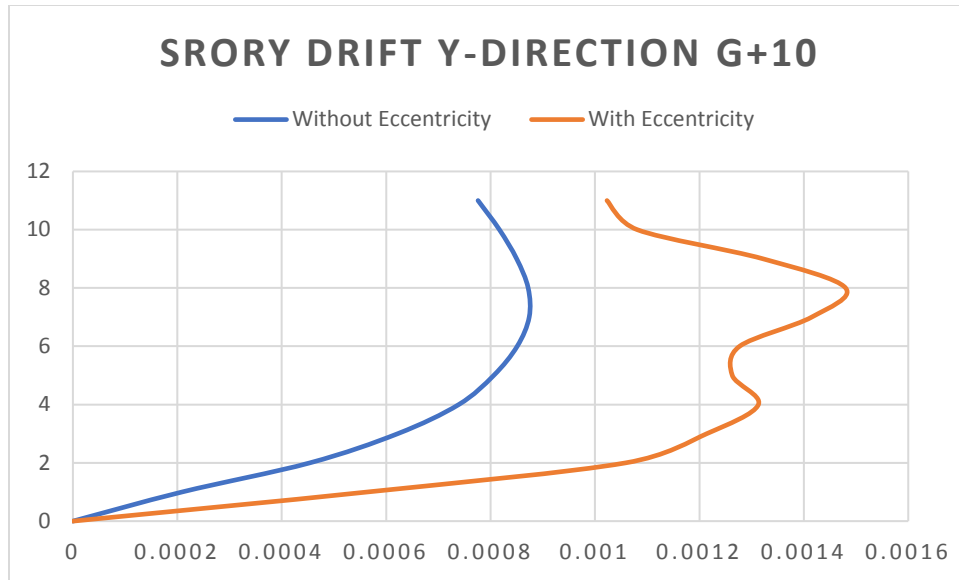
Table 20: Story Diaphragm Displacements at each floor level for G+10 model

Story/ Diaphragm	Model 2-0 Without Eccentricity		Model 2-1 With Eccentricity	
	UX (mm)	UY (mm)	UX (mm)	UY (mm)
10	24.934	26.96	27.376	30.392
9	22.444	24.336	25.652	27.497
8	19.847	21.584	23.44	24.369
7	17.156	18.718	20.827	21.1
6	14.4	15.761	17.898	17.718
5	11.64	12.777	15.006	14.446
4	8.952	9.851	12.218	11.37
3	6.409	7.06	9.4	8.451
2	4.084	4.489	6.447	5.623
1	2.128	2.318	3.67	3.116
GR	0.69	0.732	1.269	1.056

Table 21: Story Drift at each floor level for G+10 model

Story/ Diaphragm	Model 2-0 Without Eccentricity		Model 2-1 With Eccentricity	
	Drift X (mm)	Drift Y (mm)	Drift X (mm)	Drift Y (mm)
10	0.000729	0.000776	0.000773	0.001023
9	0.000764	0.000817	0.001147	0.001081
8	0.000792	0.00085	0.001423	0.001322
7	0.000809	0.000872	0.001598	0.00148
6	0.000806	0.000874	0.001522	0.001415
5	0.00078	0.000851	0.001379	0.001277
4	0.000734	0.000806	0.001372	0.001263
3	0.000668	0.000739	0.001438	0.001313
2	0.00056	0.000622	0.001349	0.001214
1	0.000411	0.000454	0.001182	0.001059
GR	0.000197	0.000209	0.000631	0.000564





3.5.3 Case 3: G+15 Building (Model 3)

The effect of eccentricity on the performance of wall frame structures has been investigated in this section using a sixteen story G+15 building model. The effect of eccentricity on the performance of this medium story structures is clearly shown using two sixteen story building models with different wall frame structural layout. The first building model has a structural wall frame arrangement that will produce zero eccentricity and the second building the wall and frames are arranged in such a way to produce high story eccentricity. The loading and material properties of all structural elements used is briefly shown in the previous section.

The buildings laid out on a rectangular grid with a typical 6m width bays in the X and Y directions. For purpose of simplicity the beams, slab and shear walls have constant cross section throughout the building and the columns have been designed and the optimum section is used. The structures are hypothetical and have been chosen for the illustrative purpose. The member cross sections selected for this analysis were;

Table 22: The member cross sections used in G+15 models

Item No.	Structural Element	Section Name		Width (mm)	Height (mm)
		Model 1-0 Without Eccentricity	Model 1-1 With Eccentricity		
1	Column	CL 40X40	CL 40X40	400	400
		CL 50X50	CL 50X50	500	500
		CL 60X60	CL 60X60	600	600
		CL 70X70	CL 70X70	700	700
		CL 80X80	CL 80X80	800	800
2	Beam	BM 30X50		300	500
				Thickness (mm)	
3	Wall	Wall 300mm		300	
4	Slab	Slab 200mm		200	

The structural models have then been modeled, analyzed and designed according to Ethiopian building code and standards EBCS EN:2014. The analyses and design results are compared with each other and briefly discussed to investigate the effect of eccentricity on the performance of building structure.

The cross-section design of the two building models has revealed that;

- The column immediately near the shear wall require a slightly bigger sections at the top story levels in both wall frame structures with eccentricity and without eccentricity.
- The other column, beam, slab and shear wall cross sections remain similar for both cases.

Table 23: Center of mass and Center of rigidity for G+15 model without eccentricity

Model	Story	Mass (kg)	Center of Mass (m)		Center of Rigidity (m)		Eccentricity (m)	
			CMX	CMY	CRX	CRY	ex	ey
Model 3-0	GR	720,643.63	12.00	12.00	12.00	12.00	0.00	0.00
	1	720,643.63	12.00	12.00	12.00	12.00	0.00	0.00
	2	711,092.97	12.01	12.01	12.00	12.00	0.00	0.00
	3	697,926.39	12.00	12.00	12.00	12.00	0.00	0.00
	4	693,943.38	12.00	12.00	12.00	12.00	0.00	0.00
	5	685,848.87	12.00	12.00	12.00	12.00	0.00	0.00
	6	674,738.05	12.00	12.00	12.00	12.00	0.00	0.00
	7	671,354.63	12.00	12.00	12.00	12.00	0.00	0.00
	8	670,883.52	12.00	12.00	12.00	12.00	0.00	0.00
	9	667,188.07	12.00	12.00	12.01	12.01	0.00	0.00
	10	658,237.00	12.00	12.00	12.01	12.01	0.00	0.00
	11	651,335.56	12.00	12.00	12.01	12.01	0.00	0.00
	12	650,087.43	12.00	12.00	12.01	12.01	0.00	0.00
	13	650,087.43	12.00	12.00	12.01	12.01	0.00	0.00
	14	650,087.43	12.00	12.00	12.01	12.00	0.00	0.00
	15	628,566.32	12.00	12.00	12.01	12.00	0.00	0.00

Table 24: Center of mass and Center of rigidity for G+15 model with eccentricity

Model	Story	Mass (kg)	Center of Mass (m)		Center of Rigidity (m)		Eccentricity (m)	
			CMX	CMY	CRX	CRY	ex	ey
Model 3-1	GR	710,701.40	12.29	12.33	5.36	6.48	6.93	5.85
	1	710,132.40	12.30	12.34	4.82	4.09	7.48	8.24
	2	701,138.50	12.29	12.33	5.09	3.48	7.21	8.86
	3	688,482.80	12.28	12.32	5.50	3.49	6.79	8.84
	4	683,943.02	12.28	12.33	5.94	3.74	6.35	8.59
	5	676,393.05	12.28	12.33	6.37	4.10	5.92	8.22
	6	665,634.02	12.28	12.32	6.74	4.50	5.54	7.82
	7	661,296.15	12.28	12.32	7.08	4.88	5.20	7.44
	8	660,898.46	12.28	12.32	7.41	5.29	4.87	7.03
	9	657,129.59	12.28	12.32	7.74	5.71	4.54	6.61
	10	647,780.83	12.28	12.32	8.02	6.09	4.25	6.23
	11	639,365.11	12.27	12.32	8.24	6.43	4.03	5.89
	12	636,868.85	12.27	12.31	8.42	6.71	3.85	5.60
	13	636,868.85	12.27	12.31	8.64	7.02	3.63	5.30
	14	636,868.85	12.27	12.31	8.90	7.36	3.37	4.96
	15	619,427.11	12.28	12.32	9.21	7.67	3.07	4.65

The location of center of mass and center of rigidity of the buildings used to investigate the performance of structure subjected to high eccentricity in this section of case study is shown in the tables above. The eccentricity of both buildings has been quantified and clearly compared in the table below to show that the first building has no eccentricity while the second one is with high eccentricity.

Table 25: Eccentricity at each floor level for G+15 model

Story	Model 3-0 Without Eccentricity		Model 3-1 With Eccentricity	
	Eccentricity (m)			
	ex	ey	ex	ey
GR	0.00	0.00	6.93	5.85
1	0.00	0.00	7.48	8.24
2	0.00	0.00	7.21	8.86
3	0.00	0.00	6.79	8.84
4	0.00	0.00	6.35	8.59
5	0.00	0.00	5.92	8.22
6	0.00	0.00	5.54	7.82
7	0.00	0.00	5.20	7.44
8	0.00	0.00	4.87	7.03
9	0.00	0.00	4.54	6.61
10	0.00	0.00	4.25	6.23
11	0.00	0.00	4.03	5.89
12	0.00	0.00	3.85	5.60
13	0.00	0.00	3.63	5.30
14	0.00	0.00	3.37	4.96
15	0.00	0.00	3.07	4.65

Table 26: Period and Frequencies at each floor level for G+15 model

Mode	Model 3-0 Without Eccentricity		Model 3-1 With Eccentricity	
	Period (sec)	Frequency (cyc/sec)	Period (sec)	Frequency (cyc/sec)
1	2.160	0.463	2.65	0.38
2	2.016	0.496	2.26	0.44
3	1.701	0.588	1.53	0.66
4	0.619	1.615	0.94	1.07
5	0.589	1.697	0.66	1.51
6	0.565	1.771	0.54	1.84
7	0.353	2.836	0.39	2.54
8	0.282	3.543	0.37	2.69
9	0.256	3.912	0.31	3.24
10	0.247	4.053	0.28	3.55
11	0.191	5.231	0.22	4.51
12	0.163	6.140	0.19	5.31

The lowest natural frequency (f) of vibration of a structure corresponds to the longest time period (T) of vibration, as frequency and time period are inversely proportional ($T=1/f$). This is also referred to as the first mode of vibration or fundamental period of vibration. Structure will have multiple natural modes of vibration for which frequency will be higher and time period will be shorter than the fundamental period as can be observed from the table above. The analysis result on the G+15 model has shown that the fundamental period of the wall-frame structure without eccentricity (2.16sec) is lower than the fundamental period of the wall frame structure (2.65sec) with eccentricity.

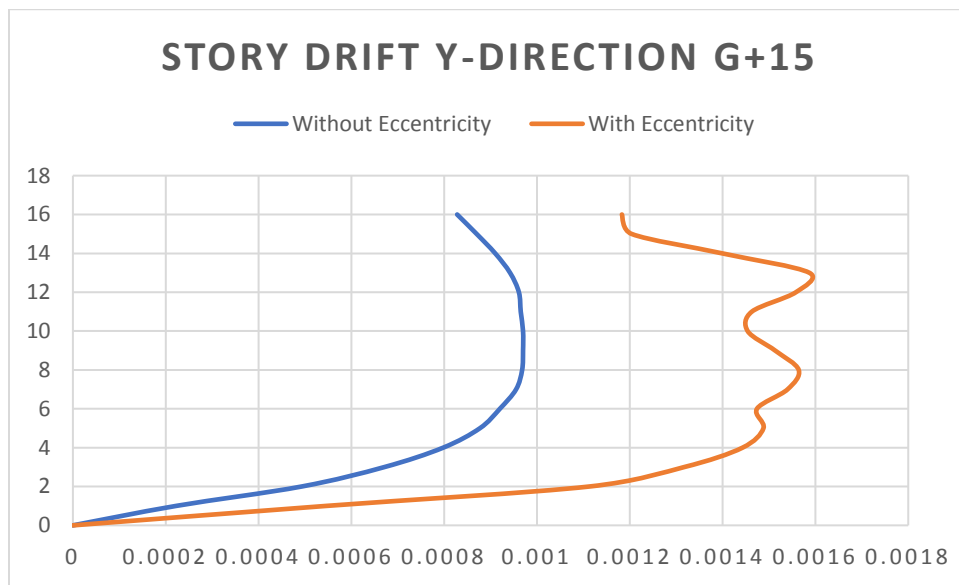
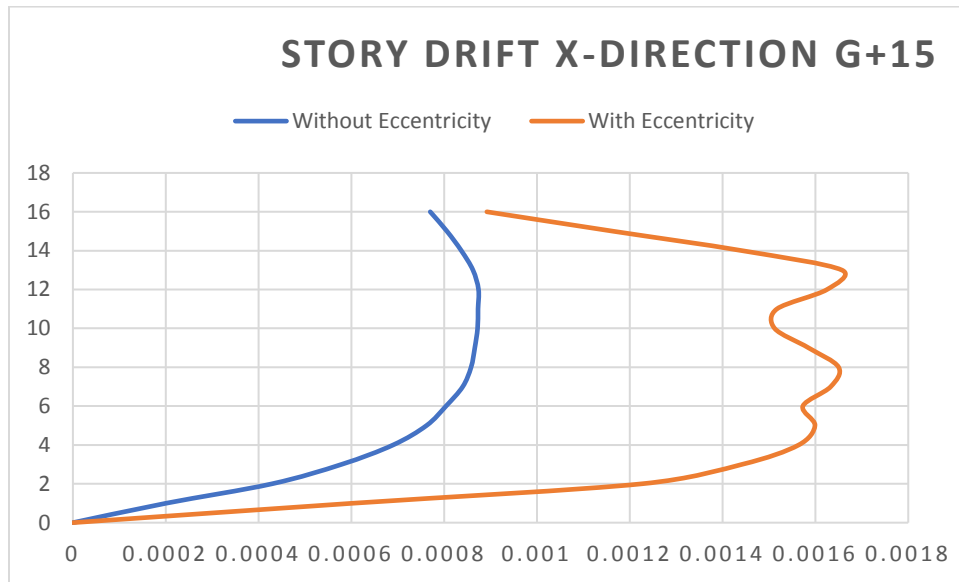
The modal diaphragm center of mass displacements has been investigated and compared from the analysis result of both models and the story drift of each model is calculated in the tables below.

Table 27: Story Diaphragm Displacements at each floor level for G+15 model

Story/ Diaphragm	Model 3-0 Without Eccentricity		Model 3-1 With Eccentricity	
	UX (mm)	UY (mm)	UX (mm)	UY (mm)
15	39.399	43.813	50.61	55.201
14	36.932	41.207	48.231	51.864
13	34.384	38.499	45.383	48.239
12	31.754	35.683	42.141	44.403
11	29.044	32.756	38.558	40.367
10	26.275	29.741	35.012	36.375
9	23.473	26.666	31.566	32.469
8	20.641	23.528	28.133	28.603
7	17.775	20.325	24.561	24.682
6	14.89	17.074	20.882	20.734
5	12.035	13.829	17.274	16.892
4	9.267	10.662	13.782	13.219
3	6.625	7.622	10.276	9.647
2	4.226	4.847	6.904	6.325
1	2.203	2.504	3.828	3.417
GR	0.702	0.778	1.255	1.099

Table 28: Story Drift at each floor level for G+15 model

Story/ Diaphragm	Model 3-0 Without Eccentricity		Model 3-1 With Eccentricity	
	Drift X (mm)	Drift Y (mm)	Drift X (mm)	Drift Y (mm)
15	0.00077	0.000828	0.000892	0.001183
14	0.000806	0.00087	0.001164	0.001204
13	0.000837	0.00091	0.001442	0.001399
12	0.000862	0.000942	0.001657	0.001587
11	0.000874	0.000961	0.001625	0.001558
10	0.000873	0.000965	0.001517	0.001463
9	0.000872	0.00097	0.001512	0.001454
8	0.000866	0.00097	0.001585	0.001513
7	0.000858	0.000968	0.00165	0.001564
6	0.00084	0.000955	0.001633	0.00154
5	0.000804	0.00092	0.001573	0.001474
4	0.000762	0.000877	0.001599	0.001488
3	0.00069	0.000798	0.001564	0.001443
2	0.000579	0.000671	0.001442	0.001317
1	0.000429	0.000493	0.001226	0.001113
GR	0.000201	0.000222	0.000604	0.000545



3.5.4 Case 4: G+20 Building (Model 4)

The effect of eccentricity on the performance of wall frame structures has been investigated in this section using a twenty-one story G+20 building model. The effect of eccentricity on the performance of this medium story structures is clearly shown using two twenty-one story building models with different wall frame structural layout. The first building model has a structural wall frame arrangement that will produce zero eccentricity and the second building the wall and frames are arranged in such a way to produce high story eccentricity. The loading and material properties of all structural elements used is briefly shown in the previous section.

The buildings laid out on a rectangular grid with a typical 6m width bays in the X and Y directions. For purpose of simplicity the beams, slab and shear walls have constant cross section throughout the building and the columns have been designed and the optimum section is used. The structures are hypothetical and have been chosen for the illustrative purpose. The member cross sections selected for this analysis were;

Table 29: The member cross sections used in G+20 models

Item No.	Structural Element	Section Name		Width (mm)	Height (mm)
		Model 1-0 Without Eccentricity	Model 1-1 With Eccentricity		
1	Column	CL 40X40	CL 40X40	400	400
		CL 50X50	CL 50X50	500	500
		CL 60X60	CL 60X60	600	600
		CL 70X70	CL 70X70	700	700
		CL 80X80	CL 80X80	800	800
		CL 90X90	CL 90X90	900	900
		CL 100X100	CL 100X100	1000	1000
2	Beam	BM 30X50		300	500
				Thickness (mm)	
3	Wall	Wall 300mm		300	
4	Slab	Slab 200mm		200	

The structural models have then been modeled, analyzed and designed according to Ethiopian building code and standards EBCS EN:2014. The analyses and design results are compared with each other and briefly discussed to investigate the effect of eccentricity on the performance of building structure.

The cross-section design of the two building models has revealed that;

- The column immediately near the shear wall require a slightly bigger sections at the top story levels in both wall frame structures with eccentricity and without eccentricity.
- The other column, beam, slab and shear wall cross sections remain similar for both cases.

Table 30: Center of mass and Center of rigidity for G+20 model without eccentricity

Model	Story	Mass (kg)	Center of Mass (m)		Center of Rigidity (m)		Eccentricity (m)	
			CMX	CMY	CRX	CRY	ex	ey
Model 4-0	GR	761,519.97	12.01	12.02	12.04	12.05	0.00	0.00
	1	755,897.26	12.01	12.01	12.02	12.03	0.00	0.00
	2	741,176.64	12.00	12.00	12.02	12.02	0.00	0.00
	3	730,977.43	12.01	12.01	12.02	12.02	0.00	0.00
	4	729,594.70	12.01	12.02	12.01	12.02	0.00	0.00
	5	716,220.10	12.01	12.02	12.01	12.02	0.00	0.00
	6	704,136.46	12.00	12.01	12.01	12.01	0.00	0.00
	7	702,992.34	12.01	12.01	12.01	12.01	0.00	0.00
	8	697,926.39	12.00	12.00	12.01	12.01	0.00	0.00
	9	693,943.38	12.00	12.00	12.01	12.01	0.00	0.00
	10	686,405.64	12.00	12.00	12.01	12.01	0.00	0.00
	11	675,221.39	12.00	12.00	12.01	12.01	0.00	0.00
	12	671,354.63	12.00	12.00	12.01	12.01	0.00	0.00
	13	670,883.52	12.00	12.00	12.01	12.01	0.00	0.00
	14	667,188.07	12.00	12.00	12.01	12.01	0.00	0.00
	15	658,237.00	12.00	12.00	12.01	12.01	0.00	0.00
	16	651,335.56	12.00	12.00	12.01	12.01	0.00	0.00
	17	650,087.43	12.00	12.00	12.01	12.01	0.00	0.00
	18	650,087.43	12.00	12.00	12.01	12.01	0.00	0.00
	19	650,087.43	12.00	12.00	12.01	12.01	0.00	0.00
	20	628,566.32	12.00	12.00	12.01	12.01	0.00	0.00

Table 31: Center of mass and Center of rigidity for G+20 model with eccentricity

Model	Story	Mass (kg)	Center of Mass (m)		Center of Rigidity (m)		Eccentricity (m)	
			CMX	CMY	CRX	CRY	ex	ey
Model 4-1	GR	753,104.26	12.30	12.35	6.65	7.74	5.66	4.61
	1	746,741.23	12.30	12.34	5.70	5.18	6.60	7.16
	2	732,663.02	12.29	12.33	5.75	4.30	6.54	8.03
	3	723,060.36	12.29	12.33	6.05	4.14	6.24	8.19
	4	720,949.55	12.29	12.34	6.40	4.27	5.89	8.07
	5	707,477.05	12.28	12.34	6.73	4.51	5.56	7.83
	6	695,904.30	12.27	12.32	7.02	4.82	5.25	7.51
	7	694,760.17	12.28	12.32	7.33	5.15	4.96	7.17
	8	689,051.80	12.28	12.32	7.60	5.48	4.68	6.84
	9	684,499.79	12.28	12.32	7.85	5.81	4.43	6.52
	10	677,433.16	12.27	12.32	8.08	6.12	4.19	6.19
	11	666,117.37	12.27	12.31	8.28	6.42	3.99	5.89
	12	661,767.26	12.27	12.32	8.46	6.70	3.82	5.61
	13	661,296.15	12.28	12.32	8.65	6.99	3.63	5.33
	14	657,129.59	12.28	12.32	8.84	7.27	3.44	5.05
	15	647,780.83	12.28	12.32	9.01	7.54	3.27	4.78
	16	639,365.11	12.27	12.32	9.14	7.77	3.13	4.55
	17	636,868.85	12.27	12.31	9.25	7.96	3.02	4.35
	18	636,868.85	12.27	12.31	9.38	8.18	2.89	4.14
	19	636,868.85	12.27	12.31	9.56	8.42	2.71	3.90
	20	619,427.11	12.28	12.32	9.78	8.65	2.50	3.68

The location of center of mass and center of rigidity of the buildings used to investigate the performance of structure subjected to high eccentricity in this section of case study is shown in the tables above. The eccentricity of both buildings has been quantified and clearly compared in the table below to show that the first building has no eccentricity while the second one is with high eccentricity.

Table 32: Eccentricity at each floor level for G+20 model

Story	Model 4-0 Without Eccentricity		Model 4-1 With Eccentricity	
	Eccentricity (m)			
	ex	ey	ex	ey
GR	0.00	0.00	5.66	4.61
1	0.00	0.00	6.60	7.16
2	0.00	0.00	6.54	8.03
3	0.00	0.00	6.24	8.19
4	0.00	0.00	5.89	8.07
5	0.00	0.00	5.56	7.83
6	0.00	0.00	5.25	7.51
7	0.00	0.00	4.96	7.17
8	0.00	0.00	4.68	6.84
9	0.00	0.00	4.43	6.52
10	0.00	0.00	4.19	6.19
11	0.00	0.00	3.99	5.89
12	0.00	0.00	3.82	5.61
13	0.00	0.00	3.63	5.33
14	0.00	0.00	3.44	5.05
15	0.00	0.00	3.27	4.78
16	0.00	0.00	3.13	4.55
17	0.00	0.00	3.02	4.35
18	0.00	0.00	2.89	4.14
19	0.00	0.00	2.71	3.90
20	0.00	0.00	2.50	3.68

Table 33: Period and Frequencies at each floor level for G+20 model

Mode	Model 4-0 Without Eccentricity		Model 4-1 With Eccentricity	
	Period (sec)	Frequency (cyc/sec)	Period (sec)	Frequency (cyc/sec)
1	2.936	0.341	3.40	0.29
2	2.756	0.363	3.04	0.33
3	2.186	0.457	2.13	0.47
4	0.892	1.121	1.19	0.84
5	0.821	1.217	0.94	1.06
6	0.760	1.316	0.70	1.43
7	0.455	2.200	0.59	1.70

8	0.427	2.341	0.48	2.09
9	0.387	2.586	0.46	2.19
10	0.321	3.115	0.36	2.78
11	0.250	4.000	0.29	3.50
12	0.245	4.086	0.28	3.64

The lowest natural frequency (f) of vibration of a structure corresponds to the longest time period (T) of vibration, as frequency and time period are inversely proportional ($T=1/f$). This is also referred to as the first mode of vibration or fundamental period of vibration. Structure will have multiple natural modes of vibration for which frequency will be higher and time period will be shorter than the fundamental period as can be observed from the table above. The analysis result on the G+20 model has shown that the fundamental period of the wall-frame structure without eccentricity (2.936sec) is lower than the fundamental period of the wall frame structure (3.40sec) with eccentricity.

The modal diaphragm center of mass displacements has been investigated and compared from the analysis result of both models and the story drift of each model is calculated in the tables below.

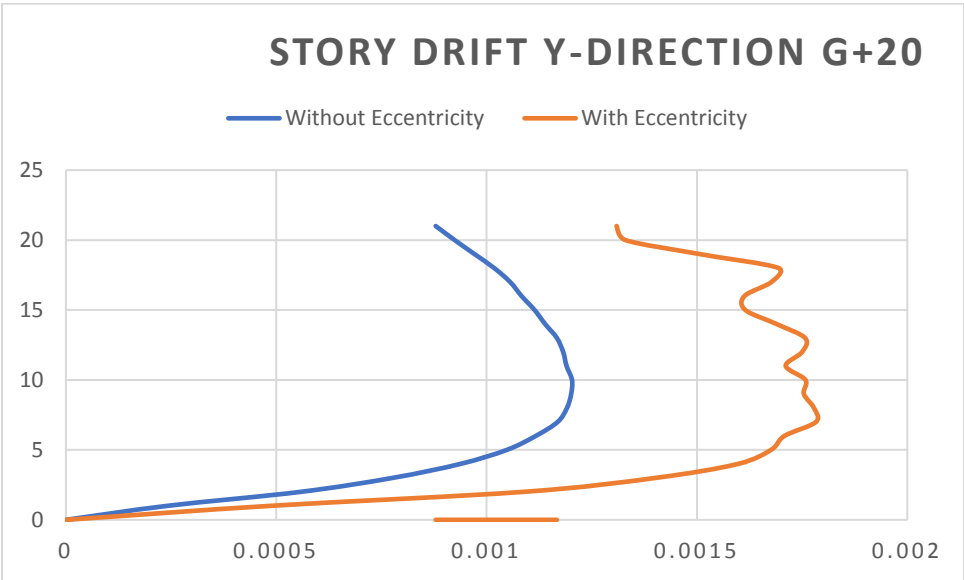
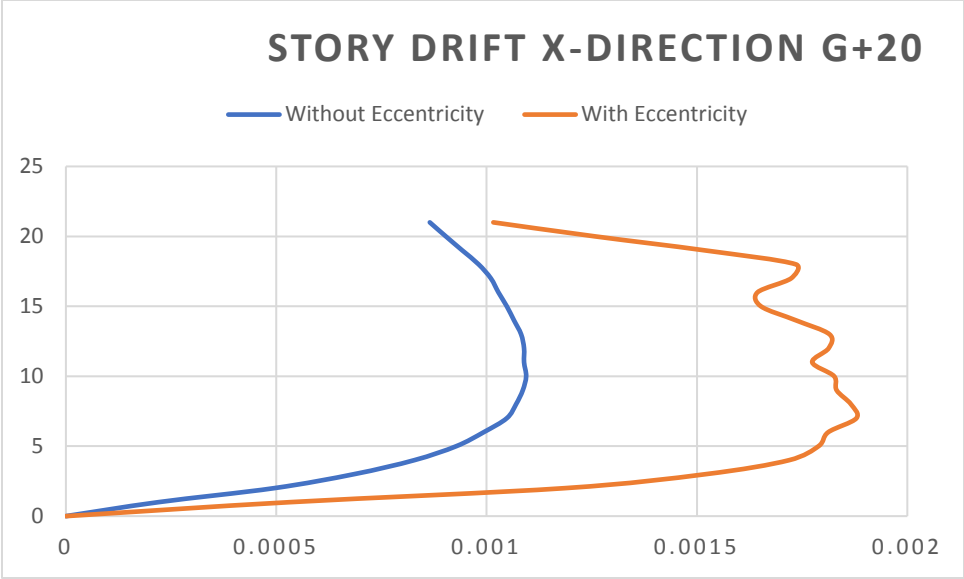
Table 34: Story Diaphragm Displacements at each floor level for G+20 model

Story/ Diaphragm	Model 4-0 Without Eccentricity		Model 4-1 With Eccentricity	
	UX (mm)	UY (mm)	UX (mm)	UY (mm)
20th	64.918	70.247	81.178	88.357
19th	62.156	67.479	78.214	84.512
18th	59.301	64.593	74.798	80.359
17th	56.337	61.569	70.999	75.97
16th	53.254	58.39	66.85	71.348
15th	50.064	55.068	62.712	66.734
14th	46.791	51.625	58.622	62.144
13th	43.429	48.055	54.506	57.536
12th	39.968	44.349	50.213	52.803
11th	36.412	40.512	45.767	47.96
10th	32.797	36.583	41.353	43.135
9th	29.146	32.589	37.016	38.372
8th	25.444	28.517	32.592	33.554
7th	21.745	24.423	28.198	28.792
6th	18.065	20.33	23.784	24.05
5th	14.45	16.293	19.396	19.387
4th	11.003	12.42	15.171	14.955
3rd	7.771	8.773	11.052	10.722

2nd	4.879	5.497	7.168	6.833
1st	2.49	2.785	3.777	3.534
GR	0.766	0.84	1.165	1.077

Table 35: Story Drift at each floor level for G+20 model

Story/ Diaphragm	Model 4-0 Without Eccentricity		Model 4-1 With Eccentricity	
	Drift X (mm)	Drift Y (mm)	Drift X (mm)	Drift Y (mm)
20th	0.000865	0.000879	0.001016	0.001309
19th	0.000903	0.000924	0.001257	0.00133
18th	0.000942	0.000971	0.001518	0.001506
17th	0.000981	0.001018	0.001735	0.001694
16th	0.00101	0.001056	0.001724	0.001677
15th	0.001028	0.001083	0.001644	0.001613
14th	0.001048	0.001114	0.001652	0.001615
13th	0.001065	0.001139	0.001735	0.001689
12th	0.001082	0.001167	0.001815	0.001757
11th	0.001089	0.001182	0.001813	0.00175
10th	0.001089	0.00119	0.001774	0.001711
9th	0.001094	0.001203	0.001826	0.001758
8th	0.001086	0.001201	0.001832	0.001754
7th	0.00107	0.001191	0.001866	0.001778
6th	0.001048	0.001169	0.001879	0.001783
5th	0.000993	0.001117	0.001812	0.001708
4th	0.000928	0.001048	0.001789	0.001676
3rd	0.000828	0.000938	0.001719	0.001598
2nd	0.000685	0.000777	0.001521	0.001402
1st	0.000495	0.000558	0.001186	0.001088
GR	0.00022	0.000241	0.000533	0.000486



3.5.5 Case 5: G+30 Building (Model 5)

The effect of eccentricity on the performance of wall frame structures has been investigated in this section using a thirty-one story G+30 building model. The effect of eccentricity on the performance of this medium story structures is clearly shown using two thirty-one story building models with different wall frame structural layout. The first building model has a structural wall frame arrangement that will produce zero eccentricity and the second building the wall and frames are arranged in such a way to produce high story eccentricity. The loading and material properties of all structural elements used is briefly shown in the previous section.

The buildings laid out on a rectangular grid with a typical 6m width bays in the X and Y directions. For purpose of simplicity the beams, slab and shear walls have constant cross section throughout the building and the columns have been designed and the optimum section is used. The structures are hypothetical and have been chosen for the illustrative purpose. The member cross sections selected for this analysis were;

Table 36: The member cross sections used in G+30 models

Item No.	Structural Element	Section Name		Width (mm)	Height (mm)
		Model 1-0 Without Eccentricity	Model 1-1 With Eccentricity		
1	Column	CL 40X40	CL 40X40	400	400
		CL 50X50	CL 50X50	500	500
		CL 60X60	CL 60X60	600	600
		CL 70X70	CL 70X70	700	700
		CL 80X80	CL 80X80	800	800
		CL 90X90	CL 90X90	900	900
		CL 100X100	CL 100X100	1000	1000
		CL 110X110	CL 110X110	1100	1100
		CL 120X120	CL 120X120	1200	1200
2	Beam	BM 40X60		400	600
				Thickness (mm)	
3	Wall	Wall 400mm		400	
4	Slab	Slab 200mm		200	

The structural models have then been modeled, analyzed and designed according to Ethiopian building code and standards EBCS EN:2014. The analyses and design results are compared with each other and briefly discussed to investigate the effect of eccentricity on the performance of building structure.

The cross-section design of the two building models has revealed that;

- The column immediately near the shear wall require a slightly bigger sections at the top story levels in both wall frame structures with eccentricity and without eccentricity.
- The other column, beam, slab and shear wall cross sections remain similar for both cases.

Table 37: Center of mass and Center of rigidity for G+30 model without eccentricity

Model	Story	Mass (kg)	Center of Mass (m)		Center of Rigidity (m)		Eccentricity (m)	
			CMX	CMY	CRX	CRY	ex	ey
Model 5-0	GR	949,589.09	12.00	11.99	12.03	12.03	0.00	0.00
	1	947,853.94	11.99	11.99	12.02	12.02	0.00	0.00
	2	947,853.94	11.99	11.99	12.01	12.01	0.00	0.00
	3	932,564.31	12.00	11.99	12.01	12.01	0.00	0.00
	4	914,187.40	12.00	12.00	12.01	12.01	0.00	0.00
	5	908,479.02	12.00	11.99	12.01	12.01	0.00	0.00
	6	906,915.19	11.99	11.99	12.01	12.00	0.00	0.00
	7	893,081.72	12.00	11.99	12.01	12.00	0.00	0.00
	8	877,402.97	12.00	12.00	12.01	12.00	0.00	0.00
	9	872,176.72	11.99	11.99	12.01	12.00	0.00	0.00
	10	870,087.93	11.99	11.99	12.01	12.00	0.00	0.00
	11	870,087.93	11.99	11.99	12.01	12.00	0.00	0.00
	12	855,526.39	11.99	11.99	12.01	12.00	0.00	0.00
	13	840,542.68	11.99	11.99	12.00	12.00	0.00	0.00
	14	837,982.78	11.99	11.99	12.00	12.00	0.00	0.00
	15	826,451.01	11.99	11.99	12.00	12.00	0.00	0.00
	16	815,654.67	12.00	12.00	12.00	12.00	0.00	0.00
	17	811,392.66	11.99	11.99	12.01	12.00	0.00	0.00
	18	808,767.91	11.99	11.99	12.01	12.00	0.00	0.00
	19	808,767.91	11.99	11.99	12.00	12.00	0.00	0.00
	20	808,767.91	11.99	11.99	12.00	12.00	0.00	0.00
	21	795,405.55	11.99	11.99	12.00	12.00	0.00	0.00
	22	784,275.14	11.99	11.99	12.00	12.00	0.00	0.00
	23	784,275.14	11.99	11.99	12.00	12.00	0.00	0.00

	24	772,968.53	11.99	11.99	12.00	12.00	0.00	0.00
	25	763,893.87	11.99	11.99	12.00	12.00	0.00	0.00
	26	763,893.87	11.99	11.99	12.00	12.00	0.00	0.00
	27	763,893.87	11.99	11.99	12.00	12.00	0.00	0.00
	28	763,893.87	11.99	11.99	12.00	12.00	0.00	0.00
	29	763,893.87	11.99	11.99	12.00	12.00	0.00	0.00
	30	709,073.93	11.99	11.99	12.00	12.00	0.00	0.00

Table 38: Center of mass and Center of rigidity for G+30 model with eccentricity

Model	Story	Mass (kg)	Center of Mass (m)		Center of Rigidity (m)		Eccentricity (m)	
			CMX	CMY	CRX	CRY	ex	ey
Model 5-1	GR	892,888.38	12.29	12.32	7.95	8.78	4.34	3.53
	1	890,168.18	12.29	12.31	6.99	6.61	5.30	5.70
	2	889,300.61	12.29	12.31	6.96	5.74	5.33	6.57
	3	874,910.37	12.29	12.31	7.14	5.49	5.15	6.82
	4	857,359.43	12.29	12.31	7.38	5.52	4.91	6.79
	5	851,651.06	12.28	12.31	7.63	5.69	4.65	6.62
	6	849,187.83	12.28	12.30	7.86	5.91	4.41	6.39
	7	835,386.18	12.28	12.30	8.07	6.15	4.20	6.15
	8	820,447.74	12.27	12.30	8.26	6.40	4.01	5.90
	9	815,221.49	12.27	12.30	8.43	6.64	3.83	5.66
	10	812,318.97	12.27	12.29	8.60	6.87	3.67	5.42
	11	811,622.71	12.27	12.29	8.75	7.10	3.51	5.19
	12	797,789.24	12.26	12.29	8.88	7.30	3.38	4.98
	13	783,460.19	12.25	12.28	9.00	7.50	3.25	4.79
	14	780,172.21	12.25	12.28	9.12	7.68	3.14	4.60
	15	768,672.26	12.25	12.28	9.22	7.85	3.03	4.44
	16	758,444.92	12.25	12.28	9.31	8.01	2.94	4.28
	17	754,182.91	12.25	12.28	9.41	8.16	2.84	4.12
	18	750,915.74	12.25	12.28	9.50	8.30	2.75	3.97
	19	750,390.79	12.25	12.28	9.59	8.44	2.66	3.83
	20	750,390.79	12.25	12.28	9.68	8.59	2.57	3.69
	21	737,028.43	12.24	12.27	9.76	8.72	2.48	3.55
	22	725,942.08	12.24	12.27	9.82	8.84	2.42	3.43
	23	725,942.08	12.24	12.27	9.89	8.96	2.34	3.31
	24	714,635.46	12.23	12.26	9.97	9.08	2.26	3.19
	25	705,604.86	12.23	12.26	10.00	9.16	2.23	3.10
	26	705,604.86	12.23	12.26	10.06	9.26	2.17	3.00
	27	705,604.86	12.23	12.26	10.13	9.37	2.10	2.89

	28	705,604.86	12.23	12.26	10.22	9.51	2.01	2.75
	29	705,604.86	12.23	12.26	10.33	9.67	1.90	2.59
	30	678,580.34	12.24	12.27	10.49	9.83	1.75	2.44

The location of center of mass and center of rigidity of the buildings used to investigate the performance of structure subjected to high eccentricity in this section of case study is shown in the tables above. The eccentricity of both buildings has been quantified and clearly compared in the table below to show that the first building has no eccentricity while the second one is with high eccentricity.

Table 39: Eccentricity at each floor level for G+30 model

Story	Model 5-0 Without Eccentricity		Model 5-1 With Eccentricity	
	Eccentricity (m)			
	ex	ey	ex	ey
GR	0.00	0.00	4.34	3.53
1	0.00	0.00	5.30	5.70
2	0.00	0.00	5.33	6.57
3	0.00	0.00	5.15	6.82
4	0.00	0.00	4.91	6.79
5	0.00	0.00	4.65	6.62
6	0.00	0.00	4.41	6.39
7	0.00	0.00	4.20	6.15
8	0.00	0.00	4.01	5.90
9	0.00	0.00	3.83	5.66
10	0.00	0.00	3.67	5.42
11	0.00	0.00	3.51	5.19
12	0.00	0.00	3.38	4.98
13	0.00	0.00	3.25	4.79
14	0.00	0.00	3.14	4.60
15	0.00	0.00	3.03	4.44
16	0.00	0.00	2.94	4.28
17	0.00	0.00	2.84	4.12
18	0.00	0.00	2.75	3.97
19	0.00	0.00	2.66	3.83
20	0.00	0.00	2.57	3.69
21	0.00	0.00	2.48	3.55
22	0.00	0.00	2.42	3.43
23	0.00	0.00	2.34	3.31
24	0.00	0.00	2.26	3.19

25	0.00	0.00	2.23	3.10
26	0.00	0.00	2.17	3.00
27	0.00	0.00	2.10	2.89
28	0.00	0.00	2.01	2.75
29	0.00	0.00	1.90	2.59
30	0.00	0.00	1.75	2.44

Table 40: Period and Frequencies at each floor level for G+30 model

Mode	Model 5-0 Without Eccentricity		Model 5-1 With Eccentricity	
	Period (sec)	Frequency (cyc/sec)	Period (sec)	Frequency (cyc/sec)
1	3.673	0.272	3.90	0.26
2	3.521	0.284	3.68	0.27
3	2.589	0.386	2.68	0.37
4	1.196	0.836	1.35	0.74
5	1.130	0.885	1.21	0.83
6	0.915	1.093	0.85	1.18
7	0.623	1.606	0.78	1.28
8	0.576	1.735	0.64	1.56
9	0.549	1.821	0.54	1.85
10	0.387	2.584	0.42	2.41
11	0.386	2.590	0.41	2.46
12	0.353	2.834	0.40	2.48

The lowest natural frequency (f) of vibration of a structure corresponds to the longest time period (T) of vibration, as frequency and time period are inversely proportional ($T=1/f$). This is also referred to as the first mode of vibration or fundamental period of vibration. Structure will have multiple natural modes of vibration for which frequency will be higher and time period will be shorter than the fundamental period as can be observed from the table above. The analysis result on the G+30 model has shown that the fundamental period of the wall-frame structure without eccentricity (3.673sec) is lower than the fundamental period of the wall frame structure (3.90sec) with eccentricity.

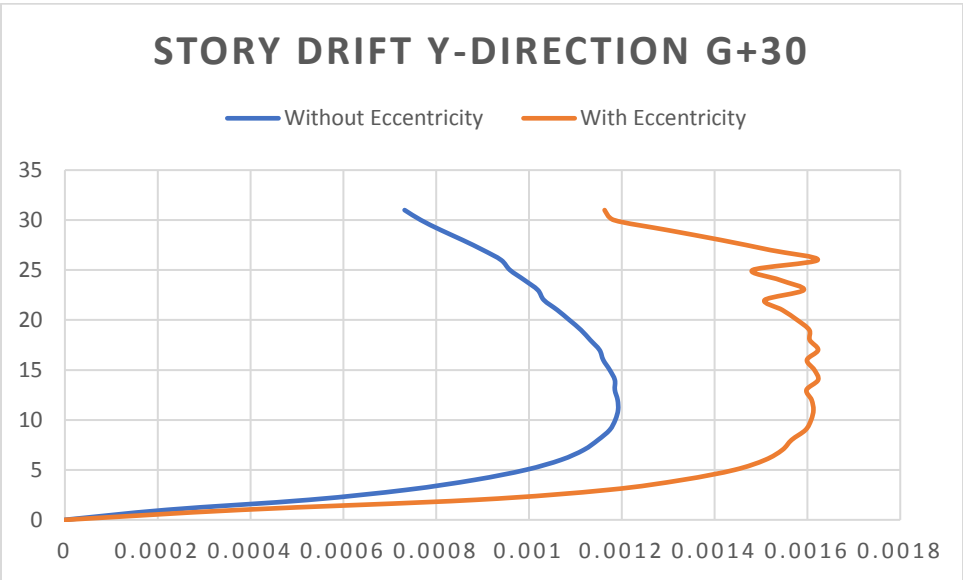
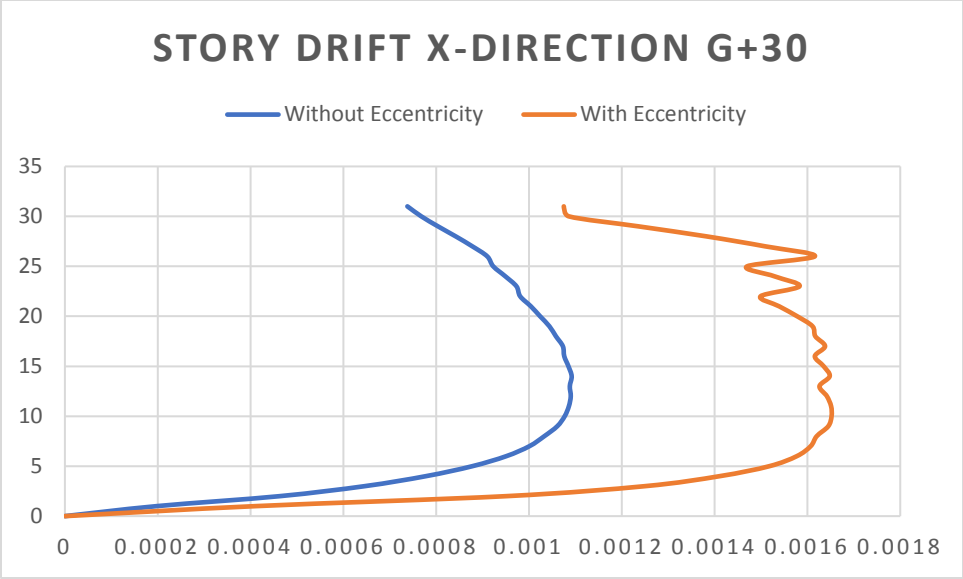
The modal diaphragm center of mass displacements has been investigated and compared from the analysis result of both modes and the story drift of each mode is calculated in the tables below.

Table 41: Story Diaphragm Displacements at each floor level for G+30 model

Story/ Diaphragm	Model 5-0 Without Eccentricity		Model 5-1 With Eccentricity	
	UX (mm)	UY (mm)	UX (mm)	UY (mm)
30th	94.819	101.997	125.248	131.252
29th	92.485	99.698	122.029	127.594
28th	90.076	97.309	118.495	123.711
27th	87.569	94.798	114.721	119.647
26th	84.95	92.144	110.722	115.402
25th	82.214	89.344	106.514	110.987
24th	79.365	86.406	102.097	106.401
23rd	76.454	83.383	97.931	101.978
22nd	73.439	80.235	93.605	97.428
21st	70.324	76.966	89.145	92.771
20th	67.157	73.626	84.854	88.228
19th	63.895	70.171	80.448	83.59
18th	60.546	66.609	75.96	78.886
17th	57.115	62.943	71.416	74.131
16th	53.623	59.198	66.875	69.373
15th	50.064	55.37	62.29	64.576
14th	46.473	51.493	57.764	59.82
13th	42.834	47.551	53.209	55.045
12th	39.157	43.556	48.654	50.275
11th	35.475	39.543	44.175	45.565
10th	31.769	35.492	39.68	40.847
9th	28.057	31.419	35.199	36.145
8th	24.363	27.345	30.754	31.487
7th	20.705	23.296	26.358	26.888
6th	17.127	19.316	22.062	22.395
5th	13.65	15.431	17.849	18.008
4th	10.339	11.71	13.772	13.789
3rd	7.275	8.248	9.915	9.829
2nd	4.559	5.165	6.388	6.254
1st	2.303	2.596	3.313	3.202
GR	0.689	0.764	0.998	0.959

Table 42: Story Drift at each floor level for G+30 model

Story/ Diaphragm	Model 5-0 Without Eccentricity		Model 5-1 With Eccentricity	
	Drift X (mm)	Drift Y (mm)	Drift X (mm)	Drift Y (mm)
30th	0.000738	0.000732	0.001075	0.001163
29th	0.000768	0.000767	0.001086	0.001183
28th	0.000804	0.00081	0.001236	0.001298
27th	0.000842	0.000857	0.001382	0.001414
26th	0.000878	0.000901	0.001509	0.00152
25th	0.00091	0.00094	0.001616	0.001622
24th	0.000923	0.000959	0.00147	0.001482
23rd	0.000949	0.00099	0.001529	0.001541
22nd	0.000973	0.001019	0.001583	0.001592
21st	0.000981	0.001032	0.001499	0.001508
20th	0.001004	0.001061	0.001539	0.001546
19th	0.001024	0.001087	0.001578	0.001578
18th	0.001044	0.001112	0.001611	0.001604
17th	0.001058	0.001132	0.001617	0.001605
16th	0.001073	0.001152	0.001638	0.001623
15th	0.001076	0.00116	0.001616	0.001599
14th	0.001085	0.001174	0.001635	0.001615
13th	0.001092	0.001185	0.001648	0.001623
12th	0.001088	0.001185	0.001626	0.001598
11th	0.00109	0.001191	0.001642	0.001609
10th	0.001086	0.001192	0.001651	0.001613
9th	0.001077	0.001186	0.001652	0.001608
8th	0.001061	0.001174	0.001645	0.001596
7th	0.001033	0.001149	0.00162	0.001566
6th	0.001001	0.001118	0.001607	0.001546
5th	0.000951	0.001069	0.001577	0.001509
4th	0.000878	0.000993	0.001518	0.001443
3rd	0.000778	0.000883	0.001411	0.001331
2nd	0.000646	0.000736	0.00125	0.001171
1st	0.000463	0.000526	0.000951	0.000885
GR	0.000198	0.00022	0.000411	0.00038



3.5.6 Case 6: G+40 Building (Model 6)

The effect of eccentricity on the performance of wall frame structures has been investigated in this section using a forty-one story G+40 building model. The effect of eccentricity on the performance of this medium story structures is clearly shown using two forty-one story building models with different wall frame structural layout. The first building model has a structural wall frame arrangement that will produce zero eccentricity and the second building the wall and frames are arranged in such a way to produce high story eccentricity. The loading and material properties of all structural elements used is briefly shown in the previous section.

The buildings laid out on a rectangular grid with a typical 6m width bays in the X and Y directions. For purpose of simplicity the beams, slab and shear walls have constant cross section throughout the building and the columns have been designed and the optimum section is used. The structures are hypothetical and have been chosen for the illustrative purpose. The member cross sections selected for this analysis were;

Table 43: The member cross sections used in G+40 models

Item No.	Structural Element	Section Name		Width (mm)	Height (mm)
		Model 1-0 Without Eccentricity	Model 1-1 With Eccentricity		
1	Column	CL 50X50	CL 50X50	500	500
		CL 60X60	CL 60X60	600	600
		CL 70X70	CL 70X70	700	700
		CL 80X80	CL 80X80	800	800
		CL 90X90	CL 90X90	900	900
		CL 100X100	CL 100X100	1000	1000
		CL 110X110	CL 110X110	1100	1100
		CL 120X120	CL 120X120	1200	1200
		CL 130X130	CL 130X130	1300	1300
		CL 140X140	CL 140X140	1400	1400
2	Beam	BM 40X70		400	700
				Thickness (mm)	
3	Wall	Wall 500mm		500	
4	Slab	Slab 200mm		200	

The structural models have then been modeled, analyzed and designed according to Ethiopian building code and standards EBCS EN:2014. The analyses and design results are compared with each other and briefly discussed to investigate the effect of eccentricity on the performance of building structure.

The cross-section design of the two building models has revealed that;

- The column immediately near the shear wall require a slightly bigger sections at the top story levels in both wall frame structures with eccentricity and without eccentricity.
- The other column, beam, slab and shear wall cross sections remain similar for both cases.

Table 44: Center of mass and Center of rigidity for G+40 model without eccentricity

Model	Story	Mass (kg)	Center of Mass (m)		Center of Rigidity (m)		Eccentricity (m)	
			CMX	CMY	CRX	CRY	ex	ey
Model 6-0	GR	1,021,098.52	12.00	12.00	12.00	12.00	0.00	0.00
	1	1,021,098.52	12.00	12.00	12.00	12.00	0.00	0.00
	2	1,019,942.16	12.00	12.01	12.00	12.00	0.00	0.00
	3	997,423.16	12.01	12.01	12.00	12.00	0.00	0.00
	4	975,752.15	12.01	12.01	12.00	12.00	0.00	0.00
	5	972,642.83	12.01	12.00	12.00	12.00	0.00	0.00
	6	970,552.82	12.00	12.01	12.00	12.00	0.00	0.00
	7	954,843.48	11.99	11.99	12.00	12.00	0.00	0.00
	8	934,354.53	11.99	11.99	12.00	12.00	0.00	0.00
	9	927,108.01	12.01	12.01	12.00	12.00	0.00	0.00
	10	925,274.97	12.00	12.01	12.00	12.00	0.00	0.00
	11	922,456.89	12.01	12.01	12.00	12.00	0.00	0.00
	12	905,385.61	12.00	11.99	12.00	12.00	0.00	0.00
	13	889,025.29	12.00	12.00	12.00	12.00	0.00	0.00
	14	885,770.35	12.01	12.02	12.00	12.00	0.00	0.00
	15	869,204.45	11.99	12.00	12.00	12.00	0.00	0.00
	16	855,388.11	11.98	11.98	12.00	12.00	0.00	0.00
	17	853,760.64	12.00	12.00	12.00	12.00	0.00	0.00
	18	849,083.82	12.02	12.02	12.00	12.00	0.00	0.00
	19	844,749.62	12.01	12.02	12.00	12.00	0.00	0.00
	20	842,582.52	12.00	12.00	12.00	12.00	0.00	0.00
	21	829,528.52	12.01	12.01	12.00	12.00	0.00	0.00
	22	816,611.57	12.01	12.02	12.00	12.00	0.00	0.00
	23	813,382.33	12.01	12.01	12.00	12.01	0.00	0.00

	24	801,107.80	11.99	11.99	12.00	12.01	0.00	0.00
	25	790,443.61	12.00	12.00	12.00	12.00	0.00	0.00
	26	787,437.08	12.01	12.01	12.00	12.01	0.00	0.00
	27	775,642.23	12.00	12.00	12.01	12.01	0.00	0.00
	28	766,014.47	11.99	12.00	12.00	12.01	0.00	0.00
	29	764,464.10	12.00	12.00	12.00	12.01	0.00	0.00
	30	757,106.23	12.00	12.00	12.01	12.01	0.00	0.00
	31	747,615.53	11.99	11.99	12.01	12.01	0.00	0.00
	32	743,572.56	12.00	12.00	12.01	12.00	0.00	0.00
	33	742,835.92	12.01	12.01	12.01	12.01	0.00	0.00
	34	741,491.12	12.01	12.01	12.02	12.01	0.00	0.00
	35	740,420.41	12.00	12.00	12.02	12.01	0.00	0.00
	36	740,420.41	12.00	12.00	12.02	12.01	0.00	0.00
	37	740,420.41	12.00	12.00	12.02	12.01	0.00	0.00
	38	740,420.41	12.00	12.00	12.02	12.01	0.00	0.00
	39	740,420.41	12.00	12.00	12.02	12.01	0.00	0.00
	40	706,950.27	12.00	12.00	12.02	12.01	0.00	0.00
							0.00	0.00

Table 45: Center of mass and Center of rigidity for G+40 model with eccentricity

Model	Story	Mass (kg)	Center of Mass (m)		Center of Rigidity (m)		Eccentricity (m)	
			CMX	CMY	CRX	CRY	ex	ey
Model 6-1	GR	1,011,521.55	12.36	12.38	8.97	9.68	3.39	2.70
	1	1,008,463.62	12.35	12.38	8.00	7.70	4.36	4.67
	2	1,008,463.62	12.35	12.38	7.85	6.80	4.50	5.58
	3	988,034.63	12.37	12.40	7.92	6.45	4.44	5.94
	4	968,453.63	12.37	12.41	8.07	6.40	4.30	6.01
	5	963,065.86	12.35	12.38	8.25	6.48	4.10	5.90
	6	958,140.63	12.33	12.35	8.43	6.65	3.90	5.71
	7	961,055.39	12.47	12.50	8.63	6.88	3.84	5.62
	8	944,491.93	12.49	12.52	8.86	7.16	3.62	5.36
	9	937,265.00	12.47	12.50	9.09	7.45	3.38	5.05
	10	930,888.51	12.44	12.46	9.29	7.72	3.15	4.74
	11	926,619.16	12.42	12.44	9.47	7.97	2.95	4.47
	12	914,978.49	12.44	12.47	9.62	8.19	2.82	4.28
	13	898,138.49	12.45	12.48	9.76	8.40	2.69	4.08
	14	892,078.93	12.43	12.46	9.87	8.58	2.56	3.88
	15	865,512.06	12.33	12.36	9.97	8.74	2.36	3.62
	16	853,383.15	12.34	12.37	10.04	8.87	2.30	3.51

	17	847,250.17	12.29	12.33	10.09	8.97	2.21	3.37
	18	840,963.00	12.27	12.31	10.13	9.06	2.15	3.26
	19	836,731.59	12.27	12.31	10.17	9.14	2.10	3.16
	20	834,427.44	12.27	12.30	10.22	9.23	2.05	3.07
	21	822,067.25	12.27	12.30	10.26	9.31	2.02	2.99
	22	808,910.47	12.27	12.30	10.29	9.38	1.97	2.92
	23	805,158.73	12.26	12.30	10.33	9.46	1.93	2.84
	24	795,316.84	12.28	12.32	10.37	9.53	1.91	2.79
	25	784,224.36	12.27	12.32	10.41	9.60	1.86	2.72
	26	778,905.11	12.25	12.29	10.44	9.67	1.81	2.62
	27	770,553.64	12.28	12.32	10.48	9.73	1.80	2.59
	28	760,694.62	12.28	12.31	10.51	9.79	1.76	2.52
	29	755,760.82	12.24	12.28	10.54	9.84	1.70	2.43
	30	749,328.04	12.26	12.29	10.58	9.89	1.69	2.40
	31	741,833.13	12.26	12.30	10.61	9.94	1.65	2.35
	32	737,027.82	12.23	12.26	10.63	9.98	1.59	2.28
	33	734,243.99	12.23	12.25	10.65	10.01	1.58	2.24
	34	730,817.75	12.22	12.25	10.68	10.05	1.54	2.20
	35	728,744.87	12.21	12.23	10.69	10.07	1.52	2.17
	36	728,744.87	12.21	12.23	10.71	10.11	1.50	2.13
	37	728,744.87	12.21	12.23	10.75	10.17	1.46	2.07
	38	728,744.87	12.21	12.23	10.80	10.25	1.41	1.99
	39	728,744.87	12.21	12.23	10.88	10.34	1.33	1.89
	40	700,446.21	12.23	12.26	10.98	10.44	1.25	1.82

The location of center of mass and center of rigidity of the buildings used to investigate the performance of structure subjected to high eccentricity in this section of case study is shown in the tables above. The eccentricity of both buildings has been quantified and clearly compared in the table below to show that the first building has no eccentricity while the second one is with high eccentricity.

Table 46: Eccentricity at each floor level for G+40 model

Story	Model 6-0 Without Eccentricity		Model 6-1 With Eccentricity	
	Eccentricity (m)			
	ex	ey	ex	ey
GR	0.00	0.00	3.39	2.70
1	0.00	0.00	4.36	4.67
2	0.00	0.00	4.50	5.58
3	0.00	0.00	4.44	5.94

4	0.00	0.00	4.30	6.01
5	0.00	0.00	4.10	5.90
6	0.00	0.00	3.90	5.71
7	0.00	0.00	3.84	5.62
8	0.00	0.00	3.62	5.36
9	0.00	0.00	3.38	5.05
10	0.00	0.00	3.15	4.74
11	0.00	0.00	2.95	4.47
12	0.00	0.00	2.82	4.28
13	0.00	0.00	2.69	4.08
14	0.00	0.00	2.56	3.88
15	0.00	0.00	2.36	3.62
16	0.00	0.00	2.30	3.51
17	0.00	0.00	2.21	3.37
18	0.00	0.00	2.15	3.26
19	0.00	0.00	2.10	3.16
20	0.00	0.00	2.05	3.07
21	0.00	0.00	2.02	2.99
22	0.00	0.00	1.97	2.92
23	0.00	0.00	1.93	2.84
24	0.00	0.00	1.91	2.79
25	0.00	0.00	1.86	2.72
26	0.00	0.00	1.81	2.62
27	0.00	0.00	1.80	2.59
28	0.00	0.00	1.76	2.52
29	0.00	0.00	1.70	2.43
30	0.00	0.00	1.69	2.40
31	0.00	0.00	1.65	2.35
32	0.00	0.00	1.59	2.28
33	0.00	0.00	1.58	2.24
34	0.00	0.00	1.54	2.20
35	0.00	0.00	1.52	2.17
36	0.00	0.00	1.50	2.13
37	0.00	0.00	1.46	2.07
38	0.00	0.00	1.41	1.99
39	0.00	0.00	1.33	1.89
40	0.00	0.00	1.25	1.82

Table 47: Period and Frequencies at each floor level for G+40 model

Mode	Model 6-0 Without Eccentricity		Model 6-1 With Eccentricity	
	Period (sec)	Frequency (cyc/sec)	Period (sec)	Frequency (cyc/sec)
1	4.542	0.220	4.56	0.22
2	4.399	0.227	4.39	0.23
3	2.975	0.336	3.20	0.31
4	1.534	0.652	1.61	0.62
5	1.468	0.681	1.50	0.67
6	1.094	0.914	1.10	0.91
7	0.809	1.236	0.92	1.09
8	0.762	1.313	0.81	1.23
9	0.656	1.524	0.63	1.59
10	0.519	1.927	0.57	1.77
11	0.482	2.075	0.53	1.91
12	0.462	2.166	0.48	2.11

The lowest natural frequency (f) of vibration of a structure corresponds to the longest time period (T) of vibration, as frequency and time period are inversely proportional ($T=1/f$). This is also referred to as the first mode of vibration or fundamental period of vibration. Structure will have multiple natural modes of vibration for which frequency will be higher and time period will be shorter than the fundamental period as can be observed from the table above. The analysis result on the G+40 model has shown that the fundamental period of the wall-frame structure without eccentricity (4.54sec) is lower than the fundamental period of the wall frame structure (4.56sec) with eccentricity.

The modal diaphragm center of mass displacements has been investigated and compared from the analysis result of both modes and the story drift of each mode is calculated in the tables below.

Table 48: Story Diaphragm Displacements at each floor level for G+40 model

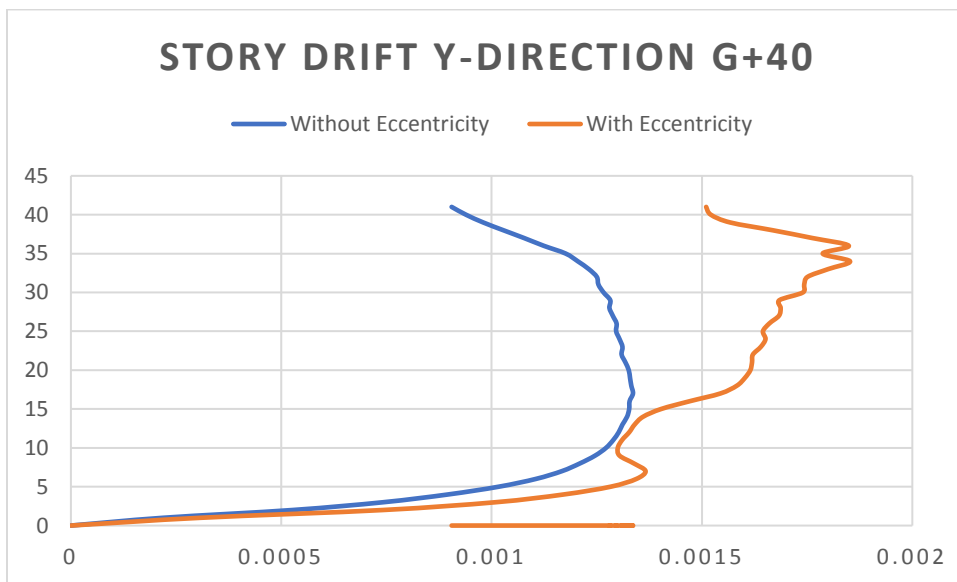
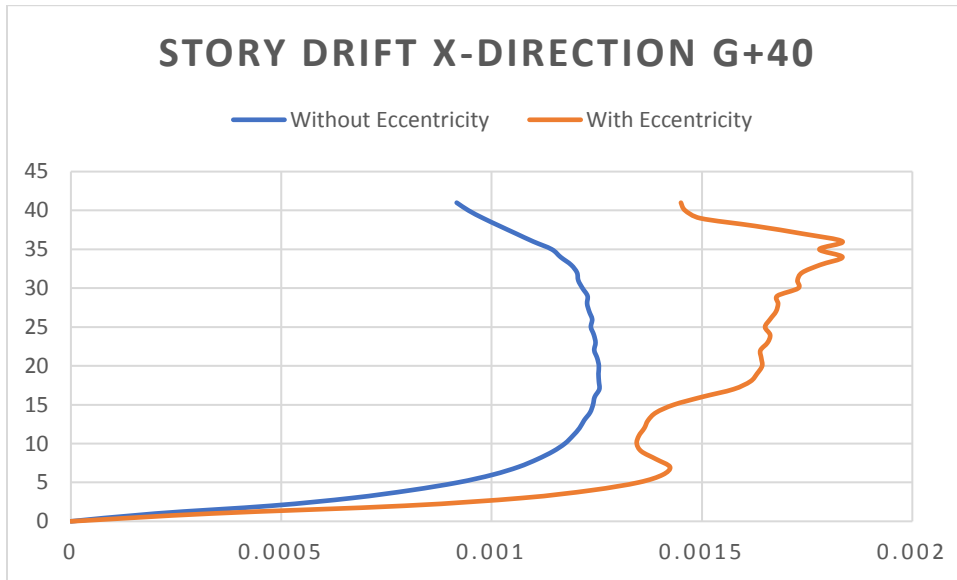
Story/ Diaphragm	Model 6-0 Without Eccentricity		Model 6-1 With Eccentricity	
	UX (mm)	UY (mm)	UX (mm)	UY (mm)
40th	152.234	160.946	187.845	193.263
39th	149.212	157.964	183.339	188.403
38th	146.114	154.888	178.593	183.371

37th	142.918	151.692	173.631	178.176
36th	139.605	148.352	168.467	172.82
35th	136.167	144.859	163.119	167.316
34th	132.604	141.212	157.617	161.679
33rd	128.93	137.429	152.211	156.116
32nd	125.148	133.517	146.703	150.45
31st	121.277	129.498	141.303	144.878
30th	117.344	125.403	135.926	139.357
29th	113.386	121.279	130.56	133.845
28th	109.374	117.093	125.251	128.381
27th	105.314	112.849	120.01	122.995
26th	101.254	108.598	114.729	117.593
25th	97.16	104.31	109.545	112.249
24th	93.029	99.977	104.365	106.948
23rd	88.896	95.639	99.202	101.673
22nd	84.733	91.259	94.056	96.41
21st	80.543	86.847	88.936	91.18
20th	76.349	82.424	83.873	85.996
19th	72.127	77.963	78.825	80.833
18th	67.886	73.472	73.791	75.693
17th	63.635	68.961	68.824	70.613
16th	59.374	64.43	63.933	65.609
15th	55.101	59.877	59.097	60.678
14th	50.857	55.344	54.555	55.982
13th	46.614	50.803	50.117	51.409
12th	42.381	46.265	45.782	46.944
11th	38.186	41.755	41.503	42.541
10th	34.016	37.261	37.308	38.207
9th	29.886	32.798	33.186	33.938
8th	25.814	28.383	29.104	29.707
7th	21.829	24.048	25.031	25.497
6th	17.966	19.829	20.885	21.24
5th	14.251	15.756	16.826	17.05
4th	10.745	11.895	12.896	13.004
3rd	7.53	8.34	9.186	9.205
2nd	4.705	5.205	5.839	5.801
1st	2.369	2.61	2.979	2.933
GR	0.703	0.765	0.879	0.864

Table 49: Story Drift at each floor level for G+40 model

Story/ Diaphragm	Model 6-0 Without Eccentricity		Model 6-1 With Eccentricity	
	Drift X (mm)	Drift Y (mm)	Drift X (mm)	Drift Y (mm)
40th	0.000917	0.000905	0.00145	0.00151
39th	0.000945	0.000939	0.00146	0.001521
38th	0.00098	0.00098	0.001496	0.001567
37th	0.001019	0.001028	0.001626	0.001668
36th	0.001059	0.001077	0.00174	0.001762
35th	0.001099	0.001124	0.001834	0.001849
34th	0.001143	0.001176	0.001779	0.001787
33rd	0.001164	0.001205	0.001834	0.001852
32nd	0.001189	0.001231	0.001781	0.0018
31st	0.001203	0.00125	0.001738	0.001751
30th	0.001206	0.001254	0.001727	0.001743
29th	0.001216	0.001266	0.001729	0.00174
28th	0.001228	0.001282	0.001679	0.001684
27th	0.001227	0.00128	0.001681	0.001687
26th	0.001232	0.001288	0.001676	0.001683
25th	0.001239	0.001297	0.001662	0.00166
24th	0.001236	0.001296	0.00165	0.001645
23rd	0.001243	0.001304	0.001662	0.001651
22nd	0.001247	0.001311	0.001656	0.00164
21st	0.001244	0.001309	0.001639	0.001621
20th	0.001251	0.001318	0.001641	0.001619
19th	0.001255	0.001326	0.001643	0.001615
18th	0.001254	0.001329	0.001631	0.001602
17th	0.001255	0.001332	0.001615	0.001583
16th	0.001256	0.001336	0.001575	0.001544
15th	0.001245	0.001328	0.0015	0.001471
14th	0.001241	0.001327	0.001433	0.001403
13th	0.001234	0.001322	0.001391	0.00136
12th	0.00122	0.001311	0.001372	0.00134
11th	0.001209	0.001302	0.001363	0.001327
10th	0.001193	0.001289	0.00135	0.00131
9th	0.001174	0.001272	0.001345	0.0013
8th	0.001146	0.001246	0.001356	0.001304
7th	0.001109	0.00121	0.001391	0.001338

6th	0.001064	0.001167	0.001424	0.001366
5th	0.001003	0.001105	0.001407	0.001344
4th	0.00092	0.001017	0.001349	0.001283
3rd	0.000808	0.000896	0.001238	0.001171
2nd	0.000668	0.000742	0.001072	0.001009
1st	0.000476	0.000527	0.000793	0.000742
GR	0.000201	0.000219	0.000332	0.000308



Conclusion and Recommendation

The performance of building with different story levels and shear wall locations has been investigated using ETABS finite element analysis software. This wall frame structural models have been configured in such a way to represent building structures with and without eccentricities. The analysis and design output results has been briefly interpreted in order to draw the following conclusions and recommendations.

- The eccentricity of the building mainly affects the period, diaphragm center of mass displacement. These differences finally lead to major differences in inter-story drift value.
- The fundamental period of the models increases as the eccentricity of the buildings increases.
- The displacement and inter story drift of the structures increase with the increase in eccentricities of the buildings
- The fundamental period and story drift vary significantly with the increase in story level for both non eccentric and eccentric wall frame structural systems.
- The columns near the location of shear wall are exposed to more loads and requires larger cross sections in both non eccentric and eccentric structural wall frame systems.
- The columns on opposite side of the shear wall in wall frame systems subjected to high eccentricity exhibit larger torsional forces and requires larger sections.

The following recommendations are drawn observing the results of the study that has been carried out;

- The distance between center of mass and center of rigidity of individual stories in a building shall be kept to a minimum more preferably zero to avoid large torsional forces in the structure and also minimize the column cross sections.
- The layout of shear walls in wall frame structures during designing stage shall have to be proportional and symmetric in order to avoid high story eccentricities on the structures.
- Provision of supplemental strength in form of additional lateral force-resisting elements such as shear walls, braced or moment frames helps to reduce the lateral displacement by increasing the stiffness of the buildings.
- Reinforced concrete shear walls can be provided in the opposite side of an existing shear wall of a building configuration in order to significantly reduces lateral sway of the building and thereby reduces damage to structural and nonstructural components. Since reinforced concrete shear walls also carry large horizontal earthquake forces, the overturning effects on them are large
- Increasing the stiffness of the building reduces its period and decreases its displacement. An important thing to recall is that it is not possible to increase stiffness without increasing strength. This method might cause some space rearrangement of the building.
- The loads are attracted to the stronger and stiffened part of the structural model hence, the columns around the shear wall must have sufficient strength to withstand the loads applied on them

References

- [1]. Mohamed Arega, “How high can buildings made of ribbed or flat slab construction without shear walls be built”, Thesis in partial fulfilment of degree for master of science, Addis Ababa University, December 2006
- [2]. Marie Jose Nollet, “Behavior of wall frame structures”, McGill University, February 1991
- [3]. Mihaela Anechitel, Doina Stefan, Ioana Olteanu, “The torsional eccentricity of the structure in miscellaneous design codes”, Universitatea Technica, 2010
- [4]. A.J. Zielinski, “Concrete structures under impact loading Rate effects”, The Netherlands: Delft University of technology, 1984
- [5]. Dr-ing. A.J. Zielinski, “Concrete structures under impact loading rate effects”, Netherlands; Delft University of Technology, Stevin Laboratory, 1984
- [6]. Abdi Mohamed, “Comparison of conventional and modal pushover analysis of buildings”, Thesis in partial fulfilment of degree for master of science, Addis Ababa institute of Technology, March 2012
- [7]. Anil K. Chopra, “Dynamics of Structures – Theory and Application to Earthquake Engineering”, Printice-Hall, 1995
- [8]. Mihaela Anechitel, Doina Stefan, Ioana Olteanu, “The torsional eccentricity of the structure in miscellaneous design codes”, Universitatea Technica,, Gheorghe Asachi”, 2010
- [9]. A.J. Zielinski, “Concrete structures under impact loading Rate effects”, The Netherlands: Delft University of Technology, 1984
- [10]. Ministry of urban development and construction, EBCS-0 Basis of structural design, EBCS EN 1990:2014: December 2014
- [11]. Ministry of urban development and construction, EBCS-1 Part 1-1 Actions on structures, EBCS EN 1991-1-1:2014: December 2014
- [12]. Ministry of urban development and construction, EBCS-2 Part 1-1 Design of concrete structures, EBCS EN 1992-1-1:2014: December 2014

- [13]. Ministry of urban development and construction, EBCS-8 Part 1
Design of structures for earth quake resistance, EBCS EN 1998-1:2014:
December 2014
- [14]. S. Zheng, U. Haussler-Combe and J. Eibl, “New approach to Strain
Rate Sensitivity of Concrete in Compression”, 2010
- [15]. Malgorzata Pajak, “The influence of the strain rate on the strength
of concrete taking into account the experimental”, 2011