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Effect of Masonry Infill Materials on Dynamic Characteristics of Buildings

A Thesis in Structural Engineering

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A Thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science

UNDERTAKING

I certify that research work titled “Effect of Masonry infill Materials on Dynamic Characteristics of Buildings” is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

Desalegn Bogale

APPROVAL

The undersigned have examined the thesis entitled ‘**Effect of Masonry Infill Materials on Dynamic Characteristics of Buildings**’ presented by **Desalegn Bogale**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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ABSTRACT

Modern codes for earthquake resistant building design require consideration of the so-called accidental design eccentricity, to account for torsional response caused by several factors not explicitly considered in design. This provision requires that the mass centers in the building floor be moved a certain percentage of the building's dimension (usually 5%) along both the x and y axes and in both positive and negative directions.

This thesis addresses the influence of infill wall on the response of building structures subjected to earthquake ground motions. Furthermore, the increase an equivalent additional eccentricity due to the action of infill wall is estimated. For this purpose, G+10 Reinforced Concrete (RC) models with infill wall type HCB A are used in the study. To study the effect of infill materials on accidental eccentricity, Response spectrum analysis performed using ETABS 2017 on a set of three-dimensional reinforced concrete multi-story buildings with irregular arrangement of infill wall. The obtained results shows The estimated equivalent additional eccentricity of the studied cases exceeded the value of 5% of the dimension of the floor (D) perpendicular to the excitation axis which is recommended by several seismic code as an accidental eccentricity.

Key words:

Infill Materials, Response Spectrum Analysis, accidental eccentricity.

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CHAPTER ONE

1. INTRODUCTION

1.1. General

This study builds on several analytical efforts to evaluate the effect of different infill masonry material arrangement on the eccentricity of Buildings. Under strong earthquake motions, non-symmetric buildings experience translational and torsional motion. The latter is caused mainly by non-symmetric distribution of element stiffness, strength and/or the building masses and since these properties are included in the building model used for the analysis, they are directly accounted for in design. However, there are also several other factors that are impossible or very difficult to quantify and hence quite difficult to be directly accounted in design. As such factors may list: non uniform ground motion and consequent excitation differences at the support points, the presence of non-structural, yet stiffness and strength possessing elements not accounted for in design, unknown non-symmetric distributions of live loads or differences between actual and design distributions of mass, stiffness and strength. The torsional effects of such factors, which can also be present in fully symmetric buildings, have been idealized in modern codes by an extra mass eccentricity termed “accidental design eccentricity” (ADE). Eurocode 8 specify this eccentricity equal to $0.05L$, where L is the maximum dimension of the floor layout in the considered direction. Eurocode 8 specify that four additional loading conditions should be considered in design, by displacing the mass centers of all floors by amounts equal to $\pm ADE$ along each of the considered building directions x and y .

Masonry is one of the oldest construction materials currently in use around the world for reasons that include accessibility, functionality, and cost. This material has been used for hundreds of years in construction projects ranging from simple roadways to complex arch designs. Masonry has also commonly been used in frame building structures as infill, where it was intended to act as an environmental divider rather than a structural element. The primary function of masonry was either to protect the inside of the structure from the environment (rain, snow, wind, etc.) or to divide inside spaces. In either case, the common practice has always been to ignore infill during the design and analysis of steel/reinforced concrete frame structures. However, infill walls tend to interact with the surrounding frame when the structure is subjected to wind or earthquake loads; the resulting system is referred to as an infilled frame.

1.2. Statement of the Problem

The present practice of structural analysis in Ethiopia is to treat the masonry infill as non-structural element and the analysis as well as design of buildings is carried out by using only the mass but neglecting the strength and stiffness contribution of infill.

Masonry infill has significant effect on seismic behavior of building. Therefore, effects of different infilled materials, on the dynamic characteristic of Building needs to be studied. The common practice which does not consider such effects may lead to misguided analysis and design results, hence, needs to be carefully considered and corrected.

1.3. Objective

1.3.1. General Objective

The general objective of this study is to examine the effect of infill masonry materials on the provision of accidental eccentricity of bare frame.

1.3.2. Specific Objective

- To study the relative effects of irregular arrangement of infill wall material on the provision of accidental eccentricity of Buildings.
- To compare the eccentricity of sample RC buildings with provision of accidental eccentricity.

1.4. Significance of Study

This thesis presents how masonry infill materials affect the eccentricity of Concrete buildings. This thesis used sample G+10 buildings to examine such effect. The thesis will help for Engineers to observe and estimate the effects of different infill materials and help them to pick the preferred option for infill walls.

Torsion in buildings during earthquake is caused due to various reasons and the most common is unsymmetrical distribution of mass and stiffness along the height of building. Recent codes have given provision to counter these torsional effects by introducing accidental eccentricity which has to be considered at the time of analysis and design. A special care and attention must be taken, so that, torsional effects do not alter the ductile behaviour of the structure.

Most of the structures are subjected to vibrations with ground motion. To protect structures from significant damage and torsional effect in structures under severe earthquakes has become an important issue in structural engineering. So it is important to control torsion or twisting in multi storey building structures.

1.5. Scope of the Study

The procedures presented in this document are applicable to all building structures constructed using reinforced concrete frame structures with unreinforced masonry infill walls. The sample buildings considered for this study are a G+10 office buildings with reinforced concrete frames. The floor heights are kept constant throughout the buildings and

the floor layouts are set to satisfy plan and elevation regularity requirements. Infill thickness of 20cm for external wall and 15cm for internal wall in cement sand mortar ratio 1:3 are used.

This study only deals with the in-plane stiffness and strength of masonry infills. Macro models of infill panels are considered for the simplified analysis.

The Masonry infilled frame is analyzed using ETABS 2017 software. The present study is concerned only with the macro models of infill panels and the equivalent struts were estimated using equations derived by Paulay and Priestley, 1992 for their average predictions of infilled frame in-plane behavior when compared with experimental results.

The effect of infill wall material on provision of accidental eccentricity of reinforced concrete buildings is studied using linear dynamic response spectrum analysis. The influence of infill walls is examined by analyzing both bare and infilled frames with different arrangement of infill wall.

1.6. Infilled Materials

Infill materials used in Ethiopia are listed below:

1.6.1. Hollow Concrete Blocks

Hollow concrete blocks are classified in to the following three classes.

Class A and B are load bearing units and suitable for

- External walls pointed, rendered and plastered
- The inner leaf of cavity walls or stonemasonry
- internal walls or partitions
- Panels in steel framed and reinforced steel framed buildings.

Class C - non-load bearing units suitable for

- Non-load bearing walls and partitions
- Non-load bearing internal panels in steel framed and reinforced concrete buildings.

The nominal dimensions and detail properties of Hollow Concrete Blocks are indicated in section 2.5.1.

1.6.2. Hollow Clay Bricks

Hollow clay bricks are bricks with parallel holes passing through it and having an area not less than 25% of the total area. The nominal dimensions and detail properties of hollow clay bricks are indicated in section 2.5.2 of this documentation.

1.7. Methodology Adopted

The objective of the research is aimed to be achieved in accordance with the method outlined below.

1.7.1. Literature Review

This thesis bases its methodologies and detail analysis on previous documentations and literatures. Chapter 2 mainly deals with the history of development of masonry infill models and their failure modes and the effect of seismic load on the masonry infilled reinforced concrete structures.

Journals and articles on the effect of masonry or concrete infill on steel or reinforced concrete moment resisting frame were reviewed while and ahead of this thesis. In addition; books and relevant design codes were also studied. The purpose of literature review was to gain firsthand knowledge on the methods of studies adopted, which could be used as a guideline for this study. The review of past studies also provided some ideas on modeling techniques and parameters to be used for different materials like reinforced concrete, hollow concrete and brick masonry.

1.7.2. Analysis of Infill Walls

Over the past few decades several methods for the analysis of infilled frames have been proposed in different literatures by various investigators. These methods can be divided into two groups, depending on the degree of refinement used to represent the structure. The first group consists of the macro models to which belong the simplified models that are based on a physical understanding of the structure. The second group involves the micro models including the finite element formulations, considering local effects in detail. Of these two types, the Macro models are the ones covered by this study.

In the case of an infill wall located in a lateral load resisting frame, the stiffness and strength contribution of the infill are considered by modeling the infill as an equivalent compression strut. Because of its simplicity, several investigators have recommended the equivalent strut

concept. In the present analysis, a trussed frame model is considered. This type of model does not neglect the bending moment in beams and columns. Rigid joints connect the beams and columns, but pin joints connect the eccentric equivalent struts to columns.

1.7.3. Analysis of Building

The sample structures studied are G+10 buildings. The analysis for each building has been carried out according to the EN 1998-1:2004 ductility class medium (DCM) with design ground acceleration of 0.2g. The structures have been assessed both as a bare frame and as an infilled one. The beams and columns are modeled as a frame element which has the capability to deform axially, in shear, in bending and in torsion. The beam-column joints are assumed to be rigid. The weight of the slab is distributed as rectangular load to the surrounding beams as per EBCS EN 1992-1-1:2014. A rigid joint diaphragm action was assumed, which ensures integral action of all the vertical and lateral load-resisting elements. In this paper different models of building with different infill masonry material arrangement are generated with the help of ETABS 2017 and Response Spectrum Method of analysis has been carried out.

1.8. Organization of the Thesis

This thesis comprises the following five chapters which clearly present its organization:

Chapter 1: Gives general introduction of the thesis work and what it encompasses.

Chapter 2: Discusses the literature review citing works done by various researchers in the field, specifically on the effect of infill on seismic behavior of buildings and modeling of structural members and infill walls.

Chapter 3: Deals with the details of the structure modeled in ETABS 2016.

Chapter 4: The results from the analysis, comparison between the RC frames with and without the masonry walls all are discussed in this chapter.

Chapter 5: conclusions and recommendations of the present study are given in this chapter followed by the references and appendix.

CHAPTER TWO

2. LITERATURE REVIEW

2.1. General

The infill frames have drawn attention of several investigators in the recent for their inherent structural advantage. This chapter provides an overview of experimental and analytical researches in infilled frames during the last half century. The focus is kept on steel and reinforced concrete infilled frames. Experimental investigations have been conducted by several researchers using a wide range of testing scales, numbers of specimens, infill materials, experimental setups and constraint studies. Several collapse and damage patterns have been observed. Experimental researches have been complimented by the analytical attempts to model infilled frame behavior. Global and fundamental models have been formulated. The infilled frame structure is however still difficult to model, partly, due to a nonlinear phenomenon associated with infill and with frame-to-infill contact areas. There are no universally accepted design guidelines for infilled frames.

2.2. Influence of Masonry Infill on Seismic Behavior of Frames

It is a common misconception that masonry infill in structural steel or reinforced concrete frames can only increase the overall lateral load capacity, and therefore must always be beneficial to seismic performance. In fact, there are numerous examples of earthquake damage, some of which are can be traced to structural modification of the basic frame by so called non-structural masonry partitions and infill panels. Even if they are relatively weak, masonry infill can drastically alter the intended structural response, attracting forces to parts of the structure that have not been designed to resist them (Pauley & Priestley, 1992).

The presence of masonry infill affects the seismic behavior of buildings in the following ways (Penelis and Kappos, 1997).

- As a consequence of increase in stiffness of buildings, the fundamental period is decreased and the base shear is increased.
- The lateral stiffness in plan and elevation is modified.
- The structural system is relieved of seismic action as part of the load is carried by the infills.
- Energy dissipation capacity of the building is substantially increased.

- The more flexible the structural system, the greater the above effects of the infills.

2.3. Strength and Stiffness of Masonry Infill

2.3.1. In-Plane Strength

The potential modes of failure of the infill as result of its interaction with the frame are as follows; (Perumal Pillial, 1994).

2.3.1.1. Interface Cracking

Interface cracking (IC) is an inherent characteristic of infilled frames and always occur in the early stages. This mode is caused by the distortion in the geometry of the frame in which one diagonal shortens and the other elongates.

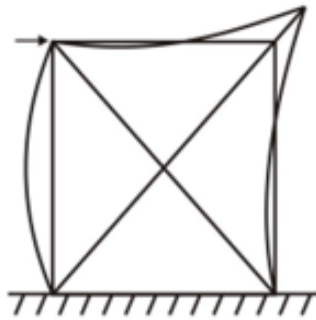


Figure 2-1: Interface Cracking Mode (Perumal Pillial, 1994)

2.3.1.2. Corner Crushing

Corner crushing mode (CC) represents crushing of the infill in at least one of its loaded corners. It is highly influenced by the relative stiffness of the frame and the infill.

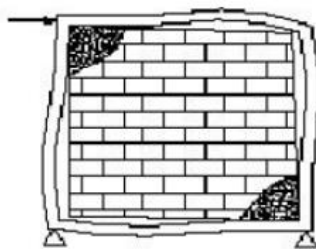


Figure 2-2: Corner Crushing Mode (Perumal Pillial, 1994)

2.3.1.3. Diagonal Cracking

Diagonal cracking represents the diagonal cracking of the wall through the masonry along a line, or lines, parallel to the loading diagonal, and caused by tensile stresses perpendicular to the loading diagonal.

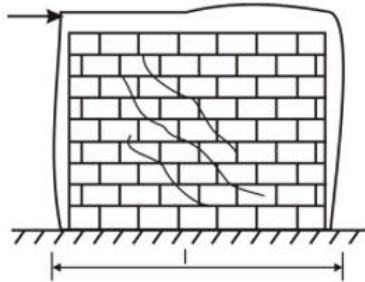


Figure 2-3: Diagonal Cracking Mode (Perumal Pillial, 1994)

2.3.1.4. Shear Slide along Bed Joints

Shear sliding (SS) mode represents horizontal sliding shear failure through bed joints of a masonry infill as shown in Figure 2-4. This mode is associated with infill of weak mortar joints and frame with strong members and joints.

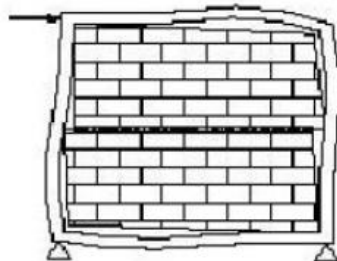


Figure 2-4: Shear Sliding Mode (Perumal Pillial, 1994)

It is worth mentioning that only the first two modes, the CC and the IC modes, are of practical importance since the third mode is very rare to occur and requires a high slenderness ratio of the infill to result in out of plane buckling of the infill under in-plane loading. This is hardly the case when practical panel dimensions are used, and the panel thickness is designed to satisfy the acoustic isolation and fire protection requirements. The fourth mode should not be considered a failure mode, since the wall still carries more loads after it cracks.

In many cases the failure may be sequential combination of some of the failure modes above. For infilled frames the strength associated with the various possible failure modes should be evaluated and the lowest value used as the basic for design.

2.3.2. In-Plane Stiffness

At low levels of in-plane lateral force, the frame and infill wall will act in fully composite fashion, as structural wall with boundary elements. As lateral deformations increase, the behavior becomes more complex because of the frame attempting to deform in a flexural mode while the panel attempts to deform in a shear mode, as shown in Figure 2-5, the result is separation between frame and panel at the corners on the tension diagonal, and the development of a diagonal compression strut on the compression diagonal. Contact between frame and panel occurs for a length z , shown in Figure 2-5. The separation may occur at 50 to 70% of the ideal lateral shear capacity of the infill for reinforced concrete frames. After separation, the effective width of the diagonal strut, w , shown in Figure 2-5, is less than that of the full panel.

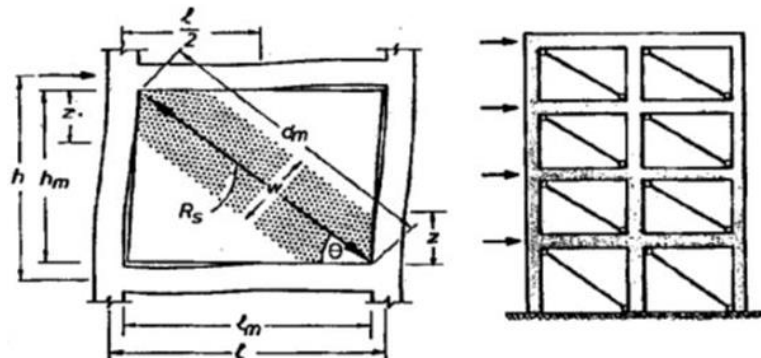


Figure 2-5: Equivalent Bracing Action of Masonry Infill [Paulay & Priestley, 1992]

Natural-period calculations should be based on the structural stiffness after separation occurs. This may be found by considering the structure as an equivalent diagonally braced frame, where the diagonal compression strut is connected by pins to the frame corners. Figure 2-5 shows the equivalent system for a two-bay, four-story frame (Pauley & Priestley, 1992).

2.4. Infill Materials

Different infill materials are used to study the effect of infill. The presence of infill affects the distribution of lateral load in frames of buildings. The study of interaction of infill with frames has been attempted by using Equivalent diagonal strut method. For this study, HCB class A, used, and classification is taken from to Ethiopian building material classification criteria.

2.4.1. Hollow Concrete Block (HCB)

Hollow concrete blocks are classified into the following three classes

Class A and B are load bearing units and suitable for

- External walls pointed, rendered and plastered
- The inner leaf of cavity walls or stonemasonry
- internal walls or partitions
- Panels in steel framed and reinforced steel framed buildings.

Class C non- load bearing units suitable for

- Non-load bearing walls and partitions
- Non-load bearing internal panels in steel framed and reinforced concrete buildings.

Dimensions

The nominal dimensions of Hollow Concrete Blocks are indicated bellow

Table 2-1: Nominal Dimensions of Hollow Concrete Blocks

Breadth (b) (mm)	Height (h) (mm)	Length (L) (mm)	Maximum unit weight (Kg/m ³)
100	200	400	1200
150	200	400	1200
200	200	400	1200
250	200	400	1200
300	200	400	1200

Minimum Compressive Strength

The minimum compressive strength for HCB according to Ethiopian material specification is indicated below

Table 2-2: Minimum Compressive Strength for HCB

Class	Average of six units		Individual units	
	Mpa	Kg/cm ²	Mpa	Kg/cm ²
A	4.2	4.2	3.8	3.8
B	3.5	3.5	3.2	3.2
C	2.0	2.0	1.8	1.8

2.4.2. Hollow Clay Bricks

Hollow clay bricks are bricks with parallel holes passing through it and having an area not less than 25% of total area. The nominal dimensions of hollow clay bricks are indicated below.

Table 2-3: Nominal Dimensions of Hollow Clay Bricks

Nominal dimension(mm)		
Height (h)(mm)	Breadth (mm)	Length (mm)
140	250	250
140	400	250
160	250	250
160	400	250

Compressive Strength

The minimum value of compressive strength of hollow clay bricks shall be as indicated below.

Table 2-4: Minimum Compressive Strength of Hollow Clay Bricks

Type	Average of 5 bricks		Individual bricks	
	Mpa	Kg/cm ²	Mpa	Kg/cm ²
KK,SS,SK	7	70	5.5	55

Note: - KK: With two faces fixed for plastering

SS: With two faces smooth and

SK: With one face smooth and the other face fixed for plastering

2.5. Modeling of Masonry Infilled Frames

Analytic modeling of masonry infill frames comprises different parameters as infill bricks, mortars and friction surfaces between frame elements and infill wall etc. There are two different main approaches for the modeling of infill walls which are Micro Modeling and Macro Modeling.

The main difference between two methods is the precision that micro modeling deals with all individual components brick, block unit and mortar are modeled in detail in micro models while macro modeling consider them as a composite unit (Lourenço, 2002).

Macro models are used to investigate the overall response of the infill wall. The behavior of macro models is based on physical behavior of infill walls. Mortar joints and units are recognized together considering collective mechanical and physical properties to obtain more simplified solution especially for large scaled models (Chen, Moon, & Yi, 2008).

Micro-modeling is used generally to understand the local behavior of masonry infill. Inelastic properties of both unit and mortar and some mechanical properties as Young's

modulus, Poisson's ratio is considered in detailed micro-modeling. On the other hand, each joint on infill wall consists of mortar and the two interface surfaces for simplified micro-modeling method. Infill consisting of elastic blocks interconnected with fracture tracks at the joints (Lourenço, 2002).

Currently Macro modeling applies for design and analysis of infill walls due to computational simplicity. The following section presents a brief review about Equivalent Strut Model which is the most widely used approach on infill design and calculations.

2.6. Previous works

Many research investigations have been carried out regarding the torsional effect of the multi-storey structures. Vipin Gupta and Dr. P.S. Pajgade (2015) presented a review about the investigation done on torsional behaviour of multistory buildings with plan as well as vertical irregularities. It also focuses on codal provision made for torsion. From their investigation on reviews they concluded that the torsion is the most critical factor leading to major damage or complete collapse of building; therefore, it is necessary that symmetric buildings should also be analyzed for torsion. As result the buildings should be designed by considering the design eccentricity & accidental eccentricity. It was observed that the irregular profile buildings got larger forces and displacement as compared to regular one. Structures are never perfectly regular and hence the designers routinely need to evaluate the likely degree of irregularity and the effect of this irregularity on a structure during an earthquake.

Amin Alavi et al., (2013) made an attempt to realize the seismic response of the structures, for various location of shear walls on RC building having re-entrant corners on high seismic zones. They studied a five storey building with six different shear wall locations. They considered the accidental torsion of both negative and positive X and Y directions. The results proved that the structures are more vulnerable when they are more irregular, and also the eccentricities between center of mass and center of resistance are more significant to the torsional behaviour of structures during an earthquake.

Gunay ozmen (2004) explained the conditions which cause torsional irregularity coefficient to exceed the upper bound value of 2 are investigated. A series of eight walled and framed sample structures with different structural shear wall configurations was chosen and their behaviour under earthquake loading were considered. It was found that torsional

irregularity coefficient was maximum when the number of axes and number of stories are low. Also when structural walls are placed as close as possible to the gravity centres without coinciding them, the coefficient were found to be maximum.

R.Riddell and J.Vasquez (1984) discussed the existence of centre of resistance as origins of eccentricity are restricted to a particular class of structures, and concluded that for a general multi-storey building such concepts are physically meaningless. They created a pseudo model and torsional uncoupling had been carried out. They found that the definition of centres of resistance as origins for measuring eccentricity must be associated to the possibility of torsion free dynamic response. Also, concluded that the centres of resistance must be such that if the eccentricity is zero in all stories then the modes of vibration of the building uncoupled into purely torsional modes and purely translational modes. When centres of resistance do exist, they all lie in a vertical line and torsion free vibration is achieved when all storey centres of mass lie in that same line.

A detailed literature review of accidental design eccentricity and its causes may be found in the review paper on torsion by Anagnostopoulos et al. (2013, 2015). In all these publications the problem of accidental eccentricity is studied under several simplifying assumptions for the building model and in most of them by addressing one source of it.

Oman Sayyed (2017) focused his study on the effect of infill and mass irregularity on different floor in RC buildings. The results were concluded that the brick infill enhances the seismic performance of the RC buildings and poor seismic responses were shown by the mass irregular building, therefore it should be avoided in the seismic vulnerable regions.

Himanshu Bansal (2014) analysed vertical irregular building with Response spectrum analysis and Time history Analysis. Irregularities considered are mass irregularity, stiffness irregularity and vertical geometry irregularity. The storey shear force was found maximum for the first storey and it decreases to minimum in the top storey in all cases.

Numerous analytical investigations about infill walls in steel or reinforced concrete frames have been achieved over the few decades. The initial studies about the response of the composite infilled frames were conducted by Polyakov, Holmes, Smith and Carter, Klingner and Bertero with experimental and analytical studies to understand linear behavior and complex disposition of infilled frames. Later, very wide experimental and analytical researches have been conducted in literature.

Polyakov (1956), one of the leading people in this regard, observed that the stress transmission between infills and frame elements only occurs in the compression zone based on elastic theory. From the extensive experiments on model infilled frames with different infills, he studied the Nature and cause of cracks formation, effect of opening and effect of strengthening masonry by RC element. In all these tests, he found that the initial failure was by cracking around the perimeter allowing the separation of frame and infill except at the loaded corners. From the result of Polyakov proposed infill as a diagonal bracing strut.

Holmes (1961) proposed a method of determining the stiffness and strength of infilled frames, where it was suggested that the infill be replaced by an equivalent strut of width equal to one third of the diagonal length and thickness equal to that of infill. Steel frames were modeled, with brick and concrete infill and compared the theoretical values of ultimate loads with the experimental ones by equivalent strut approach. It was found that the theoretical values were differed by 2 to 8% with brick infill and 5% to 8% with concrete infill from the experimental ones.

For diagonally loaded rectangular panel, Smith (1962) had tried two methods for finding stresses. The first one was based on the strength of material and the second method was based on the theory of elasticity approach by considering finite difference approximation to the infilled frames. For finding lateral stiffness and lateral strength, the same theory of diagonal strut assumption had been suggested to find lateral strength from non-dimensional parameter ' λh '. For lateral stiffness calculation, empirical formulae were suggested by considering rigid frame and non-rigid frame and concluded that the results of experimental and theoretical work differed by 15%.

Smith (1966) studied the behavior of square frames and tried to compare the theoretical results with experimental ones and derived expressions for diagonal strength. It was suggested that the concept, that the infill acts as a diagonal strut, as shown in Figure 2-6 of certain width along the loaded corners. To derive the effective width finite difference method was used. A relationship was suggested between the diagonal stiffness and a non-dimensional parameter ratio of relative stiffness of infill to the frame.

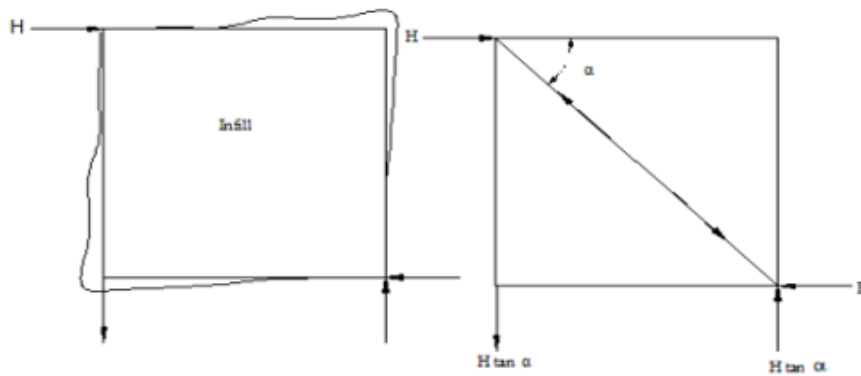


Figure 2-6: Smith's Equivalent Strut

Different concepts were proposed based on equivalent strut method considering frame/infill interaction. Mainstone (1970) proposed an empirical equation about strut width to model infill walls subjected to monotonic lateral loads using the equivalent strut approach. This empirical equation was developed by Mainstone & Weeks (1971) subsequently, included in FEMA 274, FEMA 306, FEMA 356, are widely used nowadays. The equation considers initial stiffness and ultimate strength and stiffening and strengthening effect of the masonry infill.

A 1974 study by Kadir indicated that the dimensions of the strut are affected by the adjacent columns and beams and proposed a formula to define diagonal strut dimensions.

Mehrabi et al. (1966) suggested that different levels of infill and circumambient frame strength as predictors for damage initiation for various story drift.

Perumal Pillai and Govindan (1994) have studied the structural response of two quarter size, five storey RC frame with and without brick infill and assessed the performance based on the ductility and energy absorption capacity. The frames were tested under static reversed cyclic loading to stimulate seismic effects. The study covers the entire elastic loading range from the initial elastic stage until the ultimate failure stage. The comparison of experimental and theoretical results is reported to be generally good. The failure mechanism in such case is brittle.

Goutam Modal and Sudhir K. Jain (2008) have carried out a parametric finite element analysis on single bay, single story, single bay two story and single bay three story infilled frame to examine the effect of central openings of different sizes on the initial stiffness of infilled frames. Based on the study he has concluded the effect of opening on the initial lateral stiffness of infilled frames should be neglected if the area of opening is less than 5% of

the area of the infill panel, and the strut width reduction factor should be set equal to one i.e. the frame is to be analyzed as a solid infilled frame. The effect of infill on the initial lateral stiffness of infilled frame may be ignored if the area of opening exceeds 40% of the area of the infill panel, and the strut-width reduction factor should be set to zero, i.e. the frame is to be analyzed as a bare frame. The proposed reduction factor is applicable for infilled frame with normal openings. Extreme cases where openings are extended to full height or full width of the infilled frame cannot be covered by the reduction factor.

Paulay et al. (1992) investigated the seismic performance of buildings which were severely damaged or even collapsed because of the structural modifications to the basic structural system induced by the non-structural masonry partitions. They found that relatively weak, masonry infill can drastically modify the intended structural response, attracting forces to parts of the structure that had not been designed to resist them.

Kulkarni et al. (2013) investigated the performance of RC framed building with infill, having different percentages of opening. He concluded that increase in opening percentage leads to decrease on lateral stiffness of infill frame.

Decanini et al. (2004) studied seismic analysis on the RC frames with infill walls; they showed that infill walls provide more stiffness and result a significant decrease in value of top displacement, as natural consequence of the stiffness of the buildings.

2.7. Determination of the Width of the Diagonal Strut

The width of the equivalent diagonal strut, w , can be found using several expressions given by different researchers. Holmes (1961) states that: “the width of equivalent strut is one-third of the diagonal length of infill”, which implies that the infill strength is independent of frame stiffness.

$$w = \frac{1}{3}d_{inf} \quad (2.1)$$

Where: d_{inf} is the diagonal length of infill.

Later Stafford Smith and Carter (1969) proposed a theoretical relation for the width of the diagonal strut based on the relative stiffness of infill and frame.

$$w = 0.58 \left(\frac{1}{H}\right)^{-0.445} * (\lambda_h * H_{inf})^{0.335} d_{inf} \left(\frac{1}{H}\right)^{0.064} \quad (2.2)$$

$$\lambda_h = \sqrt[4]{\frac{E_{inf} t \sin 2\theta}{4E_c I_c H_{inf}}} \quad (2.3)$$

Where:

t , H_{inf} , and E_{inf} are the thickness, the height and the modulus of the infill respectively,

θ is the angle between diagonal of the infill and the horizontal,

E_c is the modulus of elasticity of the column,

I_c is the moment of inertia of the columns,

H is the total frame height, and

λ_h is a dimensionless parameter (which considers the effect of relative stiffness of the masonry panel to the frame).

Mainstone (1970) gave equivalent diagonal strut concept by performing tests on model frames with brick infill. His approach estimates the infill contribution both to the stiffness of the frame and to its ultimate strength.

$$w = 0.16 d_{inf} (\lambda_h H_{inf})^{-0.3} \quad (2.4)$$

Mainstone and Weeks (1971) and Mainstone (1974), also based on experimental and analytical data, proposed an empirical equation for the calculation of the equivalent strut width:

$$w = 0.175 d_{inf} (\lambda_h H_{inf})^{-0.4} \quad (2.5)$$

Bazan and Meli (1980) based on parametric finite-element studies for one-bay, one-story, infilled frames, produced an empirical expression to calculate the equivalent width w for infilled frame:

$$w = (0.35 + 0.22\beta) h \quad (2.6)$$

$$\beta = \frac{E_c A_c}{G_{inf} A_{inf}} \quad (2.7)$$

Where:

β is a dimensionless parameter,

A_c is the gross area of the column,

$A_{inf} = (L_{inf} * t)$ is the area of the infill panel in the horizontal plane and

G_{inf} is the shear modulus of the infill.

Figure 2-7 illustrates the ratio w / d_{inf} according to Eq. 2.6. It is also important to note that it is difficult to compare these results with previous expressions because they are related to two different parameters ($\Theta=25^0$ and $\Theta=50^0$).

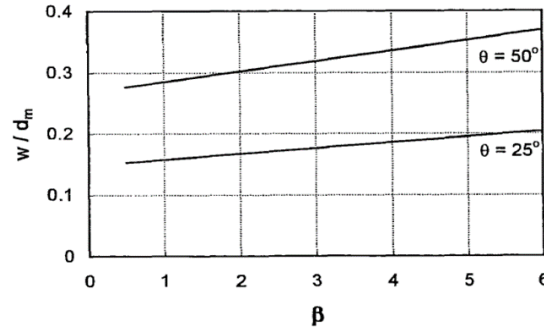


Figure 2-7: Ratio w/d_m ; for Framed Masonry Structure [Bazan & Meli, 1980]

Liau and Kwan (1984) proposed the following equations based on experimental and analytical data:

$$w = \frac{0.95 H_{inf} \cos \theta}{\sqrt{\lambda_h H_{inf}}} \quad (2.8)$$

Paulay and Priestley (1992) pointed out that a high value of w will result in a stiffer structure, and therefore potentially higher seismic response. They suggested a conservative value useful for design proposal, given by:

$$w = 0.25 d_{inf} \quad (2.9)$$

Durrani and Luo (1994) analyzed the lateral load response of reinforced concrete infilled frames based on Mainstone's equations. They proposed an equation for effective width of the diagonal strut, w , as

$$w = \gamma \sqrt{L^2 + H^2} \sin 2\theta \quad (2.10)$$

$$\text{Where: } \gamma = 0.32 \sin 2\theta \left[\frac{H^4 E_{inf} t}{m E_C I_C H_{inf}} \right]^{-0.1}; \quad (2.11)$$

$$m = 6 \left[1 + \frac{6 E_C I_b H}{\pi E_C I_C L} \right] \quad (2.12)$$

L is the length of frame C/C

FEMA 306 (1998) proposed that the equivalent strut is represented by the actual infill thickness that is in contact with the frame (t_{inf}) and the diagonal length (d_{inf}) and an equivalent width, W , is given by:

$$w = 0.175 d_{inf} (\lambda_h H_{inf})^{-0.4} \quad (2.13)$$

Hendry (1998) has also presented equivalent strut width that would represent the masonry that contributes in resisting the lateral force in the composite structure:

$$w = 0.5 \sqrt{\alpha h^2 + \alpha L^2} \quad (2.14)$$

$$\alpha h = \frac{\pi}{2} \left[\frac{4E_C I_C H_{inf}}{E_{inf} t \sin 2\theta} \right]^{1/4} \text{ and } \alpha L = \pi \left[\frac{4E_C I_b L_{inf}}{E_{inf} t \sin 2\theta} \right]^{1/4} \quad (2.15)$$

Where:

αh , αL , are contact length between wall and column and beam respectively at the time of initial failure of wall,

I_b is the moment of inertia of the beam, and L_{inf} is the length of the infill (clear distance between columns).

Al-Chaar (2002) proposed that the equivalent masonry strut is to be connected to the frame members as depicted in Figure 2-8. The infill forces are assumed to be mainly resisted by the columns, and the struts are placed accordingly. The strut should be pin-connected to the column at a distance L_{column} from the face of the beam. This distance is defined by the following equations.

$$L_{column} = \frac{W}{\cos \theta_{column}} \quad (2.16)$$

$$\tan \theta_{column} = \frac{H_{inf} - \frac{W}{\cos \theta_{column}}}{L_{infil}} \quad (2.17)$$

Where the strut width (w) is calculated by using Mainstone and Weeks Equation without any reduction factors:

$$w = 0.175 d_{inf} (\lambda_h H_{inf})^{-0.4} \quad (2.18)$$

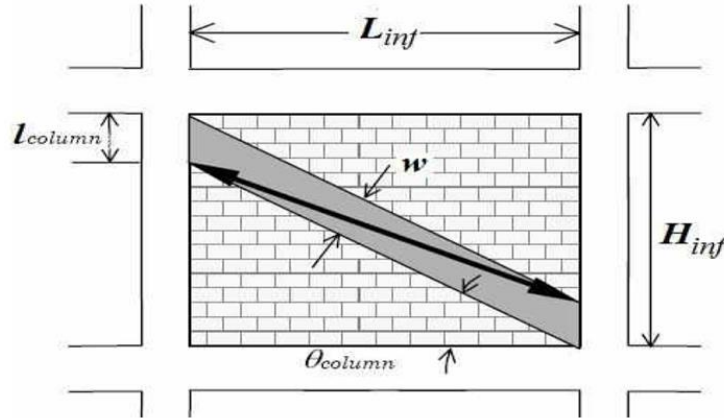


Figure 2-8: Placement of Strut [Al-Chaar, 2002]

Papia et al. (2008) developed an empirical equation for the effective width of the diagonal strut as:

$$w = \frac{c}{z} \frac{1}{\lambda} d_{inf} \quad (2.19)$$

Where:

$$c = 0.249 - 0.0116V_{inf} + 0.567V_{inf}^2 \quad (2.20)$$

$$\beta = 0.146 + 0.0073V_{inf} + 0.126V_{inf}^2 \quad (2.21)$$

$$\lambda^* = \frac{E_{inf} t H_{inf}}{E_c A_c} \left(\frac{H_{inf}^2}{L_{inf}^2} + \frac{A_c L_{inf}}{4A_b H_{inf}} \right) \quad (2.22)$$

$$Z = 1 \text{ if } \frac{L_{inf}}{H_{inf}} = 1 \ \& \ \ z = 1.125 \text{ if } \frac{L_{inf}}{H_{inf}} \geq 1.5 \quad (2.23)$$

Where z is an empirical constant, λ^* is the stiffness parameter, ν_{inf} is the poisson ratio for the infill, E_c was the Young's modulus of the frame, A_c was the cross-sectional area of the column and A_b was the cross-sectional area of the beam.

The comparative study of different expressions shows that the Paulay and Priestley equation is the most suitable choice for calculating the diagonal equivalent strut width, due to its simplicity and gives an approximate average value to experimental result (N. AL-Mekhlafy et al, 2013).

Figure 2:9 illustrates the variation of the ratio (w/d_{inf}) as a function of (H/L) according to the previous expressions. Holmes's proposition (Eq. 2:1) gives an upper-bound value for the strut

width, and Mainstone's proposition (Eq. 2:4) a lower-bound one. On the other hand, the constant value suggested by Paulay and Priestley [(Eq. 2:9) gives a value that is more or less an average value of the two extremes.

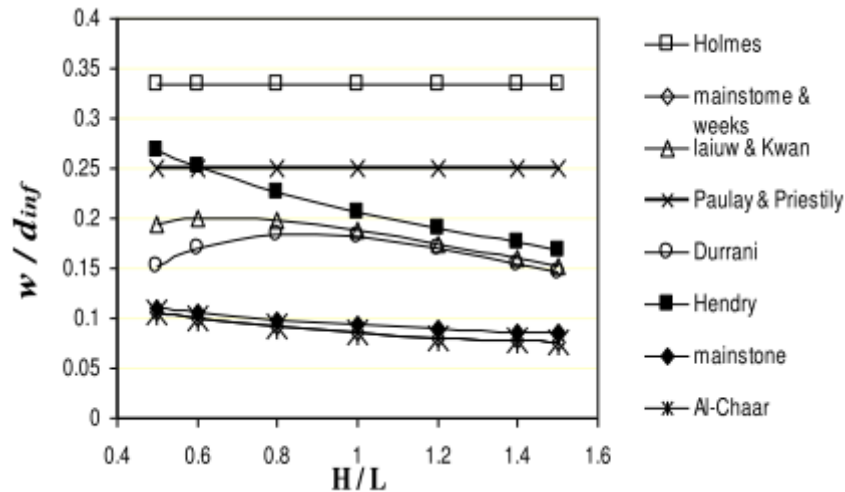


Figure 2-9: Variation of the ratio w/d_{inf} for infilled frame as a function of (H/L) [N. AL-Mekhlafy et al, 2013]

2.8. Reduction Factors for Infilled Frames with Openings

The openings in the infill walls lead to significant uncertainty in the assessment of the Seismic behavior of the structure due to the variability of their size and location. In general, the presence of openings results in a reduction of stiffness and ultimate strength of the panel and in a reduction of the energy dissipation capacity. Moreover, openings may accelerate the out-of-plane failure because the arching mechanism cannot develop as in the case of a solid infill wall. The presence of openings also affects the crack pattern as cracks may develop first at the corners of the opening and propagate towards the compressed corners. However, in general, the crack pattern depends on the position and size of the opening (Mosalam et al., 1997).

The influence of openings on strength and stiffness has been investigated by several researchers. One of the first experimental studies on infilled frames with openings was carried out by Polyakov (1956) on eight infilled steel frames with openings of different sizes. In this study, the ultimate strength of the perforated models was estimated between 23% and 76% of the frame with the solid panel. Another early study was performed by Benjamin and Williams (1958) on an infilled steel frame with a central opening with dimensions of 1/3 of the infill panel dimensions; the reduction of the ultimate strength due to the opening was about 45%. Other experimental and numerical tests were performed since 1960. Very recent research has

been carried out by Stavridis et al. (2012) and Mohammadi and Nikfar (2013). Literature reviews on the effect of openings in infill panels were carried out by Moghaddam and Dowling (1987) and by Smyrou (2006), who highlighted the lack of recommendations and of an integrated way to quantify the effect of openings.

2.9. The Reduction Factors Proposed by Different Researchers

Polyakov (1956) proposed the following expression, valid for $\alpha_h \leq 65\%$ and $\alpha_a \leq 60\%$

$$\rho_{pol} = 1 - 0.01(1.155\alpha_h + 0.385\alpha_a) \quad (2.24)$$

Sachanski (1960), based on theoretical and experimental investigations, suggested the following expression for the strut width reduction factor:

$$\rho_{sach} = 1 - (0.004\alpha_l + 0.006\alpha_h) \quad (2.25)$$

Imai (1989) used the following reduction factor for the evaluation of the shear strength in panels with openings:

$$\rho_{Imai} = \min(1 - 0.01\alpha_l ; 1 - 0.1\alpha_a^{0.5}) \quad (2.26)$$

Durrani and Luo (1994), based on finite element analyses on RC infilled frames with central openings, suggested the following formula:

$$\rho_{DL} = 1 - \left(\frac{A_d}{LH}\right)^2 ; A_d = LH - \frac{(d \sin 2\theta - d_0 \sin(\theta + \varphi))^2}{2 \sin 2\theta} \quad (2.27)$$

Where L, H, d, do, θ , and φ are geometrical features (Fig. 2.9).

According to Al-Chaar (2002), the width reduction factor should be calculated using the following equation:

$$\rho_{Al-Chaar} = 0.6 \left(\frac{\alpha_a}{100}\right)^2 - 1.6 \left(\frac{\alpha_a}{100}\right)^2 + 1 \quad (2.28)$$

Valid for $\alpha_a < 60\%$. If the area of the openings is greater than or equal to 60% of the area of the infill panel, then the effect of the infill should be neglected.

Mondal and Jain (2008) investigated the effect of central openings on the initial lateral stiffness of infilled frames by means of finite element analyses and proposed the following equation for the reduction of the strut width:

$$\rho_{MJ} = 1 - 2.6 \left(\frac{\alpha a}{100} \right) \quad (2.29)$$

This equation implies that when $\alpha a > 38.5\%$, the contribution of the infill is neglected.

The above expressions do not consider the presence of reinforcing elements around the opening.

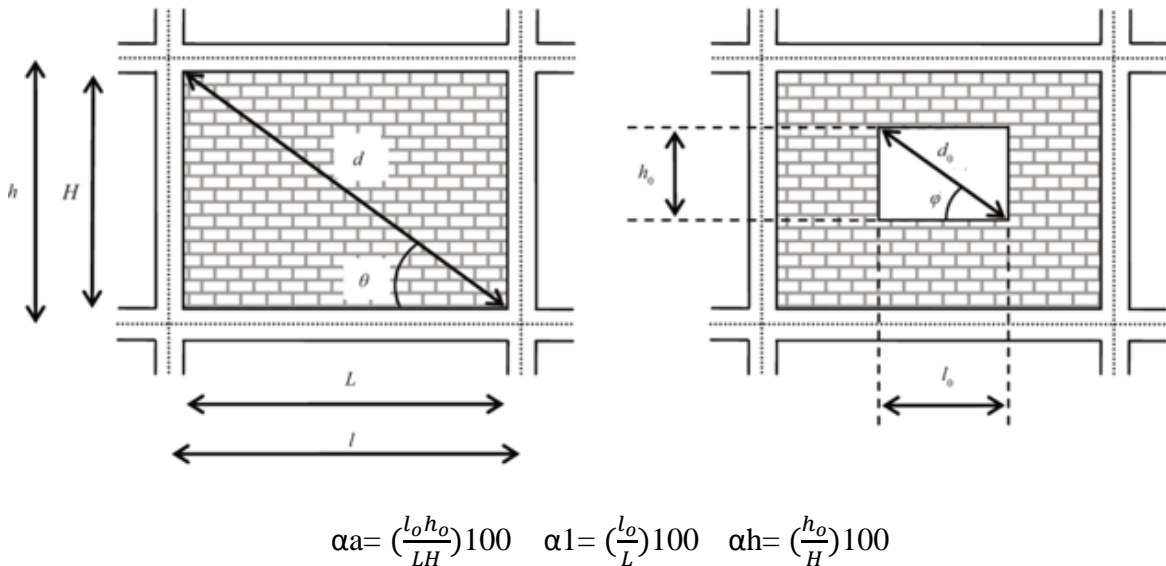


Figure 2-10: Notations for Reduction Factor Parameters

2.10. Summary of Literature Review

Past and present research in infilled frames has been motivated by using brickwork infill in resisting blast loads, providing stability to the tall buildings, rehabilitation of masonry structures and in seismic engineering. In this chapter experimental research, Influence of masonry infill on seismic behavior of frames and modeling strategies used in the study of infilled frames has been reviewed.

In the following, Table 2-5 and Table 2-6 summarized width of the diagonal strut and the reduction factors proposed by different researchers.

Table 2-5: Width of the Diagonal Strut by Different Researchers

Holmes	$w = \frac{1}{3} d_{inf}$
Stafford Smith and Carter	$w = 0.58 \left(\frac{1}{H} \right)^{-0.445} * (\lambda_h * H_{inf})^{0.335} d_{inf} \left(\frac{1}{H} \right)^{0.064}$
Mainstone	$w = 0.16 d_{inf} (\lambda_h H_{inf})^{-0.3}$

Mainstone and Weeks	$w = 0.175 d_{inf} (\lambda_h H_{inf})^{-0.4}$
Bazan and Meli	$w = (0.35 + 0.22\beta)h$
Liau and Kwan	$w = \frac{0.95 H_{inf} \cos \theta}{\sqrt{\lambda_h H_{inf}}}$
Paulay and Priestley	$w = 0.25 d_{inf}$
Durrani and Luo	$w = Y\sqrt{L^2 + H^2} \sin 2\theta$
FEMA	$w = 0.175 d_{inf} (\lambda_h H_{inf})^{-0.4}$
Hendry	$w = 0.5\sqrt{\alpha h^2 + \alpha L^2}$
Al-Chaar	$w = 0.175 d_{inf} (\lambda_h H_{inf})^{-0.4}$
Papia et al.	$w = \frac{c}{z} \frac{1}{\lambda} d_{inf}$

Nowadays, equivalent diagonal strut model extensively used to model infill because of simple and reasonable procedure to characterize the effect of the masonry infill on surrounding frame.

Table 2-6: The Reduction Factors Proposed by Different Researchers

Polyakov	$\rho_{pol} = 1 - 0.01(1.155\alpha h + 0.385\alpha a)$
Sachanski	$\rho_{sach} = 1 - (0.004 \alpha l + 0.006 \alpha h)$
Imai	$\rho_{Imai} = \min(1 - 0.01 \alpha l ; 1 - 0.1 \alpha a^{0.5})$
Durrani and Luo	$\rho_{DL} = 1 - \left(\frac{A_d}{LH}\right)^2 ; \quad A_d = LH - \frac{(d \sin 2\theta - d_0 \sin(\theta + \varphi))^2}{2 \sin 2\theta}$
Al-Chaar	$\rho_{Al-Chaar} = 0.6 \left(\frac{\alpha a}{100}\right)^2 - 1.6 \left(\frac{\alpha a}{100}\right)^2 + 1$
Mondal and Jain	$\rho_{MJ} = 1 - 2.6 \left(\frac{\alpha a}{100}\right)$

From the analytical studies and experimental investigations, the following observations have been made.

- The stiffness and strength of the frames are significantly improved by use of infill.
- Infill basically acts as a form of bracing.
- Diagonal tensile cracking, shear sliding in the joints and crushing of the infill have been observed as modes of damage.

CHAPTER THREE

3. MODELING

3.1. General

It is very important to develop a mathematical model on which response spectrum analysis is performed. The first part of this chapter presents a summary of various parameters defining the computational models, the basic assumptions and the geometry of the selected building considered for this study. Infill walls are modeled as equivalent diagonal strut elements. The last part of this chapter deals with the computational model of the equivalent strut.

3.2. Building Description

Multi-storey rigid jointed frame office buildings G+10 were selected in the seismic zone, Zone 5, of Ethiopia and analyzed based on the Ethiopian Building Code EBCS EN 1998-1:2014. ETABS 2017 was used for the analysis and design of the building by modeling as a 3D space frame system.

The plan dimensions of the buildings are 25m x 25m and the height of the buildings are 33m, with typical story height of 3m with span length 5m and is made of reinforced concrete (RC) ordinary moment resisting frames (OMRF). The concrete slab is 150mm thick at each floor level and 120mm thick for roof slab. Imposed live load is taken as 4 kN/m² for all floors. Figure 3-1 presents typical floor plan showing different column and beam locations.

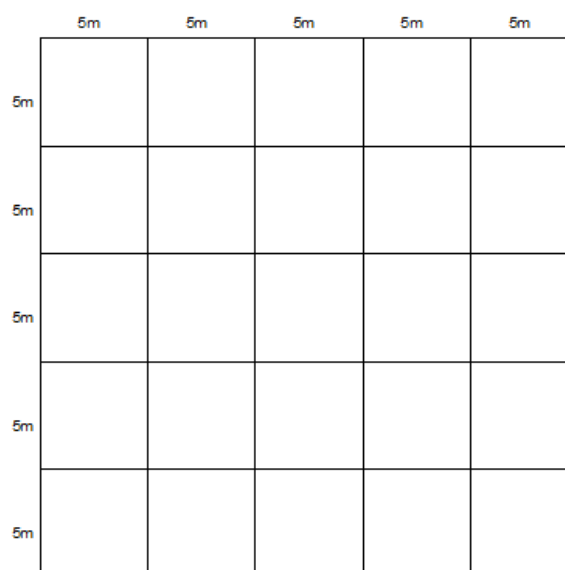


Figure 3-1: Beam Column Lay Out

Buildings Data

- Type of structure = Multi-storey rigid jointed frame
- Layout = as shown in Figure 3.1
- Zone = 5
- Importance Factor = 1
- Soil Condition = Type B
- Number of stories = Eleven (G+10)
- Height of Building =33m
- Story height = 3m
- External wall thickness =20cm
- Internal wall thickness=15cm
- Depth of the floor slab =15cm
- depth of roof slab=12cm
- Size of all columns = 70 x 70cm
- Size of all beams = 50 × 50cm
- Tie beam =30 x 30cm
- Door opening size=100 x 200cm
- Window opening size =200 x 120cm

Initial dimensioning of the beams and columns were made based on bare frame. The same sections were used for the cases of infilled frames analysis with earthquake load as per EN 1998-1:2004 such that the structure met the strength and ductility requirements of the new Ethiopian building code of standard EBCS EN 1998-1:2014. Further, it was assumed that the infill panels were neither integral nor bonding with the frame. Six different models with infill arranged irregularly were developed to analyze and to investigate the effect of infill wall materials on eccentricity of building.

The Beam column layout of the base structure shown in Figure 3-1 is configured with uniform column spacing of 5meter both along x and y axis. The building has uniform story height of 3meter from the base to top story. The building is regular both in plan and irregular in elevation.

3.3. Different Arrangement of Models

Different models which have different infill wall material arrangements are prepared in addition to that of the bare frame model; its typical floor plan was as shown in Figure 3-2. The sample building taken from bare to severe irregularities in plan due to the unsymmetrical arrangement of the infills.

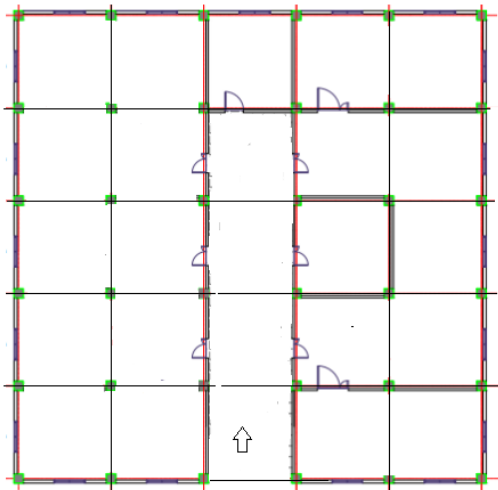


Figure 3-2: Model 1

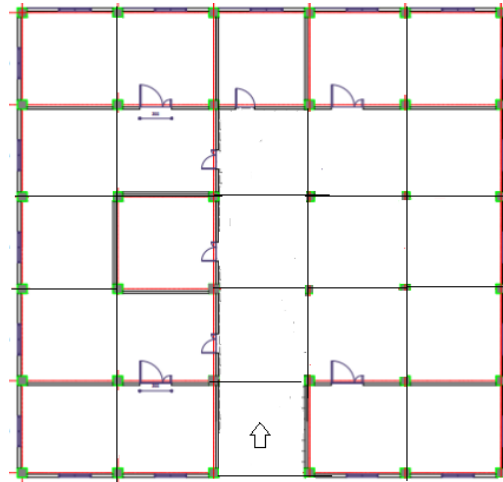


Figure 3-3: Model 2

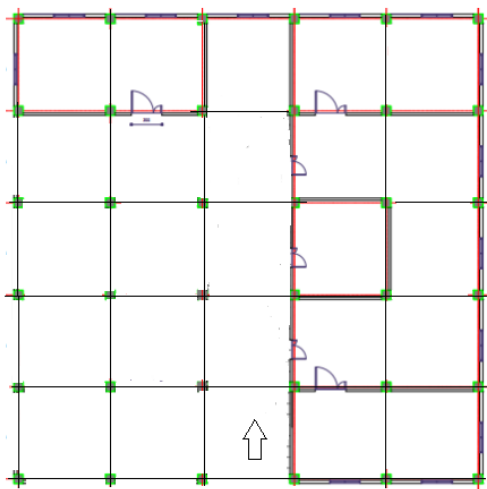


Figure 3-4: Model 3

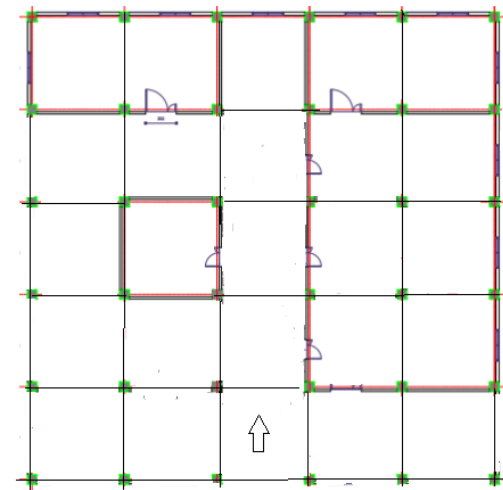


Figure 3-5: Model 4

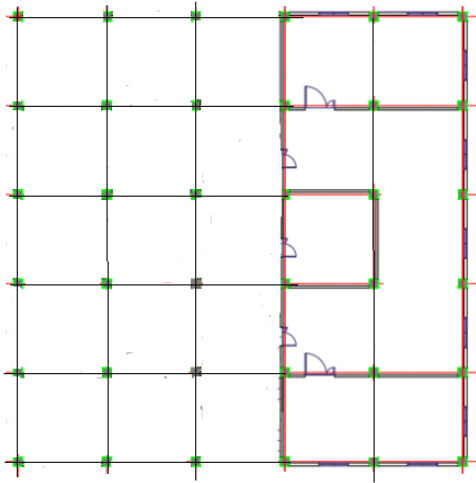


Figure 3-6: Model 5

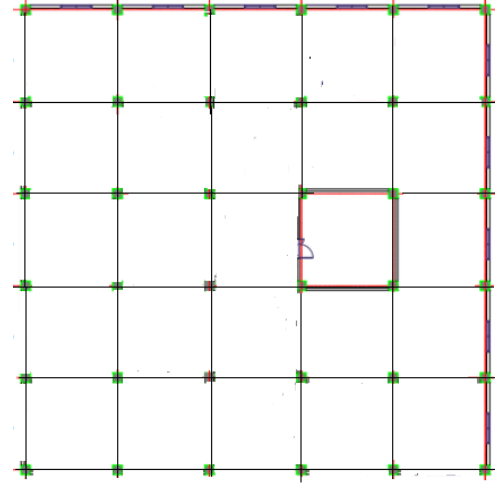


Figure 3-7: Model 6

3.4. Structural Modeling

Modeling a building involves the modeling and assemblage of its various load-carrying elements. The model must ideally represent the mass distribution, strength, stiffness and deformability. Modeling of the material properties and structural elements used in this study is discussed below.

3.4.1. Material Properties Used

For this study, the material property for concrete, reinforcing bar and different infill masonry material are as follows:

Reinforcing Bar:

- Yield strength of reinforcing bar $f_y = 500$ Mpa

Concrete:

- Unit weight (weight per unit volume) = 25 kN/m^3
- Grade of Concrete = C30/37
- Young's modulus of elasticity, $E_c = 32000$ Mpa
- Poisson's ratio, $\nu_c = 0$ for cracked concrete
- Shear Modulus, $G_c = E_c / 2(1+\nu_c) = 16000$ Mpa

For Masonry infill materials

- Horizontal mortar thickness = 2cm
- Mortar ratio = 1:3

- Unit weight (weight per unit volume) = 12 kN/m³ for infill material
- Characteristic compressive strength, f_{cm}
= 4.2 Mpa for class A HCB

Young's modulus of elasticity of the masonry, calculated by the relation given by Paulay and Priestley, 1992;

$$E_m = 750\sqrt{f_{cm}} \quad (3.1)$$

- Poisson's ratio, $\nu_m = 0$
- $G_c = \frac{E_c}{2(1+\nu_c)}$ (3.2)

Table 3-1: Infill Materials Properties

Infill material	$E_m = 750\sqrt{f_{cm}}$ (Mpa)	$G_c = E_c / 2(1+\nu_c)$ (Mpa)
HBC class A	1537	769
HBC class B	1403	702
HBC class C	1061	531
HCB	1984	992

3.5. Load Cases Used

Dead load: The Unit Weights of Materials used in this study is based on EBCS EN 1991-1-1.

Imposed Load: the imposed load used in this study is based on EBCS EN 1991-1-1

Earthquake Load: EBCS EN 1998-1:2014 Criteria for Earthquake Resistant design of Structure was used.

Load combination used

Table 3-2: Equivalent Static Force Case

Load case name	Direction and Eccentricity	% Eccentricity
EQX1	Xdir + Eccen.	0.05
EQX2	Xdir - Eccen.	0.05
EQY1	Ydir + Eccen.	0.05
EQY2	Ydir - Eccen.	0.05

Equivalent Static Force Case

- Comb1= DL+0.3LL
- Comb 1 \pm EQX1 \pm 0.3 EQY1,Y2 \pm Imp x,y (imperfection)
- Comb 1 \pm EQX2 \pm 0.3 EQY1,Y2 \pm Imp x,y
- Comb 1 \pm EQY1 \pm 0.3 EQX1,X2 \pm Imp x,y
- Comb 1 \pm EQY2 \pm 0.3 EQX1,X2 \pm Imp x,y

Response spectrum case

- Comb 1 \pm EQX Spectrum \pm 0.3 EQY Spectrum \pm Imp x,y
- Comb 1 \pm EQY Spectrum \pm 0.3 EQX Spectrum \pm Imp x,y

Total 160 load combination used for analysis of buildings.

3.6. Structural Elements

The reference structures studied are G+10 and analyzed according to the Eurocode 8 ductility class medium (DCM) provisions for bare frames for seismic zone five. The structures have been assessed both as a bare frame and as an infilled one. The beams and columns are modeled as a frame element which has the capability to deform axially, in shear, in bending and in torsion. The beam-column joints are assumed to be rigid. The weight of the slab is distributed as rectangular load to the surrounding beams as per EBCS EN 1992-1-1:2014. A rigid joint diaphragm action was assumed, which ensure integral action of all the vertical and lateral load-resisting elements.

3.7. Modeling Infill Walls

In the case of an infill wall located in a lateral load resisting frame the stiffness and strength contribution of the infill are considered by modeling the infill as an equivalent compression strut. Because of its simplicity, several investigators have recommended the equivalent strut concept. In the present analysis, a trussed frame model is considered. This type of model does not neglect the bending moment in beams and columns. Rigid joints connect the beams and columns, but pin joints connect the equivalent struts.

3.8. Modeling of Equivalent Strut

For an infill wall located in a lateral load-resisting frame, the stiffness and strength contribution of the infill must be considered. Non-integral infill walls subjected to lateral load behave like compression diagonal struts. Thus, an infill wall can be modeled as an equivalent diagonal compression strut in the building model.

The equivalent struts modeling parameters (effective width, elastic modulus and strength) were estimated using equations derived by Paulay and Priestley (1992). The length of the strut is given by the diagonal distance (d) of the panel and its thickness is equal to the thickness of the infill wall. The elastic modulus of the strut is equated to the elastic modulus of masonry (E_m). For the estimation of width (w) of the strut, a simple expression as given in Equation 2.4 is adopted.

After modeling the bare frame, the equivalent eccentric diagonal struts are added to represent the masonry infill as shown in Figure 3-8. Since most of the panels are fully infilled, the struts should, at first, be designed to represent full infill panels, and then multiplied by a proper reduction factor to account for any openings in the infill panel as stated in section 2.10.

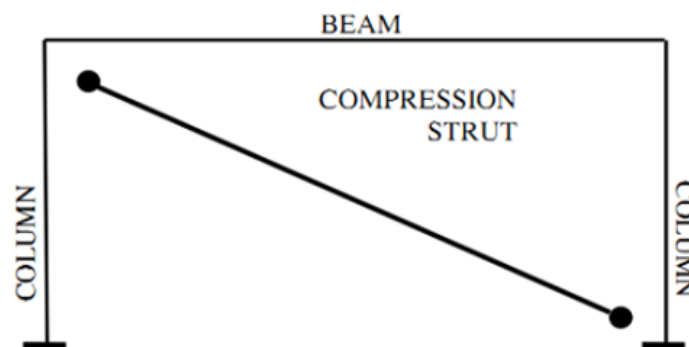


Figure 3-8: Connection of Equivalent Strut Model [Paulay and Priestley, 1992]

3.9. Iterative Linear Modal Analysis

Because the equivalent compressive struts cannot carry any tensile force, an iterative procedure was applied to find center of mass and center of rigidity of the models. In this procedure, the equivalent compressive struts were assumed to show the same linear behavior under compression and tension. After the analysis, stresses on the equivalent compressive struts were checked to make sure that they were all under compression. Whenever an equivalent compressive strut was observed to be under tension, it was removed, and then the

model was reanalyzed. This procedure was repeated until all the equivalent compressive struts were under compression.

The sum of the effective modal masses for the modes considered amounts to at least 90% of the total mass of the structure according to the requirements stated in EBCS EN 1998-1:2014. All modes with effective modal masses greater than 5% of the total mass are considered.

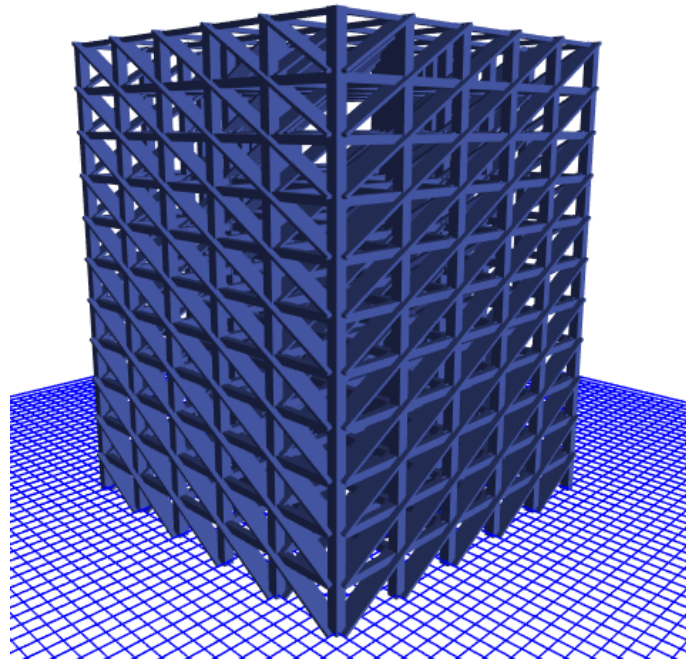


Figure 3-9: Sample Modeling of G+10 Building with Infill wall, Model 6

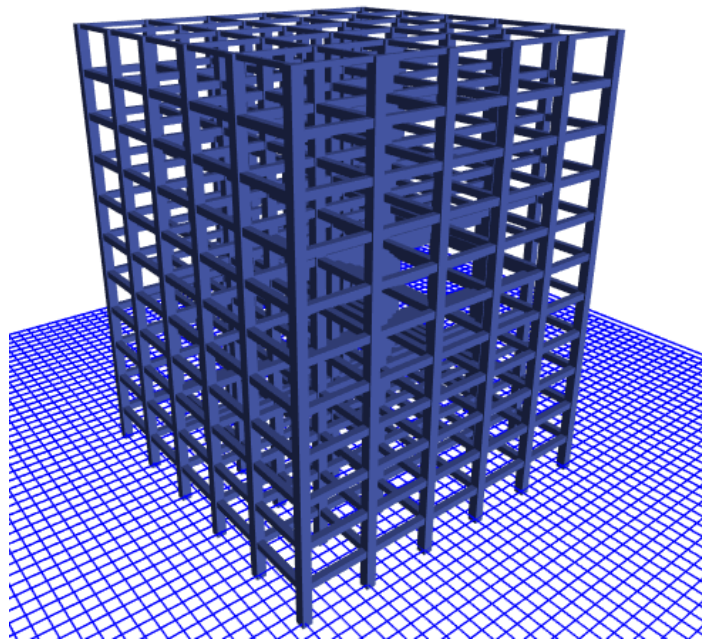


Figure 3-10: Sample Modeling of Bare G+10 Building [3D ETABS, 2017]

3.10. Damage limitation

The damage limitation requirements verified in terms of the interstorey drift (dr) according to EBCS EN 1998-1:2014 using the equation below:

$$dr * V \leq a * h \rightarrow \frac{dr}{h} \leq \frac{a}{h} \quad (3.3)$$

Where:

dr: is the design interstorey drift

v: is the reduction factor which takes into account the lower return period of the seismic action

h: is the storey height

Table 3-3: Damage Limitation for Sample G+10 Bare Building

	Drift X, dr (mm)	Drift Y, dr (mm)	Height of story, h (mm)	V	X dir. $\frac{V * dr}{h}$	Y dir. $\frac{V * dr}{h}$	X dir. $\frac{V * dr}{h} < 0.005$	Y dir. $\frac{V * dr}{h} < 0.005$
Story 11	2.03	2.00	3000	0.5	0.0003	0.0003	Ok	Ok
Story 10	2.58	2.54	3000	0.5	0.0004	0.0004	Ok	Ok
Story 9	3.61	3.56	3000	0.5	0.0006	0.0006	Ok	Ok
Story 8	4.69	4.61	3000	0.5	0.0008	0.0008	Ok	Ok
Story 7	5.69	5.60	3000	0.5	0.0009	0.0009	Ok	Ok
Story 6	6.62	6.51	3000	0.5	0.0011	0.0011	Ok	Ok
Story 5	7.47	7.35	3000	0.5	0.0012	0.0012	Ok	Ok
Story 4	8.18	8.05	3000	0.5	0.0014	0.0013	Ok	Ok
Story 3	8.53	8.39	3000	0.5	0.0014	0.0014	Ok	Ok
Story 2	7.84	7.71	3000	0.5	0.0013	0.0013	Ok	Ok
Story 1	4.17	4.10	3000	0.5	0.0007	0.0007	Ok	Ok
Base	0.00	0.00	3000	0.5	0.0000	0.0000	Ok	Ok

CHAPTER FOUR

4. RESULTS AND DISCUSSIONS

4.1. General

The bare building and infill building were studied analytically. Based on the results obtained from analytical analysis, the behaviour of structural system in terms eccentricity of stories are compared in this section of the report. For earthquake resistant design of structure, there are two types of irregularities namely.

- Plan Irregularity
- Vertical Irregularity

Plan Irregularity

The condition of being non-uniform in the plan of a structure is called plan irregularity. These can be characterized by five different types such as torsional, re-entrant corners, diaphragms discontinuity, out of plane offset and nonparallel system for plan irregularity.

Vertical Irregularity

Structures having significant physical discontinuities in a vertical configuration or in their lateral force resisting systems are termed as vertically irregular structure. The vertical irregularities in structures are Stiffness irregularity, Mass irregular, Vertical geometric irregularity, Discontinuity in capacity.

4.2. Centre of Mass

The centre of mass is a position defined as the average position of all the parts of the system, weighted according to their masses. The distribution of mass is balanced around the centre of mass and the average of the weighted position coordinates of the distributed mass defines its coordinates. During an earthquake, acceleration-induced inertia forces will be generated at each floor level and it will act at point, where the mass of entire story may be assumed to be concentrated. In a building having a symmetrical distribution of mass the positions of the centers of floor masses will not differ from floor to floor. However, irregular mass distribution over the height of a building may result in variation in centers of masses at various floors.

4.3. Centre of Rigidity

The centre of rigidity is a point at a particular story as the location of application of lateral load at that point will not produce rotation of that story. This definition is valid when the slab is modeled as a rigid diaphragm. A Diaphragm Constraint causes all of its constrained joints to move together as a planar diaphragm that is rigid against membrane deformation. As a function of structural properties, the center of rigidity is independent of loading. This study focusing on the behavior of buildings with respect to the center of mass and centre of rigidity under the action of seismic load using response spectrum analysis on irregular arranged infill wall buildings.

4.4. Effect of Infill Wall Material

4.4.1. Center of mass and Center of Rigidity

The Center of Rigidity and Center of Mass of all the six selected buildings were computed using Response spectrum methods available for modal analysis. This method has been explained in Chapter two and three of this report. The shifting of Center of Rigidity are presented in Table 4-1 to 4-6.

Table 4-1: Location of Centre of Mass and Rigidity for Model One

Story	CM		CR		The Eccentricity							
	XCM	YCM	XCR	YCR	Eccentricity		Dimension		ex/Dx		ey/Dy	
	m	M	m	m	Ex	ey	Dx	Dy	%		%	
11	12.56	12.59	12.28	12.02	0.28	0.57	25	25	1.12	Oky	2.29	oky
10	12.58	12.52	12.20	12.13	0.38	0.39	25	25	1.53	Oky	1.57	oky
9	12.58	12.52	12.15	12.15	0.43	0.36	25	25	1.73	Oky	1.44	oky
8	12.58	12.51	12.09	12.16	0.50	0.35	25	25	1.99	Oky	1.40	oky
7	12.58	12.51	12.00	12.18	0.58	0.33	25	25	2.32	Oky	1.31	oky
6	12.58	12.51	11.88	12.19	0.70	0.32	25	25	2.80	Oky	1.28	oky
5	12.58	12.51	11.70	12.22	0.88	0.29	25	25	3.52	Oky	1.17	oky
4	12.58	12.51	11.40	12.24	1.18	0.27	25	25	4.73	Oky	1.08	oky
3	12.58	12.51	11.38	12.28	1.20	0.23	25	25	4.80	Oky	0.92	oky
2	12.58	12.51	11.36	12.34	1.22	0.17	25	25	4.89	Oky	0.68	oky
1	12.58	12.51	11.25	12.48	1.33	0.03	25	25	5.32	N.oky	0.12	oky

Effect of Masonry infill Materials on Dynamic Characteristics of Buildings

Table 4-2: Location of Centre of Mass and Rigidity for Model Two

Story	CM		CR		The Eccentricity							
	XCM	YCM	XCR	YCR	Eccentricity		Dimension		ex/Dx		ey/Dy	
	m	M	m	m	Ex	ey	Dx	Dy	%		%	
11	12.19	12.58	12.21	12.05	0.03	0.52	25	25	0.10	Ok	2.09	ok
10	12.41	12.52	11.90	12.15	0.52	0.37	25	25	2.06	Ok	1.49	ok
9	12.43	12.52	11.85	12.18	0.58	0.34	25	25	2.32	Ok	1.37	ok
8	12.44	12.52	11.79	12.19	0.65	0.33	25	25	2.59	Ok	1.32	ok
7	12.45	12.51	11.71	12.21	0.73	0.31	25	25	2.93	Ok	1.23	ok
6	12.45	12.51	11.60	12.22	0.85	0.30	25	25	3.39	Ok	1.18	ok
5	12.45	12.51	11.43	12.24	1.02	0.27	25	25	4.08	Ok	1.09	ok
4	12.45	12.51	11.14	12.26	1.31	0.25	25	25	5.23	N.Ok	1.01	ok
3	12.45	12.51	10.59	12.30	1.86	0.22	25	25	7.44	N.ok	0.86	ok
2	12.45	12.51	10.29	12.35	2.16	0.16	25	25	8.64	N.ok	0.64	ok
1	12.45	12.51	10.69	12.48	1.76	0.03	25	25	7.03	N.ok	0.13	ok

Table 4-3: Location of Centre of Mass and Rigidity for Model Three

Story	CM		CR		The Eccentricity							
	XCM	YCM	XCR	YCR	Eccentricity		Dimension		ex/Dx		ey/Dy	
	m	M	m	m	Ex	ey	Dx	Dy	%		%	
11	12.91	12.49	14.08	11.37	1.17	1.12	25	25	4.66	Ok	4.49	Ok
10	12.71	12.58	14.20	11.34	1.48	1.24	25	25	5.93	N.ok	4.97	ok
9	12.70	12.59	14.16	11.37	1.46	1.23	25	25	5.84	N.ok	4.90	ok
8	12.69	12.59	14.13	11.39	1.44	1.21	25	25	5.75	N.ok	4.83	ok
7	12.69	12.60	14.09	11.41	1.41	1.18	25	25	5.63	N.ok	4.74	ok
6	12.68	12.60	14.05	11.44	1.37	1.16	25	25	5.46	N.ok	4.63	ok
5	12.68	12.60	13.99	11.49	1.31	1.11	25	25	5.22	N.ok	4.45	ok
4	12.68	12.60	13.90	11.55	1.22	1.04	25	25	4.86	Ok	4.18	ok
3	12.68	12.60	13.75	11.67	1.07	0.93	25	25	4.27	Ok	3.73	ok
2	12.68	12.60	13.49	11.86	0.81	0.74	25	25	3.22	Ok	2.95	ok
1	12.68	12.60	13.04	12.22	0.36	0.38	25	25	1.43	Ok	1.53	ok

Table 4-4: Location of Centre of Mass and Rigidity for Model Four

Story	CM		CR		The Eccentricity							
	XCM	YCM	XCR	YCR	Eccentricity		Dimension		ex/Dx		ey/Dy	
	m	M	M	m	Ex	ey	Dx	Dy	%		%	
11	12.56	11.76	13.64	10.88	1.08	0.87	25	25	4.30	Ok	3.49	ok
10	12.54	12.04	13.59	10.67	1.05	1.37	25	25	4.21	Ok	5.49	N.ok
9	12.54	12.06	13.56	10.70	1.02	1.36	25	25	4.08	Ok	5.44	N.ok
8	12.53	12.07	13.53	10.73	1.00	1.34	25	25	3.99	Ok	5.37	N.ok
7	12.53	12.08	13.51	10.76	0.97	1.32	25	25	3.89	Ok	5.27	N.ok
6	12.53	12.08	13.47	10.80	0.94	1.28	25	25	3.76	Ok	5.12	N.ok
5	12.53	12.08	13.43	10.87	0.90	1.21	25	25	3.60	Ok	4.85	ok
4	12.53	12.08	13.38	10.98	0.84	1.11	25	25	3.37	Ok	4.43	ok

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3	12.53	12.09	13.29	11.15	0.75	0.93	25	25	3.01	Ok	3.73	ok
2	12.53	12.09	13.13	11.47	0.60	0.62	25	25	2.40	Ok	2.47	ok
1	12.53	12.09	12.87	12.01	0.34	0.07	25	25	1.36	Ok	0.29	ok

Table 4-5: Location of Centre of Mass and Rigidity for Model Five

Story	CM		CR		The Eccentricity							
	XCM	YCM	XCR	YCR	Eccentricity		Dimension		ex/Dx		ey/Dy	
	m	M	M	m	ex	ey	Dx	Dy	%		%	
11	13.29	12.55	14.21	12.47	0.92	0.09	25	25	3.67	Ok	0.35	ok
10	12.86	12.40	14.58	12.65	1.72	0.25	25	25	6.89	N.ok	1.00	ok
9	12.82	12.39	14.54	12.64	1.72	0.25	25	25	6.88	N.ok	1.01	ok
8	12.81	12.38	14.52	12.65	1.71	0.26	25	25	6.83	N.ok	1.05	ok
7	12.80	12.38	14.48	12.64	1.68	0.26	25	25	6.73	N.ok	1.06	ok
6	12.80	12.38	14.44	12.64	1.64	0.26	25	25	6.56	N.ok	1.04	ok
5	12.79	12.38	14.37	12.64	1.57	0.26	25	25	6.28	N.ok	1.05	ok
4	12.79	12.38	14.25	12.64	1.46	0.26	25	25	5.83	N.ok	1.05	ok
3	12.79	12.38	14.06	12.64	1.27	0.26	25	25	5.07	N.ok	1.04	ok
2	12.79	12.38	13.72	12.63	0.93	0.25	25	25	3.72	Ok	1.02	ok
1	12.79	12.38	13.14	12.63	0.35	0.26	25	25	1.41	Ok	1.03	ok

Table 4-6: Location of Centre of Mass and Rigidity for Model Six

Story	CM		CR		The Eccentricity							
	XCM	YCM	XCR	YCR	Eccentricity		Dimension		ex/Dx		ex/Dy	
	m	M	m	m	ex	ey	Dx	Dy	%		%	
11	12.93	12.12	14.57	10.65	1.64	1.47	25	25	6.57	N.ok	5.86	N.ok
10	12.68	12.20	14.73	10.69	2.05	1.51	25	25	8.18	N.ok	6.04	N.ok
9	12.66	12.21	14.68	10.72	2.02	1.48	25	25	8.08	N.ok	5.94	N.ok
8	12.65	12.21	14.65	10.76	2.00	1.45	25	25	8.00	N.ok	5.81	N.ok
7	12.65	12.21	14.62	10.79	1.97	1.42	25	25	7.88	N.ok	5.69	N.ok
6	12.65	12.21	14.57	10.83	1.92	1.38	25	25	7.69	N.ok	5.53	N.ok
5	12.64	12.21	14.49	10.90	1.85	1.31	25	25	7.39	N.ok	5.24	N.ok
4	12.64	12.21	14.37	11.01	1.73	1.20	25	25	6.91	N.ok	4.81	N.ok
3	12.64	12.21	14.17	11.19	1.53	1.02	25	25	6.11	N.ok	4.10	N.ok
2	12.64	12.21	13.82	11.49	1.18	0.72	25	25	4.72	Ok	2.87	ok
1	12.64	12.21	13.24	12.01	0.60	0.21	25	25	2.42	Ok	0.82	ok

Results obtained from the analysis with regards to the shift in the position of center of rigidity with respect to different models considered in the analysis. Tables 4:1 -4:6 shows the location of centre of mass and center of rigidity with respect to X and Y directions for different models. In the tables, XCM and YCM represent the location of centre of mass with respect to X and Y directions respectively and similarly XCR and YCR represent the location of centre of rigidity with respect to X and Y directions respectively.

In this study, the results were presented in tabular form. The results of Center of mass and Center of rigidity are presented for all the models, and simultaneously the results of buildings are compared with provision of accidental eccentricity of building.

In order to achieve provision of accidental eccentricity of bare frame strongly irregular, unsymmetrical or non-uniform arrangements of infills in plan should be avoided (taking into account the extent of openings and perforations in infill panels).

When the masonry infill of the type implied to be used, there are two design alternatives. The infill may effectively isolate from frame deformation by providing flexible strip between the frame and panel, filled with a highly deformable material. Alternatively Infills should be included in the model and a sensitivity analysis regarding the position and the properties of the infills should be performed.

CHAPTER FIVE

5. CONCLUSION AND RECOMMENDATION

5.1. Conclusion

In this study, the effects of infill materials on the Eccentricity of RC buildings were investigated. The Eccentricity of RC buildings are determined by a modal analysis, six building models generated and analyzed in 3D using ETABS 2017. Due to the nonlinear behavior of infill walls an iterative modal analysis determine dynamic characteristic of buildings.

Nowadays the buildings with irregularities are more common because of the need or requirement of the individual and due to aesthetic appearance of the buildings. Also, the consideration of centre of mass and centre of rigidity while designing a structure for seismic loads plays a major role. From the present study it can be seen that for a vertical regular building the centre of mass and centre of rigidity were exactly at the centre of building in plan view. Whereas for a structure with vertical irregularities like buildings with infill the location of centre of mass and centre of rigidity moves to a concentrated region or in other words the point moves towards the region of more area.

Accidental eccentricity is expected to account for all unforeseen sources of torsion that cannot be readily accounted for in the mathematical model. These unforeseen sources can be categorized as uncertainty stemming from structural geometry and material properties, and that due to ground motion, especially, its torsional component. Conventional approach is to shift the center of mass on either side by a code specified, 5%, accidental eccentricity.

It was found that RC frames with infill walls had eccentricity, about 0.13% L – 8% L but provision of accidental eccentricity recommend 5% L. therefore provision of accidental eccentricity of irregular building must be increase to minimize torsion effect or Infills should be included in the model and a sensitivity analysis regarding the position and the properties of the infills should be performed.

5.2. Future Recommendations

- The macro modeling approach used here considers only the equivalent global behavior of the infill in the analysis procedures. As a result, the approach does not permit study of local effects such as frame-infill interaction within the individual infilled frame. More detailed micro-modeling approaches need to be used to capture the local conditions within the infill.
- The present study carried out uses linear response spectrum analysis method for the seismic analysis. This could be extended to nonlinear dynamic analysis to cater for structures with horizontal as well as vertical irregularity.

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APPENDIX A

VERIFICATION OF STRUT WIDTH

Experimental Study

Experiments are very important to observe the behavior of complex structures. Analytical models have been developed based on experimental results, and sometimes, experimental studies have been carried out to verify the analytically developed model.

Sumat Shrestha (2005) prepared models in 1:3 reduced scale single bay single story model of RC frame with unreinforced full infill panel. The outer dimension was 985mm between column and floor height 1003 mm. Infill panel was built with 75 mm x 35 mm x 10 mm block in 1:4 cement sand mortar. The sizes of both beam and columns were 75 mm x 75 mm. The specimens were tested under monotonic static loading applied at roof level.

For modeling of the specimens, geometric properties and properties of material used in these specimens are shown in Figure A.1, and properties of materials are listed in Table A.1. During the analytical analysis loads on the models are applied in the same way as those were applied on the specimens in the experimental studies.

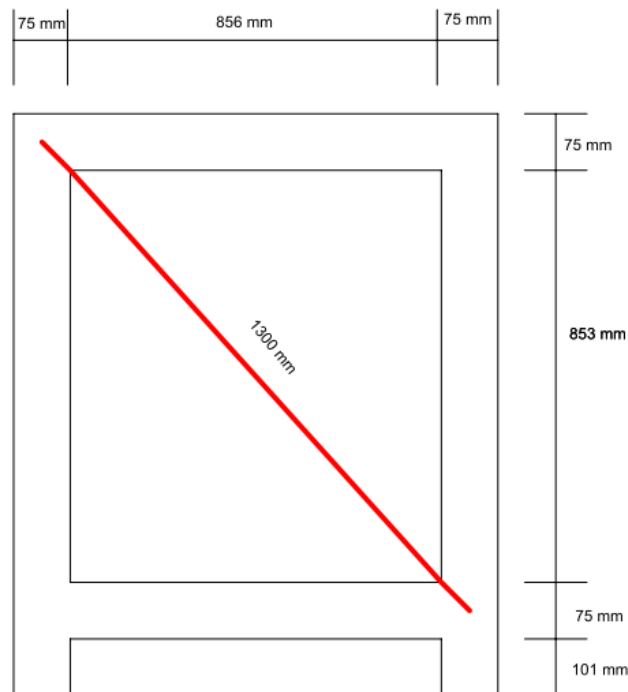


Figure A-1: Geometric Properties of Test Sample

Table A-1: Material Properties

Section	Cross-Section (mm)	center line dimension (mm)	Comp. Strength (Mpa)	Young's Modulus (Mpa)	Poisons Ratio	Long Reinf. (Mpa)
Beam	75*75	928	7.93	12500	0.15	248
Column	75*75	910	7.93	12500	0.15	248
Infill	832*853	1300		225	0.17	

Analytical Study of Specimens

The specimen for infill frame was modeled using equivalent diagonal strut method using ETABS 2016 as shown in figure A.2 using strut widths as proposed Pauley & Priestley. The experimental as well as analytical results are shown in tabular form in table A.2 and graphically in figure A.3 bellow.

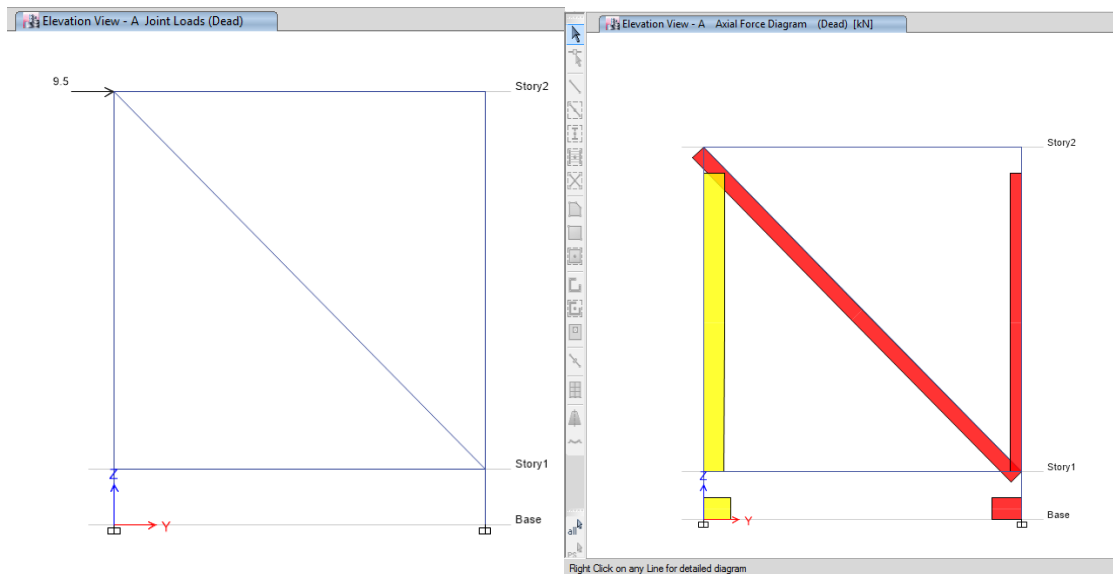


Figure A-2: Analytical Model and Result

Table A-2: Comparison of Analytical and Experimental Result

sample No.	Load (kN)	Deflection (mm) from analytical study	Deflection (mm) from experimental study	Ratio (Col 1 / Col 2)
1	9.5	12.255	12.2	1.005
2	9	11.61	11.55	1.005
3	8.5	10.965	10.9	1.006
4	8	10.32	10.25	1.007
5	7.5	9.675	9.61	1.007
6	7	9.03	8.95	1.009
7	6.5	8.385	8.31	1.009
8	6	7.74	7.66	1.010
9	5.5	7.095	7.01	1.012
10	5	6.449	6.36	1.014
11	4.5	5.804	5.72	1.015
12	4	5.159	5.07	1.018
13	3.5	4.514	4.42	1.021
14	3	3.869	3.78	1.024
15	2.5	3.224	3.13	1.030
16	2	2.579	2.48	1.040
17	1.5	1.934	1.84	1.051
18	1	1.289	1.2	1.074
19	0.5	0.644	0.54	1.193
20	0	0	0	0.000

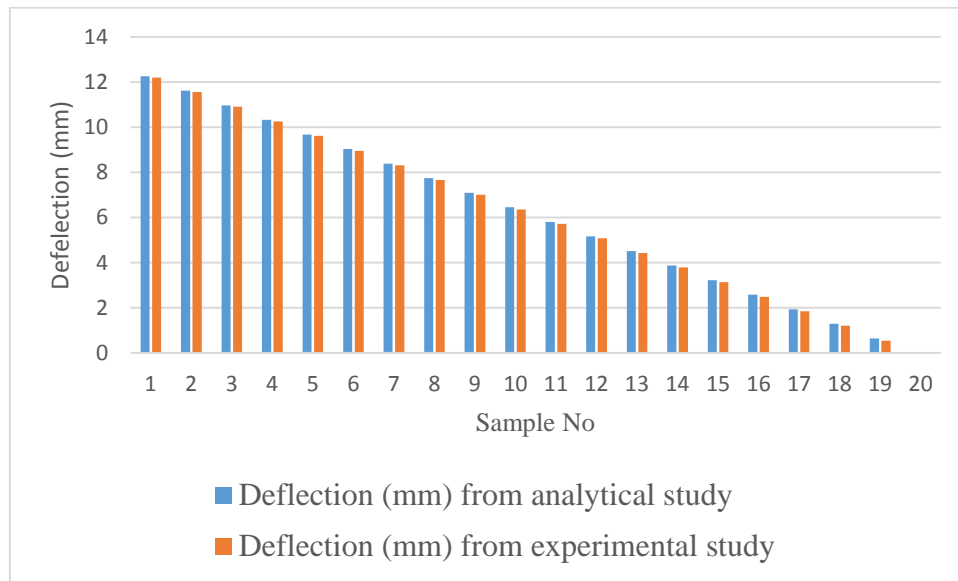


Figure A-3: Graphical Comparison of Analytical and Experimental Result

Therefore, an infill wall can be modeled as an equivalent diagonal compression strut in the building model.

APPENDIX B

MODELING OF EQUIVALENT STRUT

This illustration details the procedure for estimating the equivalent width of masonry wall for modeling in a structural analysis program (ETABS) linear modal analysis.

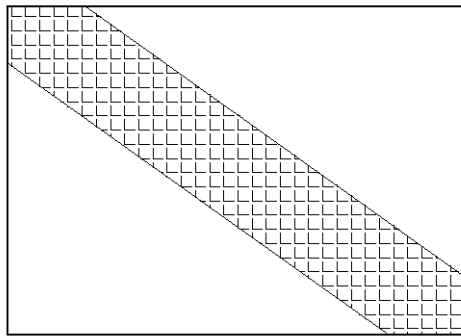


Figure B-1: Interpretation of an Equivalent Diagonal Compressed Strut

The first step is to model the bare frame according to its proper dimensions and physical properties as described in chapter Three. The frame modeled according to standard modeling procedures for RC frames. After modeling the bare frame, the equivalent eccentric diagonal struts are added to represent the masonry infill. Since most of the panels are fully infilled the struts should at first be designed to represent full infill panels and then multiplied by a proper reduction factor to account for any openings in the infill panel.

The equivalent strut width is evaluated by Pauley and Priestley (1992) and Pauley and Priestley suggested that the effective width shall be one-fourth the diagonal length which relates the width w of infill to parameter d (length of diagonal strut) and given by Equation B₁.

$$W = \frac{d}{4} \quad (B_1)$$

The thickness of strut assumed to be equal to wall thickness and the reduction factor for opening evaluated by Mondal and Jain (2008).

$$\rho_{MJ} = 1 - 2.6 \frac{\alpha a}{100} \quad (B_2)$$

Where: - ρ_{MJ} - Strut width reduction factor

αa - Opening Area Ratio

$$\alpha_a = \frac{\text{Area of opening}}{\text{Area of infill panel}} * 100$$

Table B-1: Strut Width Reduction Factor

Opening	Area of Opening (m ²)	Area of infill panel	width reduction factor
Door	2	15	0.65
Window	2.4	15	0.584

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