ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

INVESTIGATION OF THE EFFECT OF SHEAR WALL CURTAILMENT FOR MEDIUM AND HIGH-RISE BUILDINGS IN SEISMIC REGION

A Thesis Submitted to Addis Ababa University School of Civil and Environmental Engineering in Partial Fulfillment of the Requirements for the Degree of Master of Science (Structural Engineering)

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DECLARATION

I, the undersigned, declare that this thesis work, to the best of my knowledge and belief, is my original work, has not been presented for a degree in this or any other universities, and all sources of materials used for the thesis work have been fully acknowledged.

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ABSTRACT

In this thesis work, the effect of shear wall curtailment for medium and high-rise buildings in seismic region is investigated. When the shear wall frame system is loaded laterally in medium and high-rise buildings, the upper part of the shear wall takes a negative role in resisting the lateral loads because of the difference in the free deflected forms of shear walls and moment resisting frame. The discontinuity of shear walls may prove an effective technique in reducing this negative effect at the top. Consequently, in order to have good wall-frame interaction, a shear wall should be curtailed above a point on which the shear wall is not effective in reducing the seismic responses of shear walls.

Analysis is done for six G+10, seven G+20, and ten G+30 models using finite element method (FEM) of response spectrum analysis with respect to the seismic response parameters such as top story displacement, story drift, story shear, base moment and fundamental time period. Moreover, the Finite element analysis (FEA) is tried to be supplemented by three continuum methods (algebraic solutions) which includes the Component Stiffness Method-Equation C, Alex Coull's method and Marie-José Nollet's method.

In the FEA and continuum analysis results, the maximum top story (roof) displacement where the shear wall is curtailed above 8th, 14th and 22nd floor for the G+10, G+20 and G+30 building models respectively do not significantly vary with respect to that of the full height shear wall-frame models. Moreover, the other parameters (i.e., the story drift, story shear, story overturning moment, story stiffness and fundamental time period) also reveal that curtailment of shear wall above 70% - 80% of the height of the building will not much affect the performance of the building in resisting seismic loads.

Therefore, even if ES-EN 1998: 2015 recommend shear walls to continue up to the roof level for the sake of regularity (i.e., the analysis will be simplified and the structure response will also be easily predictable), curtailment of the shear walls in the top floors is not necessarily detrimental to the performance of the structure. When building height increases, the stiffer the frames will be, the greater the interaction become between wall and frame and the lesser the contribution of the shear wall. So, if we curtail the shear wall for the increased building height (high rise), since the shear wall contribution is minimized, the curtailment become safe.

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A	cross-sectional area of a column
B	total width of frame
C	distance of the centroid of a column section from the common center of area of all the columns of the frame
D	beam depth
E	Young's modulus
Δ_B	deflection at top of frame due to bending of members
W	total distributed lateral load
h	story height
H	total height
G	shear modulus
$y_c(H)$	top deflection of a curtailed wall-frame
$y_1(H_1)$	top deflection of sub-structure 1
$y_1'(H_1)$	the slope at the top of the wall,
$y_2(H_2)$	the top deflection of the upper rigid frame of substructure 2
ϕH_1	$\text{top slope of sub} - \text{structure 1}$
I	total flexural inertia
I_g	gross flexural inertia
I_c	the inertias of the columns
I_w	the inertias of the walls
K^2	non-dimensional characteristic parameter for axial deformations of the columns of the frame.
K_B	rotational stiffness of shear wall support
K_f	point load at top of frame to cause unit deflection in its line of action
P	interaction force at top
K_w	total point load applied at the top of a shear wall to cause unit deflection in its line of action; also, specifically the stiffness of a shear wall without openings
γ_w	dimensionless parameter which relates the rotational stiffness of the wall to that of the foundation
α^2	ratio of the frame shear rigidity to the wall flexural rigidity
F_S, F_g	functions of s and g, dependent on the type of loading

αH	non-dimensional characteristic parameter for the relative stiffness
K_f	point load at top of frame to cause unit deflection in its line of action
Δ_A	deflection at top of frame due to axial deformation of exterior columns
Δ_B	deflection at top of frame due to bending of members
β_D, β_c	constants dependent on geometry of the structure
s, g	constants dependent on moment of inertia of column and beam

CHAPTER 1: INTRODUCTION

1.1 Background of the Study

As the population is increasing tremendously with every passing year and the land available for use of habitation is the same as it was a decade ago. There has been an increase in the density of population in the urban area, since population from rural areas is migrating in large numbers to metro cities. Due to this population growth and other driving forces like the need for luxury living, profit for developers and scarcity of building lands, metro cities are getting densely populated day by day. So, the only solution to the problem is for vertical growth. Now this need for the increase in the vertical height of the buildings has made the building become tall and slender.

From the structural viewpoint, tall building means a multi-story construction in which the effects of horizontal actions and the need to limit the relative displacements take on primary importance (Taranath 1988, 2005). While in the design of low-rise structures the strength requirement is the dominant factor, with increasing height, the importance of the rigidity and stability requirements to be met to counter wind and earthquake actions grows until they become the prevailing design factors. Thus, height of the building has now become the primary point of focus of today's world. Since buildings are getting taller and slender the primary concern of design engineers is shifting from gravity loads to lateral loads. The effect of lateral forces becomes more and more dominant as the building becomes taller and taller. These lateral forces can produce critical stresses in the structure, induce undesirable stresses and vibrations or cause excessive lateral sway of the structure. Even if there is no hard and fast rule when it comes to classifying buildings as low-rise, mid-rise or high-rise, it is generally, well known that when building height increases lateral load like earthquake load effect increases.

Earthquake resistance can be achieved through a wide range of structural systems. Structural systems, also called as lateral load resisting systems, are classified into Frame Systems, Structural Wall Systems, Tubular Systems and soon or in combination of two or more structural systems as a Dual Systems (like Frame-shear wall dual systems) (Fazlur Khan, 1969). To achieve satisfactory seismic performance, vertical components of lateral resisting systems should comply with the structural requirements, such as plan and elevation regularities. Seismic behavior depends on materials of construction, system configurations and failure modes. The one that will be considered in this paper is dual system.

In the Dual System, both frames and shear walls contribute in resisting horizontal forces. The reinforced concrete shear wall is an important structural element placed in multi-story buildings which is situated in seismic zones because they have a high resistance to lateral loads such as wind or earthquake loads (Smith and Coull, 1991). Thus, shear walls are introduced into modern tall buildings in addition to the frames to make the structural system more efficient in resisting the horizontal and gravity loads, ground motions as well thereby causing less damage to the structure during earthquake.

While using frame with shear wall, frame bends in accordance with shear mode, whereas the deflection of the shear walls is by a bending mode like the cantilever walls. As a result of the difference in deflection properties between frames and walls, the frames will try to pull the shear walls in the top of the building and try to push the walls at bottom. So, the frames will resist the lateral loads in the upper part of the building, which means an increase in the dimensions of the cross-section area of the columns in the upper part of frame more than what it needs to resist the gravity loads, while the shear walls will resist most of the vertical loads in the lower part of the building.

There are different research papers done on the assessment of effect of shear wall curtailment on buildings with varying height in seismic region like effect of Curtailment of Shear Walls for Medium Rise Structures by Bhatt and Titiksh in 2007, which was trying to investigate the effect of the height of shear wall on the response of dual systems, study of lateral load resisting systems of variable heights in all soil types of high seismic zone by Baikerikar and Kanagali in 2014, but this research paper makes different from others first by its trial to compare and investigate the effect of curtailment of the shear walls using that of continuum mechanics with that of finite element analysis results and next the paper also tries to study the effect of height of structure on wall-frame horizontal interaction.

1.2 Objective

1.2.1 General objectives

To prove that curtailment of the shear walls in the top floors is not necessarily detrimental to the performance of the structure using that of continuous mechanics and finite element analysis methods and to study the effect of height of structure on wall-frame horizontal interaction.

1.2.2. Specific Objectives

1. Calculating the top story displacement of the dual system for different height shear wall curtailment of the medium and high-rise buildings (G+10, G+20 and G+30) using the continuum model and also using FEM.
2. Determine the top (roof) story displacement, story drift, story overturning moment, story stiffness, story shear and fundamental period for varying height of shear wall using Response Spectrum Method and see the effect of curtailment.
3. Compare the results obtained from the continuum model and the FEA model.

1.3 Statement of the problem

It is advantageous to use dual systems as a lateral load resisting system in the design of medium and high-rise structures. Considering both the frame and the shear wall contribute in lateral load resistance to be rational and economical, when the shear wall frame system is loaded laterally in medium and high-rise buildings, the free deflected forms of the wall and frame cause them to interact horizontally through the floor slabs (Coull and Smith, 1967). The upper part of the shear walls (above contraflexure point) actually takes a reverse role in resisting the lateral loads because of the difference in the free deflected forms of shear walls and moment resisting frames. The discontinuity of shear walls may prove to be an effective technique in reducing this negative effect at the top. Consequently, in order to have good wall-frame interaction, a shear wall should be curtailed above a point on which the shear wall is not effective in reducing the seismic responses of shear walls.

So, this paper tries to prove that even if ES-EN 1998: 2015 recommend shear walls to continue up to the roof level for the sake of regularity (i.e., the analysis will be simplified and the structure response will also be easily predictable), curtailment of the shear walls in the top floors is not necessarily detrimental to the performance of the structure.

1.4 Significance of the study

Checking if both the continuum mechanics and the FEA prove that even if the Ethiopian building standard (ES-EN 1998: 2015) recommend shear walls to continue up to the roof level for the sake of regularity, curtailment of the shear walls in the top floors is not necessarily detrimental to the performance of the structure, in addition, it will be safe to curtain the shear wall instead of providing full height shear wall having equal level with that of building height.

1.5 Scope and Limitation of the Research

The scope of this study is to investigate the effect of shear wall curtailment for medium and high-rise buildings and compare the results of the FEA using response spectrum analysis with that of the continuum mechanics. The modeling and analysis will be carried out with the help of ETABS 19.0, finite element software package. Due to the unavailability of testing machines (cyclic loading testing machine) and economic constraints, experimental investigation required for this particular study is not covered in this thesis and no design work was also involved for all shear walls. This study will be conducted on six G+10, seven G+20 and ten G+30 sub-models with varying shear wall height that are not subjected to torsion and are mainly plan-symmetric structures subjected to symmetric loading, therefore, that can be analyzed as equivalent planar models. Structures that are asymmetric about the axis of loading inevitably subjected to torsion. Although the benefits from horizontal interaction between the walls and frames apply also to twisting structures, their consideration in a general way is extremely complex because the amount of interaction is highly dependent on the relative plan location of the bents.

1.6 Thesis Organization

This thesis is organized into six main chapters. The introductory chapter presents the background of the study, objectives of the study, statement of the problem, significance of the thesis and scope and limitation of the thesis.

Chapter 2: deals with the review of literatures written by previous researchers and theoretical discussions related to the area of interest.

Chapter 3: deals with the modeling and analysis of the different models with varying height of shear wall together with the finite element analysis (i.e., the response spectrum analysis) and the analytic solutions. Moreover, the materials used to conduct the research, such as the input data in ETBAS 19.0 for response spectrum analysis of all shear wall models are presented.

Chapter 4: presents the results and discussions of the study using tables and graphs. The results of all shear wall models are compared with respect to the seismic response parameters (displacement, story drift, story shear, overturning moment and more).

Chapter 5: presents the conclusions drawn based on the results obtained and recommendations and suggestions for future studies. Finally, reference materials used during the study are listed.

CHAPTER 2: LITERATURE REVIEW

2.1 General

In the past years, structural members were assumed to carry primarily the gravity loads. However, by the advancement in structural system has made buildings taller and slender. The effect of lateral forces due to wind and earthquake becomes more and more dominant as the building becomes taller and taller. By definition a high-rise (tall) building is a structure which, because of its height, is affected by lateral forces due to wind or earthquake actions to an extent that they play an important role of their structural design, (Smith and Coull, 1991). There are many structural systems that can be commonly used for lateral resistance of tall buildings like rigid frame structure, braced frame structure, shear wall frame structure, braced frame and shear wall frame structure, outrigger structure, frame tube structure, braced tube structure, bundled tube structure, trussed tube and diagrid system. Of the various types of lateral load resisting systems, the most common is the shear wall-frame structural system, comprising sets of shear walls and moment resisting frames.

A structure whose resistance to horizontal loading is provided by a combination of shear walls and rigid frames or, in the case of a steel structure, by braced bents and rigid frames, may be categorized as a wall-frame. Wall-frame Structures are widely used to resist lateral loads in tall buildings up to 60 stories (Smith and Coull, 1991). When a wall-frame structure is loaded laterally in high-rise buildings, the walls and frames interact horizontally to share in carrying the lateral loading, while both carry their share of gravity loading. When walls and frames arranged in parallel planes, the different free deflected forms of the walls and the frames cause them to interact horizontally through the floor slabs or in the same plane interact horizontally through connecting beams. In both cases, the horizontal interaction results from the change in relative stiffnesses between the walls and frames over the height, the walls restraining the frame from deflecting in the lower region and the frames restraining the walls in the upper region. Consequently, the individual distributions of lateral loading on the wall and the frame may be very different from the distribution of the external loading.

Moment resistant frame consists of columns and beams. The lateral stiffness of a moment resisting frame depends on the bending stiffness of the columns and beams. The advantage of moment resisting frame is that it is open rectangular arrangement which allows freedom of planning and easy fitting of doors and windows. It is economical only for buildings up to about 25 stories. Above 25 stories the relatively high flexibility of the frame calls for uneconomically large members in order to control the drift and displacements (Coull and Smith, 1967).

A shear-wall is an efficient method of ensuring the lateral stability of tall buildings and also efficient against torsional effects when combined together with frame structures. Deflections due to lateral forces are relatively small unless the height-to-width ratio of the wall becomes large enough to cause overturning problems. This would generally occur when there are excessive openings in the shear walls or when the height-to-width ratio of wall is in excess of five or so (Taranath, 1998). Shear walls are generally characterized by high in-plane stiffness and strength which makes them ideally suited for bracing tall buildings. (Taranath, 1998) states that structural forms of shear wall are commonly used in buildings up to 40 stories. Buildings engineered with structural walls are almost always stiffer than framed structures, reducing the possibility of excessive deformations and hence damage.

2.2 Shear Wall-Frame Interaction

A wall-frame high-rise structure typically consists of the walls and frames that involve from the architectural plan of the building (Smith and Coull, 1991). The benefit of accounting for the wall-frame interaction in the lateral load analysis, as opposed to assuming that the walls carry all the lateral loading, is that it recognizes the increased lateral stiffness due to the interaction, and also the wall and frame members to be designed more correctly and economically.

In these systems, reinforced concrete frames interacting with shear walls together provide the necessary resistance to lateral forces, while each system carries its appropriate share of the gravity load (Taranath, 2010). Because the deflected shape of a laterally loaded wall is quite different from that of a frame, the wall responds as a propped cantilever as shown in Fig. 2.1.

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 (Coull and Smith, 1967). 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, Fig. 2.1.

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, Fig. 2.1. The wall restrains or holds back the frame in the lower stories, 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 stiffnesses. The degree of horizontal interaction is governed by the relative stiffnesses of the walls and frames, and the height of the structure.

Considering the horizontal stiffnesses at the tops of a 10-story elevator core and a moment-resisting frame of the same height, the core say four times stiffer than the framing. If the same core and framing were extended to a height of twenty stories, the frame would then be exactly as stiff as the core. At forty stories frame would be four times as stiff as the core. This change in relative stiffness with height occurs because the top flexibility of the core, which behaves as a flexural cantilever, is proportional to the third power of its height, whereas the flexibility of the frame, which behaves as shear cantilever, is directly proportional to its height. Consequently, 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.

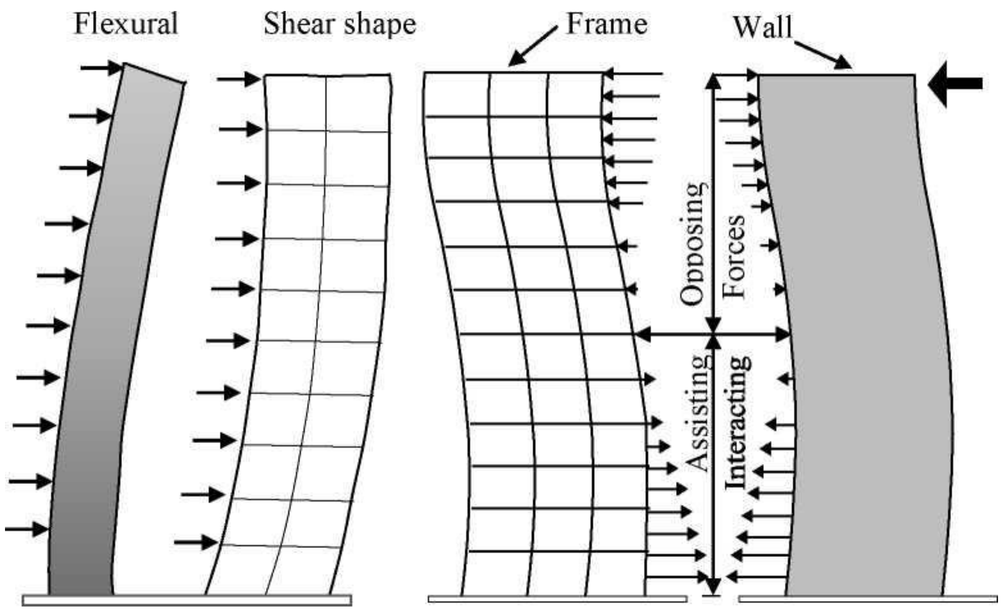


Fig. 2.1: Frame-Shear wall interaction

The potential advantages of a wall-frame structure depend on the amount of horizontal interaction, which is governed by the relative stiffnesses of the walls and frames, and the height of the structure (Coull and Smith, 1967). The taller the building and, in typically proportioned structures, the stiffer the frames, the greater the interaction. It used to be common practice in the design of high-rise structures to assume that the shear walls or cores resisted all the lateral loading, and to design the frames for gravity loading only. Although this assumption would have incurred little error for buildings of less than 20 stories with flexible frames, it is possible that in many cases where the frames were stiff and the buildings taller, opportunities were missed to design more rational and economical structures.

The principal advantages of accounting for the horizontal interaction in designing a wall-frame structure are as follows:

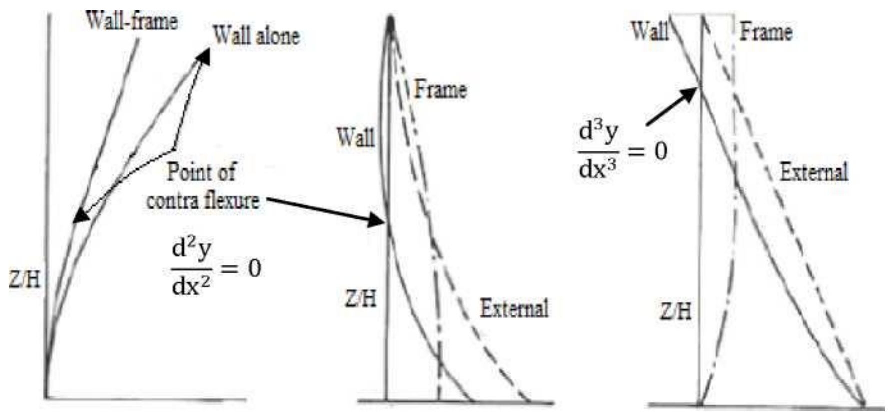
- ✓ The estimated drift may be significantly less than if the walls alone were considered to resist the horizontal loading.
- ✓ The estimated bending moments in the walls or cores will be less than if they were considered to act alone.
- ✓ The columns of the frames may be designed as fully braced.
- ✓ The estimated shear in the frames, in many cases, may be approximately uniform through the height: consequently.' the floor framing may be designed and constructed on a repetitive basis, with obvious economy.

2.3 Uniform Wall-Frame Structures

In the dual system, both frames and shear walls contribute in resisting the lateral loads. The frame is a group of beams and columns connected with each other by rigid joints and the free deflected forms of the wall and frame cause them to interact horizontally through the floor slabs. A solution for such a uniform wall-frame structure has been developed using an equivalent continuous medium or "continuum model" (Heidebrecht and Smith, 1973). The frames bend in accordance with shear mode, whereas the deflection of the shear walls is by a bending mode like the cantilever walls. As a result of the difference in deflection properties between frames and walls, the frames will try to pull the shear walls in the top of the building, while in the bottom, they will try to push the walls. The upper part of the shear walls actually takes a negative role in resisting the lateral loads because of the difference in the free deflected forms of shear walls and moment resisting frames. So, the frames will resist the lateral loads in the upper part of the building, which means an increase in the dimensions of the cross-section area of the columns in the upper part of the frame more than what it needs to resist the gravity loads, while the shear walls will resist most of the vertical loads in the lower part of the building.

Referring to the distribution of bending moment for the full-height-wall structure [Fig. 2.2 (b)]; the wall moment in the region above the point of inflection, where $d^2y/dx^2 = 0$, is opposite in sense to the external load moment, while the moment in the frame (which is carried mainly by axial forces in the columns) is actually greater than the external load moment. Therefore, if the wall were curtailed anywhere in the region above the point of inflection, the moment carried by the frame would be reduced to become equal to the external moment (Nollet and Smith, 1993).

Similarly, for the distribution of the shear force in the full-height-wall structure [Fig. 2.2 (c)], the shear in the wall above the point of zero shear, where $d^3y/dx^3 = 0$, is opposite in sense to the external load shear, while the shear in the frame exceeds the external shear. Therefore, if the wall were curtailed anywhere in that uppermost region, the shear in the frame would be reduced to become equal to the external load shear (Nollet and Smith, 1993).



Horizontal deflection　　Bending moment　　Shear force

(a)　　(b)　　(c)

Fig.2.2 Typical deflection diagram, moment diagrams and shear diagrams for components of laterally loaded wall-frame structure

2.4 Wall-Frame Structures with Curtailed Wall

2.4.1 General

It is common in the design of practical wall-frame structures to reduce the size, or omit completely, the shear walls and cores in the upper parts of the building, where fewer elevator shafts are required (Smith and Coull, 1991).

The question of how this curtailment will affect the stiffness of the building can be answered by considering the behavior of the wall-frame structure with full-height shear walls or cores.

The “full-height” wall-frame structures shows that in the lower region of the structure the wall and frame both contribute to resisting the external moment and shear. In the upper region above the point of inflection, where moment is zero, however, the moment in the wall is reversed to be of the same sense as the external moment. Further, in the uppermost region above the level where shear is zero, the shear in the wall is also reversed, and so the shear in the frame exceeds the external shear. Consequently, if the wall is reduced or eliminated above the point of contra flexure, the moment on the upper part of the frame is reduced, and if the wall is reduced or eliminated above the level where the shear is zero, both the moment and the shear in the frame are reduced. In both cases the reduction or curtailment may reduce the lateral displacement at the top.

Intuitively, it might be thought that curtailing the shear walls would cause the building to drift more and to have larger internal forces than the corresponding, more substantial, wall-frame structure with full-height walls. In fact, this is not necessarily the case. In some circumstances the curtailed structure drifts less and has comparable or even smaller internal forces than the corresponding full-height-wall structure.

The discontinuity of shear walls may prove an effective technique in reducing this negative effect at the top. In the words of B.M. Atik, "The optimum level of curtailment always lies between the point of inflection and zero wall shear in the corresponding full height wall structures". Above the point of inflection i.e., where the bending moment curve changes direction, the wall moment is opposite to the external load moment, while the moment in the frame is actually greater than the moments due to external loads. Thus, if we curtail the shear walls above this point of inflection, the moments in the frame will become equal to the moments due to the external loads. A similar trend is observed in the case of shear forces. Hence, if the walls are curtailed above 60% of the height of the building, both the bending moment and the shear in the frames will become equal to the bending moment and the shears due to the external loads. (Atik et al, 2011) studied this effect using continuum analysis and proposed a corrected curve for the prediction of shear wall curtailment, originally proposed by Nollet and Smith. (Nollet and Smith, 1993) proposed that the curtailment of shear walls in a dual system is not necessarily detrimental to the performance of the structure as far as its lateral load carrying capacity is considered. If the curtailment is done at any level above the point of contraflexure in the wall, the changes in the maximum roof displacement is minimum.

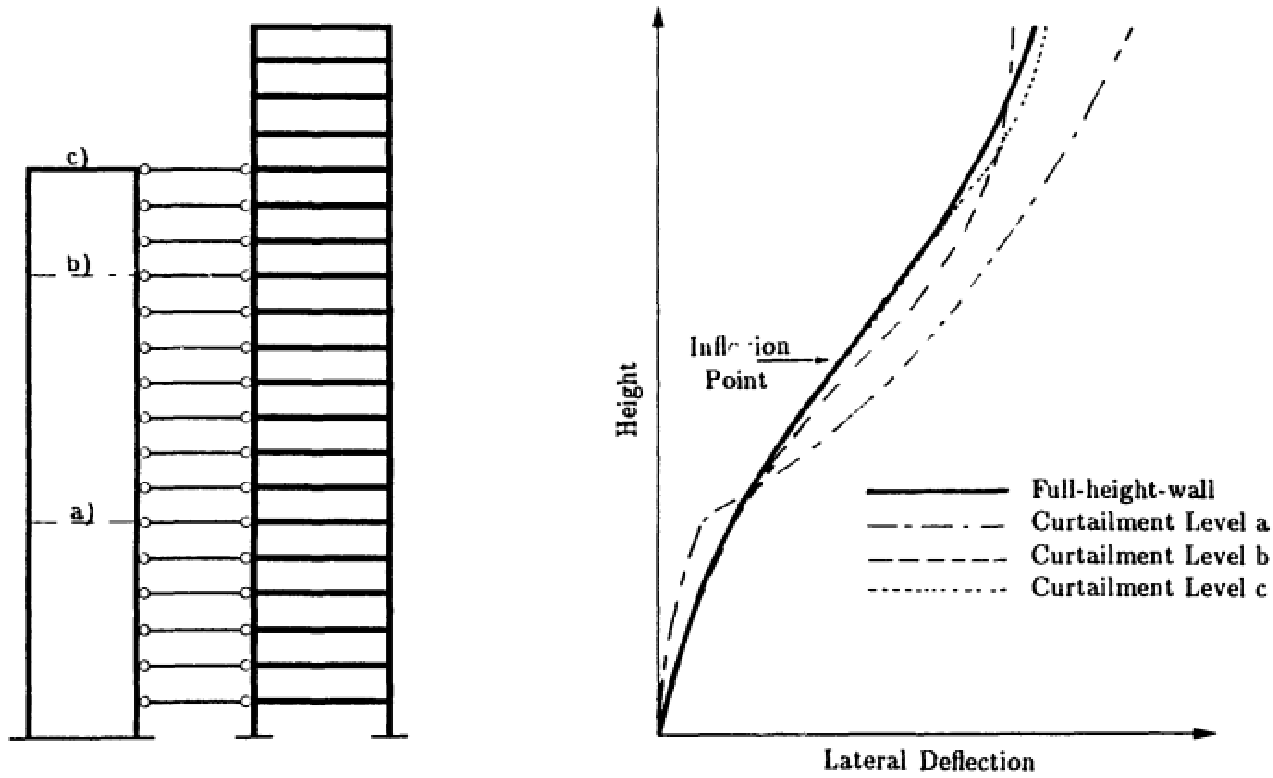


Fig. 2.3: General effect of curtailment on the deflection

The deflected shape of a wall-frame structure shown in fig. 2.3 has two regions: lower region in which the flexural mode is dominant, and an upper region in which shear mode is dominant (Nollet and Smith, 1993). Therefore, curtailing the wall in the lower region, where it is effectively stiffer and serves to restrain the frame from deflecting, releases the frame and allows it to deflect more (fig. 2.3, curtailment level a). Conversely, curtailing the wall in the upper region, where the frame is effectively stiffer, does not significantly modify the top deflection, but allows the frame to become almost vertical at the top, which is typical of a separate, uniformly loaded shear cantilever (fig 2.3).

In order to further investigate the effects of curtailment on the efficiency of dual system, (Atik et al, 2011) prepared a mathematical model for the solution of a continuum model of curtailed shear wall. They proposed that- "The level of curtailment which leads to delete the negative shear is the same that leads to delete the negative momentum." This means the discontinuity of shear walls at this particular level will eliminate the reverse force applied by the walls on the frame, leading to minimum top deflection. The studies done by (Atik et al, 2011) supported the finding of (Nollet & Smith, 1993) leading us to confirm that the optimum level of shear wall curtailment indeed lies in between point of inflection and point of zero wall shear in the corresponding full height structure.

Then Nollet and Smith developed a generalized theory for the deflection of wall-frame buildings on the basis of a continuum model. Their model has been used to analyze the effect of the wall curtailment on the performance of the structure. The deflection at the top of the structure is minimized to provide guidance for the optimum level of wall curtailment. They found that the optimum level is generally situated between the points of inflection and zero shear in the corresponding full-height wall structure. In contrast, some results of their study showed that, in spite of existing negative moments and shear forces in the shear wall, there is no need to curtail the wall. Such a result seems inconsistent and requires a thorough review of the calculation. This will be carried out in the present analysis.

The effects of curtailment on the top deflection of the structure may be examined by considering the effects that curtailment causes to the distributions of shear and moment on the frame, and the consequent effect of these actions on its top deflection. Therefore, a minimum change in the frame force distribution after curtailment of the wall will directly lead to a minimum change or increase in the top deflection. A study of the force distribution should help towards an understanding of the effects of curtailment on the behavior of the wall-frame structure (Nollet and Smith, 1993).

2.4.2 Shear Force Distribution

Considering a typical shear force distribution in a uniform wall-frame structure subjected to a uniformly distributed lateral load of intensity *w*, Fig. 2.4, it can be divided in two regions separated by the point of zero shear in the wall, where $d^3y/dx^3 = 0$. In the upper region the shear in the wall is opposite in sense to the external shear, therefore the shear in the frame in that region must exceed the external shear (Nollet and Smith, 1993).

If the wall is curtailed in the described upper region, the resulting shear in the top part of the frame will be reduced, to equal the external shear. In the lower part of the structure, the curtailed wall and the rigid frame will share the external shear. If the wall is curtailed below the point of zero shear in the wall of the reference uniform wall-frame structure, the shear in the frame in the upper region will equal the external shear and therefore, will increase locally above the level of curtailment while generally decreasing in the upper region (Nollet and Smith, 1993).

2.4.3 Moment Distribution

Considering the typical distribution of moment between the wall and the frame of a uniform full-height-wall structure subjected to a uniformly distributed load of intensity w, Fig. 2.4, it also can be divided in two regions, separated in this case by the point of zero moment in the wall, i.e., the point of inflexion, where $d^2y/dx^2 = 0$. In the upper region the moment in the wall is opposite in sense to the external moment, while the moment in the frame (which is carried mainly by axial forces in the columns) is greater than the external load moment (Nollet and Smith, 1993).

If the wall is curtailed anywhere in the region above the point of inflexion, the moment in the upper region of the frame will be reduced, to equal the external load moment. If the wall is curtailed below the point of inflexion, the moment in the frame just above the curtailment level will be increased, to equal the external load moment, while generally decreasing in the upper region.

An inspection of Fig.2.4, shows that if the wall is curtailed between the point of zero shear and moment, the shear in the frame just above the curtailment level will increase by a small amount while the moment in the frame above that level will be reduced, resulting overall in an almost unmodified force distribution.

If the wall is curtailed below the point of inflexion, both the shear and the moment in the frame will increase (Nollet and Smith, 1993).

2.4.4 Interaction Force Distribution

A related consideration in curtailing the wall is that the concentrated interaction force, which acts between the wall and the frame at the top of the full-height wall-frame structure, Fig. 2.4, is transferred in a curtailed wall-frame structure to the level of curtailment. The total concentrated force at that level is caused partly by the uniformly distributed load between the base of the structure and the level of curtailment. In addition, the concentrated shear and moment at the curtailment level, which result from the lateral loading on the upper part of the structure, both make contributions to the concentrated interactive force between the top of the curtailed wall and the frame. Interactive force between the top of the curtailed wall and the frame (Nollet and Smith, 1993).

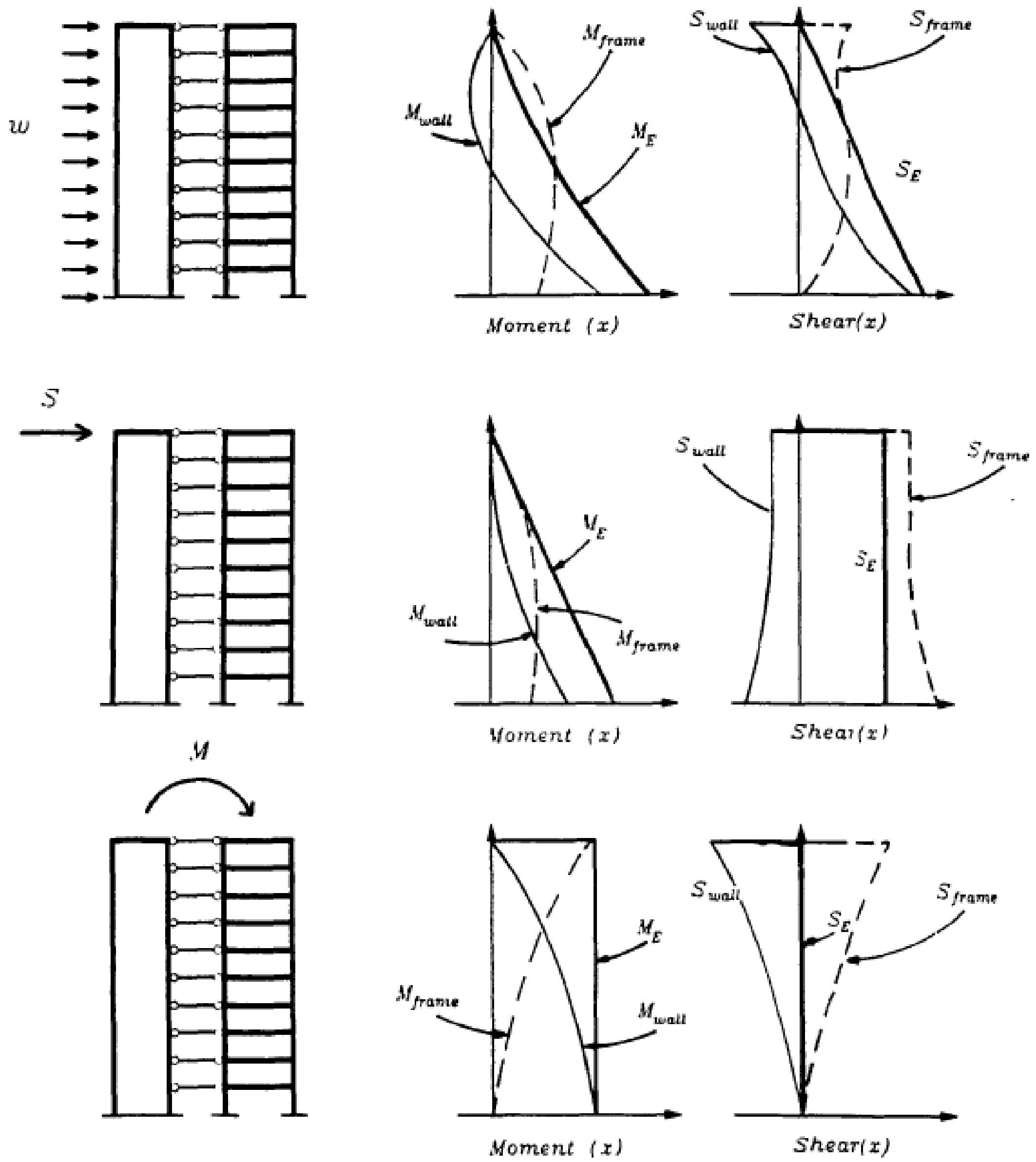


Fig. 2.4: Distribution of the external forces on the lower region of curtailed wall-frame structure

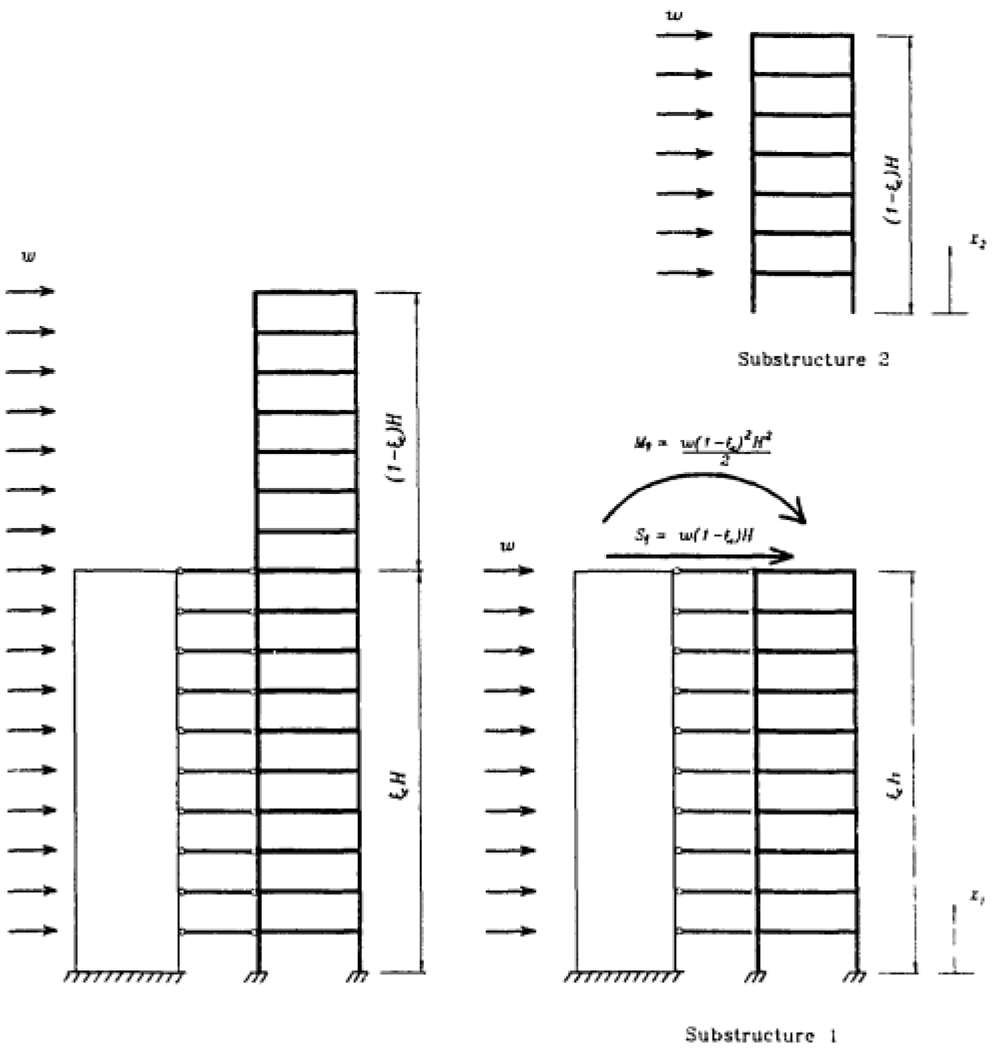
2.4.5 Algebraic solution for wall-frame structures with Curtailed Wall

To estimate the changes in top deflection caused by curtailment of the wall, compared with the top deflection of the corresponding full – height wall structure, an algebraic solution for the deflection of a continuum model of a curtailed wall-frame structure is discussed below.

In wall-frame structures that are plan-symmetrical about the axis of loading, and therefore do not twist, the high in-plane stiffness of the floor slabs causes the lateral deflections of the walls and frames to be effectively identical (Nollet and Smith, 1993). Consequently, the structure can be represented by a planar model. Under these conditions a mathematical solution for a continuous medium, or "continuum," model of the curtailed wall-frame structure has been developed by Nollet and Smith as follows: -

Modelling of Curtailed Wall – Frame structure

A curtailed wall-frame structure of total height H, subjected to a uniformly distributed horizontal load, w [Fig. 2.5(a)], can be considered as the superposition of two substructures [Fig. 2.5(b)]: Substructure 1 and Substruction 2.



(a) (b)

Fig 2.5 Model for Continuum Analysis: (a) Curtailed Wall-Frame Structure; and (b) Substructures of Curtailed Wall-Frame Structure

- The lower substructure, substructure 1, is a uniform wall – frame with a height H_1 = ξ_cH, subjected to the external uniformly distributed lateral load, w, a top concentrated shear force, S_1, and a top moment, M_1, where S_1 and M_1 are the accumulated shear and moment from the upper region acting on the lower region.

➢ The upper substructure, substructure 2, is a moment-resisting frame of height $H_2 = (1-\xi_c)$H with known shear rigidity and differential axial deformation properties, subjected to only the external uniformly distributed load w.

To obtain a good representation of the equivalent total structure, the lower region (substructure 1) should be subjected to:

(a) A uniformly distributed load of intensity w;

(b) A concentrated shear at the top $S_1, equal\ to$:

$$wH_2 = w(1-\xi_c)\mathrm{H} \tag{2.1a}$$

(c) A concentrated moment at the top M_1, equal to:

$$\frac{w\,H_2{}^2}{2} = \frac{w(1-\xi_c)^2H^2}{2} \tag{2.1b}$$

And substructure 2 is submitted to a uniformly distributed load w, while respecting the following conditions at its base:

(a) The lateral deflection at the base of substructure 2 is equal to the top deflection of substructure 1 ($y_1(H_1)$).

(b) The initial slope at the base of substructure 2 is equal to the slope at the top of the frame of substructure 1 $\phi \mathrm{H}_1$ due to axial deformation in the columns of Sstr. 1.

From these conditions the expression of the total deflection of the curtailed wall – frame structure, $\mathrm{y_c(H)}$, can be written as:

$$y_c(H) = y_1(H_1) + y_2(H_2) \tag{2.2}$$

Where $y_2(H_2)$ is the top deflection of the upper rigid frame of substructure 2 caused by the uniformly distributed load w, with initial slope $\phi H_1\ \ at\ its\ base$.

Both regions are analyzed as continuum structures with their properties spread over their heights.

Solution for sub-structure 1

The deflections $y_1(H_1)$ and $y_1{}'(H_1)$ at the top of substructure 1 are the summations of the deflections and slopes caused by the three different cases of loading: w, S_1 and M_1.

$$y_1(H_1) = y_1(H_1)_w + y_1(H_1)_{s_1} + y_1(H_1)_{M_1} \tag{2.3}$$

$$y_1{}'(H_1) = y_1{}'(H_1)_w + y_1{}'(H_1)_{s_1} + y_1{}'(H_1)_{M_1} \tag{2.4}$$

The equation for the top deflection and top slope of a uniform structure subjected to each of those loading cases are given in above equation. To relate the properties of substructure 1 to the properties of the corresponding uniform structure with a full- height wall of total height H, the following substitutions should be made.

$$H_1 = \xi_c H \tag{2.5}$$

$$S_1 = wH_2 = wH(1-\xi_c) \tag{2.6}$$

$$M_1 = \frac{w\,{H_2}^2}{2} = \frac{wH^2(1-\xi_c)^2}{2} \tag{2.7}$$

where: $EI_1 = EI$, ${k_1}^2 = k^2$, $(\text{k}\alpha\text{H})_1 = \xi_c(\text{k}\alpha\text{H})$

The resulting expressions for top deflection and top slope of substructure 1 are:

$$y_1(H_1) = \frac{wH^4}{EI}\frac{(K^2-1)}{K^2}\left[\frac{{\xi_c}^2}{4} - \frac{{\xi_c}^3}{6} + \frac{{\xi_c}^4}{24} - \frac{(1-{\xi_c}^2)}{2}\frac{(\cosh \text{k}\alpha H_1 - 1)}{(k\alpha H)^2 \cosh \text{k}\alpha H_1}\right] + \frac{wH^4}{EI}\frac{I}{K^2}\left[\frac{(2\xi_c - {\xi_c}^2)}{2(k\alpha H)^2} - \frac{tanhk\alpha H_1}{(k\alpha H)^3} + \frac{(coshk\alpha H_1 - 1)}{(k\alpha H)^4 coshk\alpha H_1}\right] \tag{2.8}$$

And,

$$y_1'(H_1) = \frac{wH^3}{EI}\frac{(K^2-1)}{K^2}\left[\frac{1}{2}\left(\xi_c - {\xi_c}^2 + \frac{{\xi_c}^3}{3}\right) - \frac{(1-\xi_c)^2}{2}\frac{\tanh \text{k}\alpha H_1}{k\alpha H}\right] + \frac{wH^3}{EI}\frac{I}{K^2}\left[\frac{(1-\xi_c)}{(k\alpha H)^2} - \frac{1}{(k\alpha H)^2 coshk\alpha H_1} + \frac{(\tanh \text{k}\alpha H_1)}{(k\alpha H)^3}\right] \tag{2.9}$$

It is important to appreciate that the slope at the top of the wall, $y_1'(H_1)$, is not the same as the slope at the base of the upper rigid frame. This is because the wall and the rigid frame are not constrained to rotate identically at each level.

Where:

$\alpha^2 =$ ratio of the frame shear rigidity to the wall flexural rigidity.

$$\alpha^2 = \frac{GA}{EI} \tag{2.10}$$

(GA) = the total racking shear rigidity of the set of frames. The racking shear rigidity of a single uniform frame i is given by (Smith et al. 1981):

$$(\text{GA})_i = \frac{12\,E}{h(\frac{1}{G}+\frac{1}{C})} = \frac{12\text{ E}}{\text{h}\left(\frac{1}{\Sigma\frac{\text{I}_b}{h}}+\frac{1}{\Sigma\frac{\text{I}_c}{l}}\right)_i} \tag{2.11}$$

In which:

E = the modulus of elasticity; h = the story height; $\sum I_c$= the sum of the inertias of the columns in a story of frame i; I_b = the inertia of a beam and l its span, $\sum I_b/l$ being summed for all the beams in a single floor of flame i.

$$EI = EI_c + EI_w \tag{2.12}$$

$$I = I_c + I_w \tag{2.13}$$

$$I_g = I_c + I_w + \sum A_C^2 \tag{2.14}$$

Where: I = total flexural inertia, I_g = gross flexural inertia, I_c= the inertias of the columns and I_w= the inertias of the walls

K^2 = non-dimensional characteristic parameter for axial deformations of the columns of the frame.

$$K^2 = \frac{EI + E\sum {A_C}^2}{E\sum {A_C}^2} \tag{2.15}$$

where, $\sum {A_C}^2$ = Second moment of area of the column sectional areas about their common center of area.

For structures in which: $\sum {A_C}^2$ comprises a dominant part of gross flexural inertia (I_g), or in which total flexural inertia (I) is small relative to I_g, as in rigid frames, braced frames, or wall-frames with several frames having a large structural width, K^2 is approximately unity.

When K^2 is different from unity, or total flexural inertia (I) is a significant proportion of gross flexural inertia (I_g), the flexural behavior becomes more significant, and the shear part of the total deflection is relatively less.

Solution for sub-structure 2

$$y_2(H_2) = \frac{wH^4}{EI}\left[\frac{(1-\xi_c)^2}{2(\alpha H)^2} + \frac{(k^2-1)}{8}\,(1-\xi_c)^4\right] + \phi H_1((1-\xi_c)H) \tag{2.16}$$

In which EI, α^2, and k^2 are properties of sub-structure 1 and

$$\phi H_1 = \frac{wH^3}{EI}\frac{(k^2-1)}{6}[\,(1-(1-\xi_c)^3)] - (k^2-1)\,y_1'(H_1) \tag{2.17}$$

Adding the lateral deflection at the top of substructure 1, and using the equation 2.16 and 2.17, the final expression for the top deflection of the curtailed wall $y_c(H)$ is as equation 2.2:

Additionally, a supplementary solution to get deflection at top of the structure for a uniform wall-frame structure has been developed by Heidebrecht and Smith as follows using continuum model:

$$y(H) = \frac{wH^4}{EI}\left\{\frac{(K^2-1)}{K^2} * \frac{1}{8} + \frac{1}{K^2}\left[\frac{1}{2(K\alpha\,\mathrm{H})^2} + \frac{(\cosh k\alpha\,\mathrm{H} - 1 - K\alpha\,\mathrm{H} * \sinh K\alpha\,\mathrm{H})}{(k\alpha H)^4 \cos\mathrm{h}\; k\alpha\,\mathrm{H}}\right]\right\} \tag{2.18}$$

2.5 Simplified Method of Estimating Lateral Deflection

If torsion is not considered, the simplified method of determining the interaction of frames and shear walls is to use Portland Cement Association's Advanced Engineering Bulletin No. 14 and to use the Component Stiffness Method-Equation C (Equation 2.19)

The component stiffness method has more flexibility than the other method referred above, but it lacks accuracy if the wall is more flexible than the frame. The main assumption is that the frame takes constant shear, i.e., that the interaction force between the frame and wall can be represented by a concentrated force at the top. This is a reasonable assumption for preliminary analysis, especially when the frame tends to be flexible in comparison with the wall. Generally, for uniformly distributed load condition equation C can be written as:

$$\frac{P}{W} = \frac{\frac{3}{8}(1+\frac{1}{\gamma_W})}{1+\frac{3}{4\gamma_W}+\frac{K_W}{K_f}} \tag{2.19}$$

Where: P = interaction force at top

W = total applied lateral load

γ_w = dimensionless parameter which relates the rotational stiffness of the wall to that of the foundation, i.e., ratio:

$$\gamma_w = \frac{K_B H}{4E_w I_w} \tag{2.20}$$

K_B = rotational stiffness of shear wall support

H = total height of wall

E_w = Young's modulus (subscript denotes structural system)

I_w = moment of inertia of wall

K_w = lateral point load applied at the top of a shear wall to cause unit deflection in its line of action; also specifically the stiffness of a shear wall without openings:

$$K_w = \frac{3EI_w}{H^3} \tag{2.21}$$

K_f = point load at top of frame to cause unit deflection in its line of action; i.e,

$$\frac{P}{\Delta} \; or \; \frac{P}{\Delta_B + \Delta_A} \; since\ top\ deflection\ \Delta = \frac{P}{K_f} \tag{2.22}$$

EQUATION A – for Axial deformation:

$$\Delta_A = \frac{WH^3F_n}{E_cA_CB^2} \quad (2.23)$$

Where: Δ_A= deflection at top of frame due to axial deformation of exterior columns

$F_n = function\ of\ n$, dependent on the type of loading

$$n = \text{ratio} \frac{Area\ of\ exterior\ column\ at\ top\ of\ frame}{Area\ of\ exterior\ column\ at\ bottom\ of\ frame} \quad (2.24)$$

(Linear variation of A_C with height)

For uniformly distributed load condition $F_n$can be determined by:

$$F_n = \frac{2-9n+18n^2-11n^3+6n^3log_en}{6(1-n)^4} \quad (2.25)$$

$A_C = area\ of\ exterior\ columns\ at\ first - story\ level$

B = total width of frame

EQUATION B – For Bending deformation:

$$\Delta_B = \frac{Wh^2H}{12\sum(E_CI_C)}\ [\ F_S(1-\ \beta_D)^3 + F_G\ (1-\ \beta_C)^3 2\lambda\ ] \quad (2.26)$$

Where: Δ_B = deflection at top of frame due to bending of members

W = total lateral load

h = story height

H = total height

E = Young’s modulus (subscript denotes structural system)

$\sum I_C$ = Sum of moments of inertia of columns at first - story level

F_S, F_g = functions of s and g, dependent on the type of loading

$$\left.\begin{array}{l} s = \text{ratio} \dfrac{I_C\ at\ top\ of\ frame}{I_C\ at\ bottom\ of\ frame} \\[2ex] g = \text{ratio} \dfrac{I_b\ at\ top\ of\ frame}{I_h\ at\ bottom\ of\ frame} \end{array}\right\}$$

Linear variation of $I_C\ and\ I_b$ with height. If E varies, use EI instead of I.

$$\boldsymbol{F_S} = \frac{1}{1-s} + \frac{slog_es}{(1-s)^2} \quad (2.27)$$

$$\boldsymbol{F_g} = \frac{1}{1-g} + \frac{mlog_eg}{(1-g)^2} \quad (2.28)$$

$$\beta_D = \frac{D}{h} \quad (2.29)$$

$where\ D\ is\ beam\ depth\ and\ h\ is\ height\ of\ column$

$$\beta_c = \frac{C}{\ell} \quad (2.30)$$

$where\ C\ is\ column\ width\ and\ \ell\ is\ distance\ between\ column\ center\ lines$

$$\lambda = \frac{\sum(E_C\ I_C/\ h\ )}{2\sum(E_b\ I_b/\ \ell\ )} \tag{2.31}$$

$i.e. summation\ over\ width\ of\ structure\ at\ first - story\ level$

I_b = moment of inertia of beam at bottom at bottom of structure.

Therefore, Total Deflection can be obtained by:

$$\Delta = \Delta_{\boldsymbol{B}} + \Delta_{\boldsymbol{A}} \tag{2.32}$$

Note:

For accuracy, the following conditions should be satisfied:

1. $\ell\ and\ I_b\ should\ not\ vary\ across\ the\ frame.$
2. $I_C\ should\ not\ vary\ across\ the\ frame, except\ that\ I_C\ for\ an\ interior\ column$ $should\ be\ twice\ that$ of an exterior column.
3. Columns should have points of contraflexure at mid height.
4. Story height should be constant.
5. $I_C, I_b\ and\ A_C\ should\ vary\ linearly\ with\ height.$
6. $\boldsymbol{\lambda} \leq \mathbf{5}$.
7. $\Delta_A\ should\ be\ small\ compared\ with\ \Delta_B.$

Reasonable results can be expected in many cases which satisfy the above conditions only approximately.

CHAPTER 3: MODELING AND ANALYSIS

3.1 General

This section mainly focuses on the methods and materials used to conduct this research. This study mainly includes theoretical discussions and computer modeling and analysis works in order to investigate the effect of shear wall curtailment for medium and high-rise buildings and compare the results of the continuum mechanics with that of the FEM using response spectrum analysis. Due to the unavailability of testing machines and economic constraints, experimental investigation required for this particular study was not covered in this thesis. The methodology for carrying out the research work will be by using finite element modeling of the different G+10, G+20 and G+30 buildings having varying height of shear wall as case study and the corresponding analysis results will be compared with the continuum mechanics (algebraic solution verse discrete solution) which again will be supplemented by reviewing available literatures. A three-dimensional finite element models are developed with ETABS 19.0.0, finite element software package. The results obtained from modal response spectrum analysis of all shear wall models were discussed with respect to the seismic response parameters such as lateral top story displacement, story drift, story stiffness, story shear, story overturning moment and fundamental period.

3.2 Preliminary Study

The research methodology was started with problem identification and setting up the objectives together with the scope of the study. After the problem was identified, a preliminary study (desk study) was done by studying several references which were also used to obtain some information that are related to the area of interest, such as literatures from the previous researchers. The literature review involved interpreting and synthesizing what has been researched and published in the area of interest.

This was done by reading on what other people have previously written about the area of interest. In this study, the main sources of literature review are from journals, articles, conference papers and books. Since the idea of curtailing shear walls below the roof story level deviate the structure from regularity and also the shear wall curtailment practice does not much prevail in our country, there are limited published documents. Therefore, all the articles are referred mainly from the Journals of Structural Engineering published by different society of civil engineers. This gives valuable information to support or refute the arguments stated and the findings of the study.

3.3 Structural Data

For carrying out the research work six G+10, seven G+20, and ten G+30 model buildings with varying shear wall height are considered. Later, the corresponding analysis results will be compared and cross checked with the finite element analysis which will be done using ETABS 19.0.0 software package. The following assumptions were made to maintain similar conditions for all the different models:

- ✓ Only the main block of the building is considered. The staircases are not considered in the analysis procedure.
- ✓ The building is resting directly on the ground. Also infill walls are not provided as the study focuses only on the response of frame and shear wall configuration.
- ✓ For the structural elements, Concrete grade of C30/37 has been used for the slabs, the wall, the beams and the columns & Steel grade of S-300 for reinforcement bar are used.
- ✓ The plan dimension is 25m x 25m.
- ✓ The height between each story is 3m.
- ✓ The shear wall thicknesses are 200mm for the G+10 and 400mm for G+20 and G+30 building models.
- ✓ The beam size is taken as 0.5m x 0.3m.
- ✓ The column sizes are 0.8m x 0.8m from ground- 5th floor and 0.5m x 0.5m from 6th to last for the G+10 model as shown in table 3.5
- ✓ For the G+20 and G+30 models the structural data are tabulated in table 3.7 and table 3.9 respectively.

3.4 Continuum Analysis

3.4.1 Analysis Using Alex Coull's Method

The steps followed in determining the top deflection using Alex Coull's Method is as follows:

- ✓ Determine parameter (GA)

a. Add the flexural rigidities El of all walls and cores to give the total (El).

In this case there is only a core and the core moment of inertia is calculated by:

$I = I_c + A_i d_i^2$ (3.1)

Where, I_c is centroidal moment of inertia, A_i is the area of the core (shear wall) and d_i is the transfer distance from centroid of the core to the part i.

But, as the structure is symmetric, considering either x or y direction, the moment of inertia can be obtained simply by

I_x or $I_y = 2*bh^3/12 = 2*(0.2m*(5m)^3)/12 = 4.17\ m^4$

Therefore, (El) = $33 * 10^9 * 4.17 = 13.76 * 10^7$ KNm^2

b. Evaluate the shear rigidities (GA) of the rigid frame wall – frame bents. Using equation 2.11, the shear rigidities (GA) of the frame can be obtained by using the expression.

$$(GA) = \frac{12\ E}{h(\frac{1}{G}+\frac{1}{C})},$$

Where $G = \sum \frac{I_b}{h} = \frac{(5*(0.3*0.5^3)}{5m*12} = 0.0031m^3$, since the beam dimension is 0.3m x 0.5m and,

$C = \sum \frac{I_c}{l} = \frac{(6*(0.5*0.5^3)}{3m*12} = 0.0104m^3$, since the top column dimension is 0.5mx0.5m

For frame: $(GA) = \frac{12*33*10^9}{3.0(\ 1/0.0031+\ 1/0.0104\ )} = 3.15 * 1\ 0^5$KN

- ✓ Use the values obtained in items a and b in the above step and evaluate α H , Using equation 2.10:

For the given structure, $\alpha\ H = H\sqrt{\frac{(GA)}{(El)}} = 30\ \sqrt{\frac{2.77*10^5}{12.1*10^7}} = 1.43$

An analysis for uniformly distributed loading is then made as follows:

- ✓ Determine Horizontal Displacement

The displacement at height z from the base is obtained by substituting α H and z/H in the equation 3.2 below.

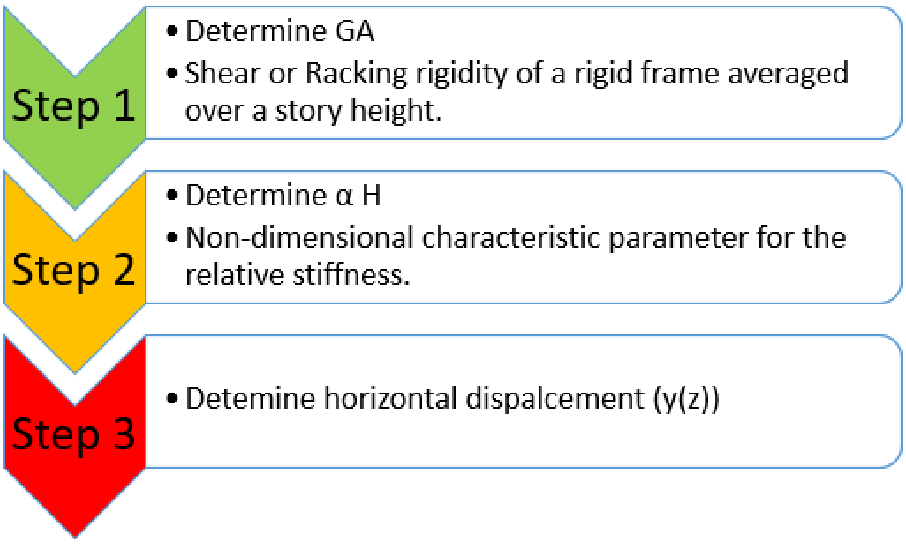
$$y(z) = \frac{wH^4}{El}\left\{\frac{1}{(\alpha H)^4}\left\{\frac{(\alpha H \sin h\,(\alpha H)+1)}{\cos h\,(\alpha H)}(\cos h\,(\alpha z) - 1) - \alpha H \sinh(\alpha z) + (\alpha H)^2\left[\frac{z}{H} - \frac{1}{2}\left[\frac{z}{H}\right]^2\right]\right\}\right\} \tag{3.2}$$

From table 3.3, the sum of the difference of story loads between each level are 623.84kN, then the earth quake loading per unit height becomes:

W = 623.84kN /30m = 20.79kN/m, where Total height of the building is 30m.

At the top …. Z/H = 1.0

then, submitting these values in the above equation, the top displacement and displacements at other levels is obtained as shown below,



$$(GA) = \frac{12\,E}{h\left(\frac{1}{G}+\frac{1}{C}\right)}$$

$$\alpha H = H\sqrt{\frac{(GA)}{(El)}}$$

$$y(z) = \frac{wH^4}{El}\left\{\frac{1}{(\alpha H)^4}\left\{\frac{(\alpha H \sin h\,(\alpha H)+1)}{\cos h\,(\alpha H)}(\cos h\,(\alpha z) - 1) - \alpha H \sinh(\alpha z) + (\alpha H)^2\left[\frac{z}{H} - \frac{1}{2}\left[\frac{z}{H}\right]^2\right]\right\}\right\}$$

Table 3.1 Horizontal Displacement at each Story level

Story Level	Z/H	GA(*10^5) (kN)	α H	α Z	Sin h (α Z)	Cos h (α Z)	y(z) (mm)
10	1	3.15	1.43	1.43	1.9696951	2.20900406	14.89
9	0.9	3.15	1.43	1.287	1.6729033	1.94900118	11.36
8	0.8	3.15	1.43	1.144	1.4103791	1.72892139	8.44
7	0.7	3.15	1.43	1.001	1.1767449	1.54425661	6.07
6	0.6	3.15	1.43	0.858	0.9672149	1.39122417	4.17
5	0.5	3.15	1.43	0.715	0.7774973	1.2666894	2.70
4	0.4	3.15	1.43	0.572	0.6037058	1.16810132	1.60
3	0.3	3.15	1.43	0.429	0.4422806	1.09344048	0.83
2	0.2	3.15	1.43	0.286	0.2899149	1.04117754	0.34
1	0.1	3.15	1.43	0.143	0.1434879	1.01024194	0.08
0	0	3.15	1.43	0	0	1	0.00

3.4.2 Analysis Using Marie-José Nollet's Method

In addition to the above analysis method, we can also determine the top deflection for full height wall-frame structures and also for wall-frame structures with curtailed walls at varying height using Marie-José Nollet's Method as follows.

Algebraic solution for wall-frame structures with Curtailed Wall

An algebraic solution for the deflection of a continuum model of a curtailed wall-frame structure to estimate the changes in top deflection caused by curtailment of the wall, compared with the top deflection of the corresponding full – height wall structure. From equation 2.2 the total deflection of the curtailed wall – frame structure, can be written as:

$$y_c(H) = y_1(H_1) + y_2(H_2)$$

Where: $y_c(H)$ *is* the total deflection of the curtailed wall – frame structure, $y_1(H_1)$ is the top deflection of Sub-structure 1, $y_2(H_2)$ is the top deflection of the upper rigid frame of Sub-structure 2 caused by the uniformly distributed load w, with initial slope ϕH_1 at its base.

The resulting expressions for top deflection $y_1(H_1)$ and top slope $y_1'(H_1)$ of Sub-structure 1 can be found using equation 2.8 and 2.16.

The value of the non-dimensional parameter k^2 that accounts column axial deformation is taken as 1 because when the value of αH is low, the influence of k^2 is negligible.

Table 3.2 Top deflection results of curtailed wall G+10 model

Curtaliment level (H_1)	ξ_c	k^2	$y_1(H_1)$ (mm)	$y_1'(H_1)$	ϕH_1	$y_2(H_2)$ (mm)	$y_c(H)$
10	1	1	15.01	0.1615	0.00067	-	15.01
9	0.9	1	14.44	0.1108	(0.00127)	(0.109)	14.21
8	0.8	1	16.87	0.0974	(0.00119)	0.211	16.96
7	0.7	1	18.20	0.0843	(0.00110)	0.971	19.17
6	0.6	1	18.48	0.0717	(0.00099)	2.181	20.91
5	0.5	1	17.73	0.0593	(0.00087)	3.852	22.22

The analysis results of all the parameters for the G+10, G+20, G+30 building models are presented on chapter 4.

3.4.3 Analysis Using Component Stiffness Method-Equation C

The procedure followed in determining the top deflection using equation c starts with calculating the story response values as shown in the table below:

Table 3.3 Story Response values

Story	Elevation	Total load	Story load	Ib (*10^9)	Ic (*10^9)
	M	kN	kN	mm^4	mm^4
Story10	30	701.82	0	3.125	5.21
Story9	27	701.82	77.98	3.125	5.21
Story8	24	623.84	77.98	3.125	5.21
Story7	21	545.86	77.98	3.125	5.21
Story6	18	467.88	77.98	3.125	5.21
Story5	15	389.9	77.98	3.125	34.1
Story4	12	311.92	77.98	3.125	34.1
Story3	9	233.94	77.98	3.125	34.1
Story2	6	155.96	77.98	3.125	34.1
Story1	3	77.98	77.98	3.125	34.1
Base	0	0	0	0	0

A. Distribution loads to the vertical units using equation C (equation 2.19)

- Since, foundation rotation of the shear not considered, the term with γ_w is ignored, so, equation 2.19 can be simplified to:

$$\frac{P}{W} = \frac{\frac{3}{8}}{1+\frac{\sum K_W}{\sum K_f}}$$

(3.3)

- Calculate K_f for a single frame, using equation B, (finite widths are ignored $\beta_C = \beta_D = 0$), so, equation 2.26 can be simplified to:

$$\Delta_B = \frac{Wh^2H}{12\sum(E_C I_C)} \; [\, F_S + F_G \; 2\lambda \,] \qquad (3.4)$$

Since $I_b = 3.125*10^9 mm^4$, $\sum I_C = 6*10.8*10^9 mm^4$, H = 30m, h = 3m and L= 5m from the structural data, using equation 2.31:

$$\lambda = \frac{\sum(E_C \; I_C/\,h\,)}{2\sum(E_b \; I_b/\,\ell\,)} = \frac{6*10.8*109mm4*5m}{3.125*109mm4*3m} = 109.12$$

$$S = \text{ratio} \; \frac{I_C \; at \; top \; of \; frame}{I_C \; at \; bottom \; of \; frame} = \frac{5.21*10^9 mm^4}{34.1*10^9 mm^4} = 0.152$$

N.B. If I_c at top of the frame is equal to I_c at bottom of the frame then the analysis will be undefined.

- Then using equation 2.27 and value of S computed above

$$\boldsymbol{F_S} = \frac{1}{1-s} + \frac{s\log_e s}{(1-s)^2} = \frac{1}{1-0.4824} + \frac{0.4824\log_e 0.4824}{(1-0.4824)^2} = 0.78$$

And thus Fs = 0.78

- g = ratio $\frac{I_{b\ at\ top\ of\ frame}}{I_h\ at\ bottom\ of\ frame} = 1$

So, if g = 1, and then using equation 2.28, $\boldsymbol{F_g} = \frac{1}{1-g} + \frac{m\log_e g}{(1-g)^2} = 1$

Therefore, using equation 2.22:

$$\frac{\Delta_B}{P} = \frac{(3m)^2 * 30m}{12 * 33 * \frac{10^9 N}{m^2} * 0.2046m^4}\,[\,0.78 + 2 * 109.12\,]$$

$$= 0.0007298\text{m/kN} = 0.7298\text{mm/kN}$$

- Estimating $\frac{\Delta}{P}$ to axial deformation of columns:

EQUATION A – for Axial deformation (referring equation 2.23):

$$\Delta_A = \frac{WH^3F_n}{E_cA_CB^2}$$

$$A_C = area\ of\ exterior\ columns\ at\ first - story\ level = 0.36m^2$$

n = ratio $\frac{Area\ of\ exterior\ column\ at\ top\ of\ frame}{Area\ of\ exterior\ column\ at\ bottom\ of\ frame}$

n = $\frac{50cm*50cm}{80cm*80cm} = 0.39$

$$\boldsymbol{F_n} = \frac{2 - 9n + 18n^2 - 11n^3 + 6n^3\log_e n}{6(1-n)^4} = 0.289$$

Therefore,

$$\frac{\Delta_A}{P} = \frac{(30m)^3 * 0.289}{33 * \frac{10^9 N}{m^2} * 0.64m^2 * (25m)^2}$$

= 0.000591mm/kN

Then $\frac{\Delta_A}{\Delta_B} = \frac{0.000591}{0.7298} = 0.00081 \simeq 0.08\%$

i.e., top deflection of a frame due to column axial deformation would be around 0.08% of that due to bending.

N.B Since column axial deformation contribution for top deflection is less and also neglected by (Goldberg, 1974), it is also neglected here and:

From equation 2.22, $K_f = \frac{P}{\Delta_B} = \frac{1}{0.7298} = 1.37kN/mm$

$\sum K_f$ = 6* 1.37kN/mm = 8.22 kN/mm

✓ Calculate K_w using equation 2.21

$K_w = \frac{3EI_w}{H^3} = \frac{3*33*10^9 N/m^2*2.08m^4}{(30m)^3} = 7.627kN/mm$

$k_w = \frac{P}{\Delta_B + \Delta_A}$… would be used if Δ_A (deflection due to axial deformation of columns) were considered, but now it is ignored.

✓ Calculate P/w

$$\frac{P}{W} = \frac{\frac{3}{8}}{1+\frac{\sum K_w}{\sum K_f}}$$, where, $$\frac{\sum K_w}{\sum K_f} = \frac{7.627kN/mm}{8.22kN/mm} = 0.927$$

Then, $\frac{P}{W} = \frac{\frac{3}{8}}{1+0.927} = 0.194$

✓ Calculate loads on units from table 3.3

W= 8*77.98+0 = 623.84kN

P = 0.194*623.84kN = 121.02kN

P to each frame = 121.02kN/6 = 20.17kN

P to the wall = 121.02kN

✓ Finally, Calculate top deflection

$$\Delta = \frac{P}{K_f} = \frac{20.17kN}{1.37kN/mm} = 14.72mm$$

3.5 Finite Element Analysis

In this study, Response Spectrum Analysis (RSA) is adopted for the analysis of each prepared model. In order to perform the seismic analysis of a structure at any location, the actual records of time history are needed. But it is not practically possible to prepare such records for all the areas and hence in many cases, these records are unavailable. In such cases, response spectrum analysis is carried out. This method involves the calculation of only the maximum or peak values of member forces and displacements in each of the considered mode using prescribed design spectra. This spectrum is the average of several past earthquakes and can be used as an effective method of employing earthquake ground motions. As per ES-EN 1998: 2015, the response spectra shown in Fig. 3.1 is recommended for design of structures subjected to seismic forces. The spectra is shown for all four seismic zones in Ethiopia as per ES-EN 1998: 2015. A response spectrum is basically a plot of the peak steady-state response in terms of displacement, velocity or acceleration, for of a series of varying natural frequencies. The main limitation of RSA is that it is universally acceptable only for linear systems. For nonlinear analysis, Time-History Analysis is adopted.

3.5.1 Procedures

After the scope, aim and objectives of the project were identified and methodology to conduct the research was planned, preliminary study was conducted, modeling and analysis of all sample wall models was carried out using ETABS 19.0.0. The lateral loads were assigned according to ES EN1998-1:2015's provisions. The general ETABS procedures used during modeling and analysis of all shear wall models are briefly listed below.

1. Defining material and section properties: All shear wall models were defined using the geometric and material properties discussed in section 3.3.3. Shell element was used to define all shear walls.
2. Drawing wall elements: The walls were drawn using shell element and the material and section properties defined before.
3. Defining modal response spectrum functions: modal response spectrum parameters used as an input in ETABS 19.0.0 were: ground type B and soil factor, S=1.2 were used for spectrum type 1.

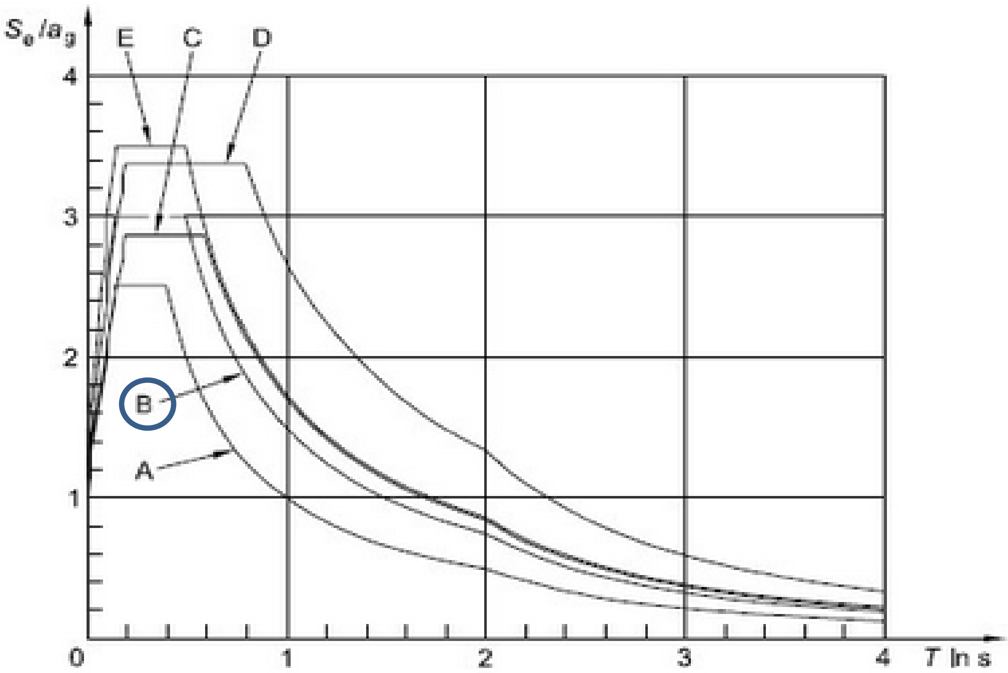


Fig. 3.1 Recommended Type 1 elastic response spectra for ground types A to E (5% damping)

The values of T_B, T_C and T_D describing the shape of elastic response spectrum depends upon the ground type and are 0.15, 0.5 and 2.0 respectively for response spectrum type 1 and ground type B and soil factor mentioned below. 5% constant damping was used during defining the response spectrum functions in both horizontal directions. Ground acceleration ratio, a_g=0.1g, 0.15g and 0.2g and lower bound factor, Beta=0.2 with importance factor, I = 1.0 were also used during defining the response spectrum function. Behavior factor(q) is taken 3.9 and 3.12 for the regular and irregular buildings respectively.

Table 3.4 Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

4. Defining minimum number of modes: The minimum number of modes taken into account for instance to the 10-storied frame and shear wall model is 30 considering three mass degree of freedom per floor i.e., two translational and one rotational.

5. Defining load patterns and load cases: The seismic load patterns (RSX and RSY) were defined in both orthogonal directions according to ES-EN 1998:2015's provisions and damping correction factor A =1. The "“Complete Quadratic Combination” (CQC) was used for combination of the response of different modes and square-root-of- sum-of-squares (SRSS) was used for directional combination of the modes.

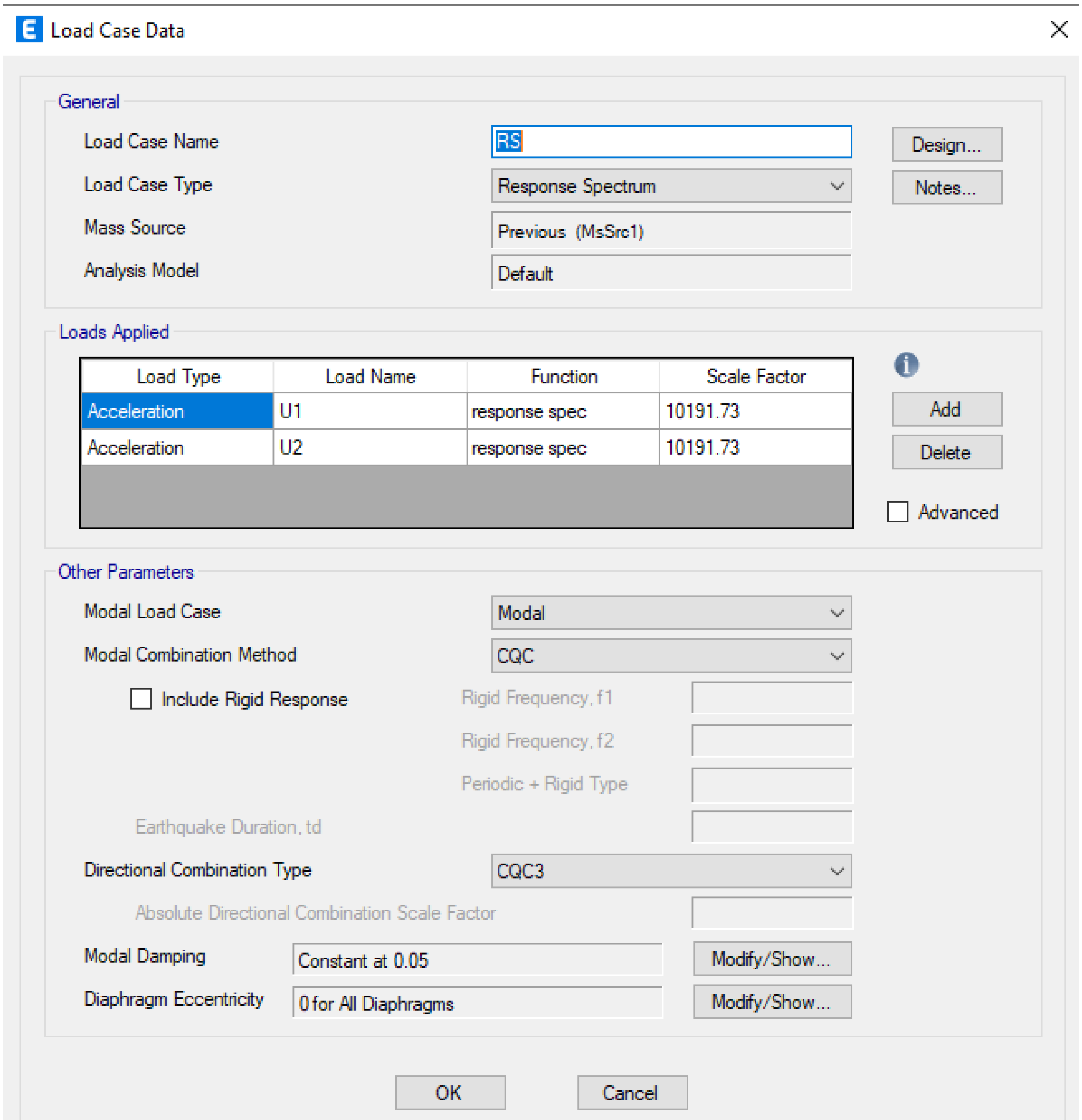


Fig. 3.2: Response spectrum input data in the x and y direction

6. Meshing of the wall element: All shear wall models were discretized in to finite elements. The property assignments to meshed area objects are the same as the original area object. Load and mass assignments on the original area object are appropriately broken up onto the meshed area objects.
7. Defining and assigning piers: Piers were defined and assigned for each shear walls and auto line constraint was assigned to each wall elements of all models.

8. Structural modeling and Analysis: lateral loads were assigned to the models; modal RSA was carried out with the help of ETABS19.0.0 software package to investigate the effect of shear wall curtailment on seismic responses of dual wall frame systems.

Ground acceleration ratio, ag = 0.1g, 0.15g and 0.2g (Table 3.5) and lower bound factor, Beta=0.2 with category of importance factor, I = 1.0 were also used during defining the response spectrum function.

Table 3.5 Bed rock acceleration ratio ag/g

Zone	5	4	3	2	1	0
$\alpha_o = a_g/g$	0.20	0.15	0.10	0.07	0.04	0

After the analysis is completed, the results obtained with respect to lateral displacement, story drift, base shear, and overturning moment of the models of each shear wall types are discussed. And top story lateral displacement is compared between FEA and Analytic solution. Finally, conclusion is drawn and recommendation is suggested for future studies.

3.5.2 Modeling

There are basically three main models considered i.e., a 10-storied frame-wall model, a 20-storied frame-wall model and a 30-storied frame-wall model. And under the main models there are 6 sub-models for the 10-storied frame-wall model, 7 sub-models for the 20-storied frame-wall model and 10 sub-models for the 30-storied frame-wall model each with different level of shear wall curtailment as shown in table 3.5:

i. For the 10-storied frame-wall model

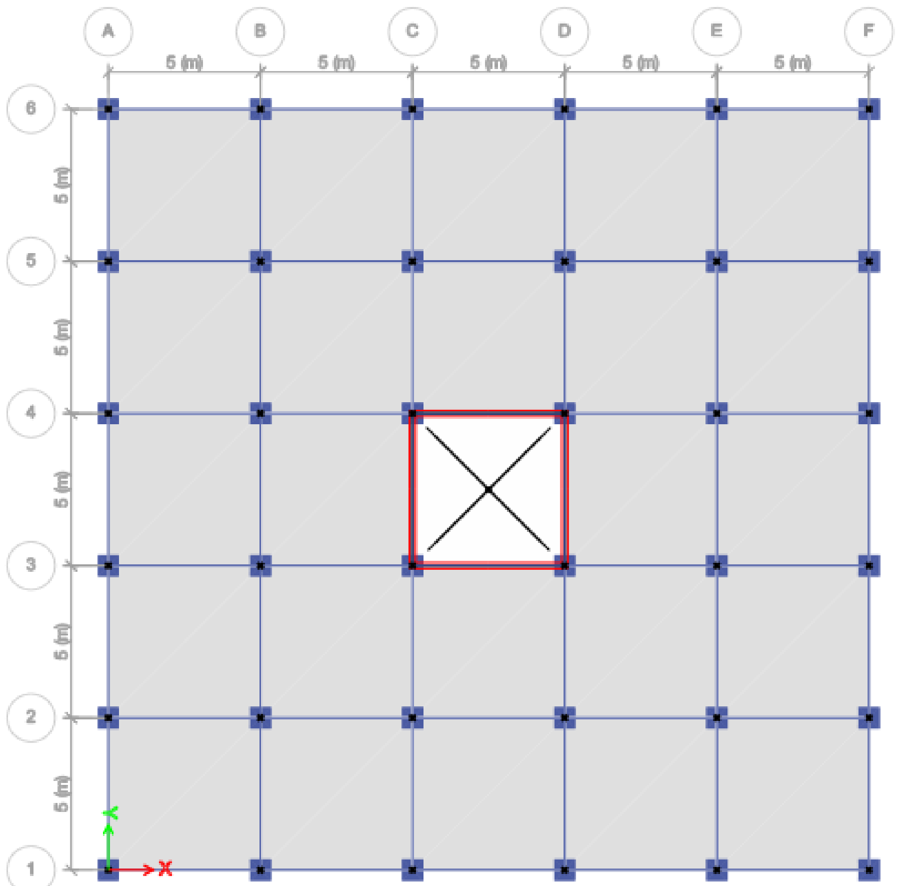
✓ Sizes of the members are as follows:

Table 3.6 Design Specification of the 10-storied frame-wall model

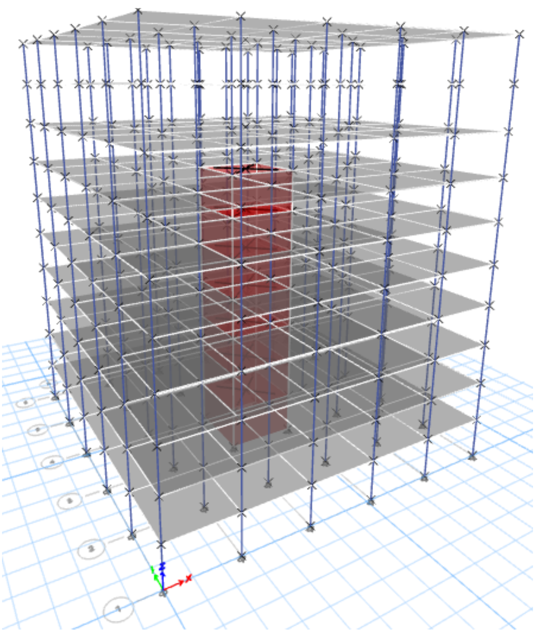
<table>
<tr><td>SN</td><td colspan="3">Specifications</td><td>Size</td></tr>
<tr><td>1</td><td colspan="3">Plan dimensions</td><td>25m x 25m</td></tr>
<tr><td>2</td><td colspan="3">Total height of Building</td><td>30 m</td></tr>
<tr><td>3</td><td colspan="3">Slab Thickness</td><td>200 mm</td></tr>
<tr><td>4</td><td colspan="3">Shear wall thickness</td><td>200 mm</td></tr>
<tr><td>5</td><td colspan="3">Beam Size</td><td>0.5 m x 0.3 m</td></tr>
<tr><td rowspan="2">6</td><td rowspan="2" colspan="2">Column Size</td><td>From Ground. - 5th floor</td><td>0.8 m x 0.8 m</td></tr>
<tr><td>From 5th – last</td><td>0.5 m x 0.5 m</td></tr>
<tr><td rowspan="5">7</td><td rowspan="5">Loads Applied</td><td rowspan="2">DL</td><td>Dead Load</td><td>as per Self Weight</td></tr>
<tr><td>Floor Finish</td><td>1 kN/m2</td></tr>
<tr><td>LL</td><td>Live Load</td><td>3 kN/m2</td></tr>
<tr><td>EQX</td><td>Seismic Load (X direction)</td><td>as per ES-EN 1998-2015</td></tr>
<tr><td>EQY</td><td>Seismic Load (Y direction)</td><td>as per ES-EN 1998-2015</td></tr>
</table>

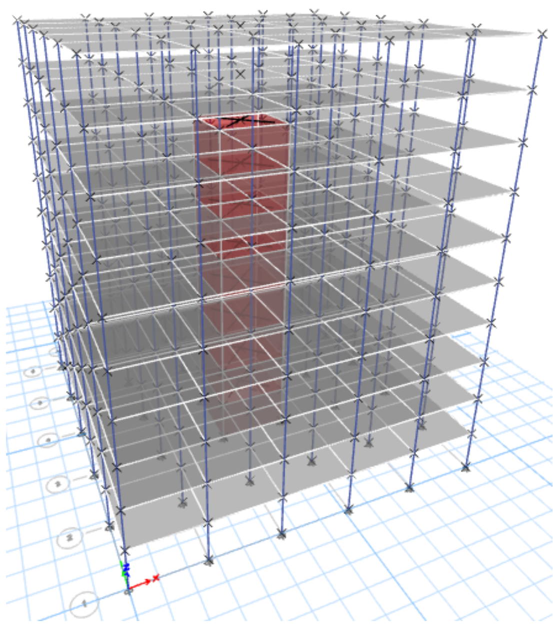
Table.3.7 Building Model Details of the 10-storied frame-wall model

Building Prototype	**Height of Shear Wall**
SW@10	100% height of frame
SW@9	90% height of frame
SW@8	80% height of frame
SW@7	70% height of frame
SW@6	60% height of frame
SW@5	50% height of frame



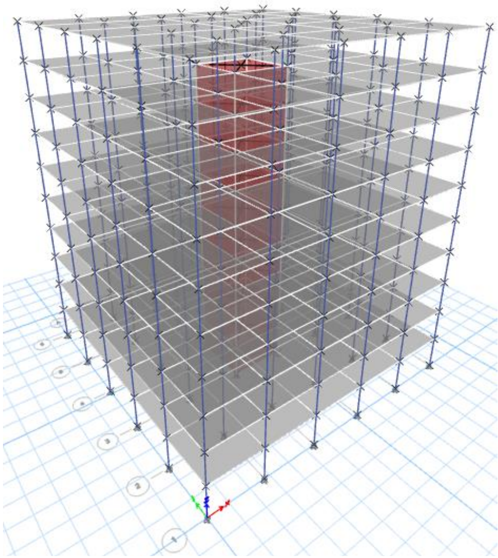
a. Plan view



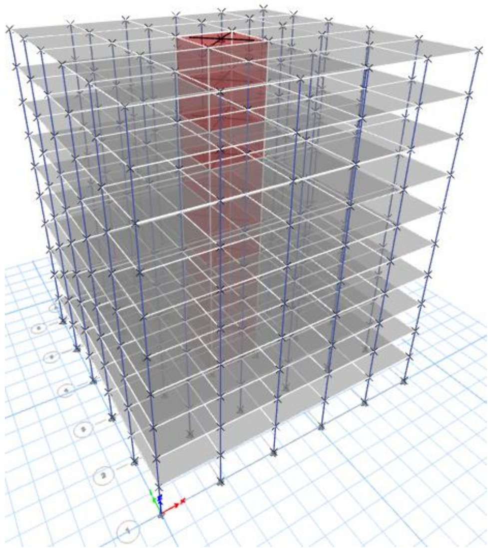


a. SW@ 7

b. SW@ 8



c. SW@ 9



d. SW@ 10

Fig 3.3 Generated plan and sample 3D sub-models of the 10-storied frame-wall model in ETABS

ii. For the 20-storied frame-wall model

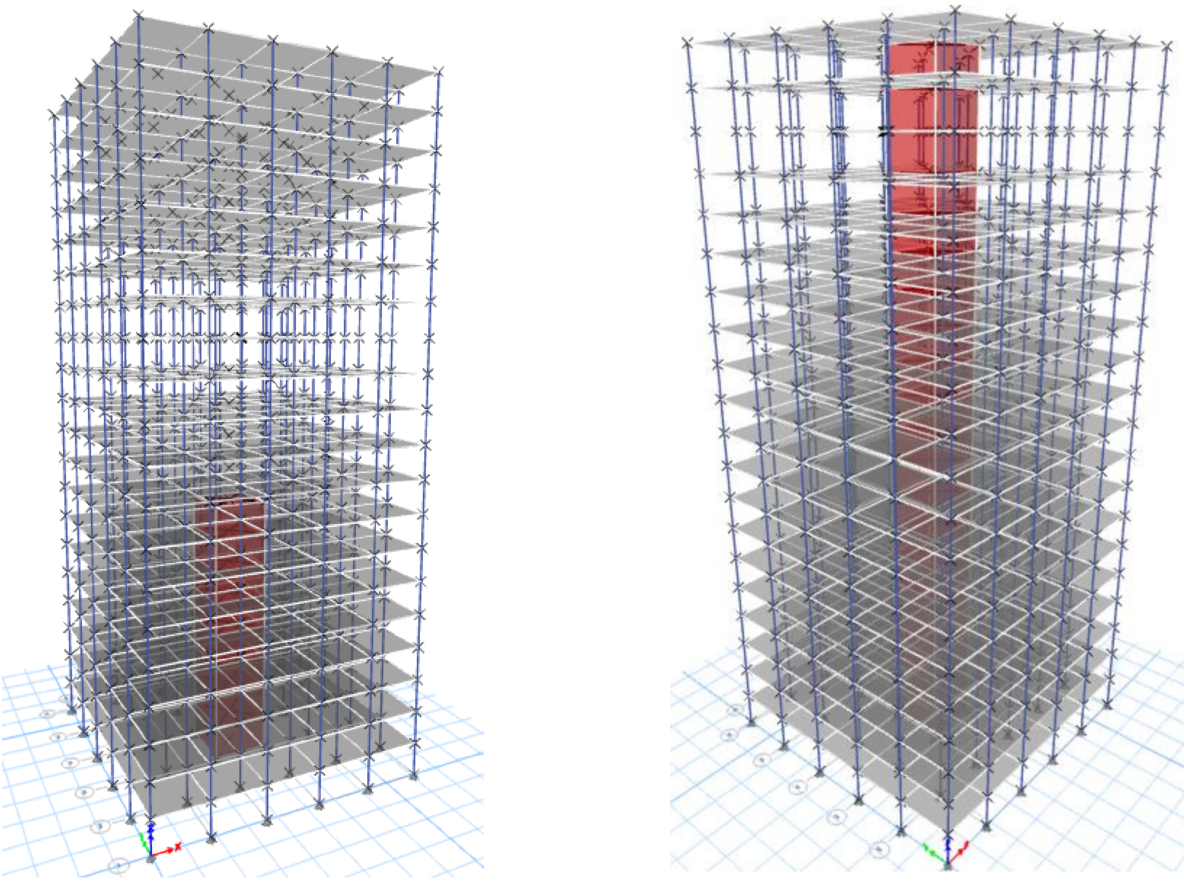
✓ Sizes of the members are as follows:

Table 3.8 - Design Specification of the 20-storied frame-wall model

SN	Specifications			Size
1	Plan dimensions			25m x 25m
2	Total height of Building			60 m
3	Slab Thickness			200 mm
4	Shear wall thickness			400 mm
5	Beam Size			0.5 m x 0.3 m
6	Column Size		From Ground. - 10^{th} floor	1 m x 1 m
			From 10^{th} – 15^{th}	0.8 m x 0.8 m
			From 16^{th} – last	0.5 m x 0.5 m
7	Loads Applied	DL	Dead Load	as per Self Weight
			Floor Finish	1 kN/m2
		LL	Live Load	3 kN/m2
		EQX	Seismic Load (X direction)	as per ES-EN 1998-2015
		EQY	Seismic Load (Y direction)	as per ES-EN 1998-2015

Table.3.9 Building Model Details of the 20-storied frame-wall model

Building Prototype	**Height of Shear Wall**
SW@20	100% height of frame
SW@18	90% height of frame
SW@16	80% height of frame
SW@14	70% height of frame
SW@12	60% height of frame
SW@10	50% height of frame
SW@8	40% height of frame



a. SW 8 b. SW 20

Fig 3.4 Generated sample 3D sub-models of the 20-storied frame-wall model in ETABS

iii. For the 30-storied frame-wall model

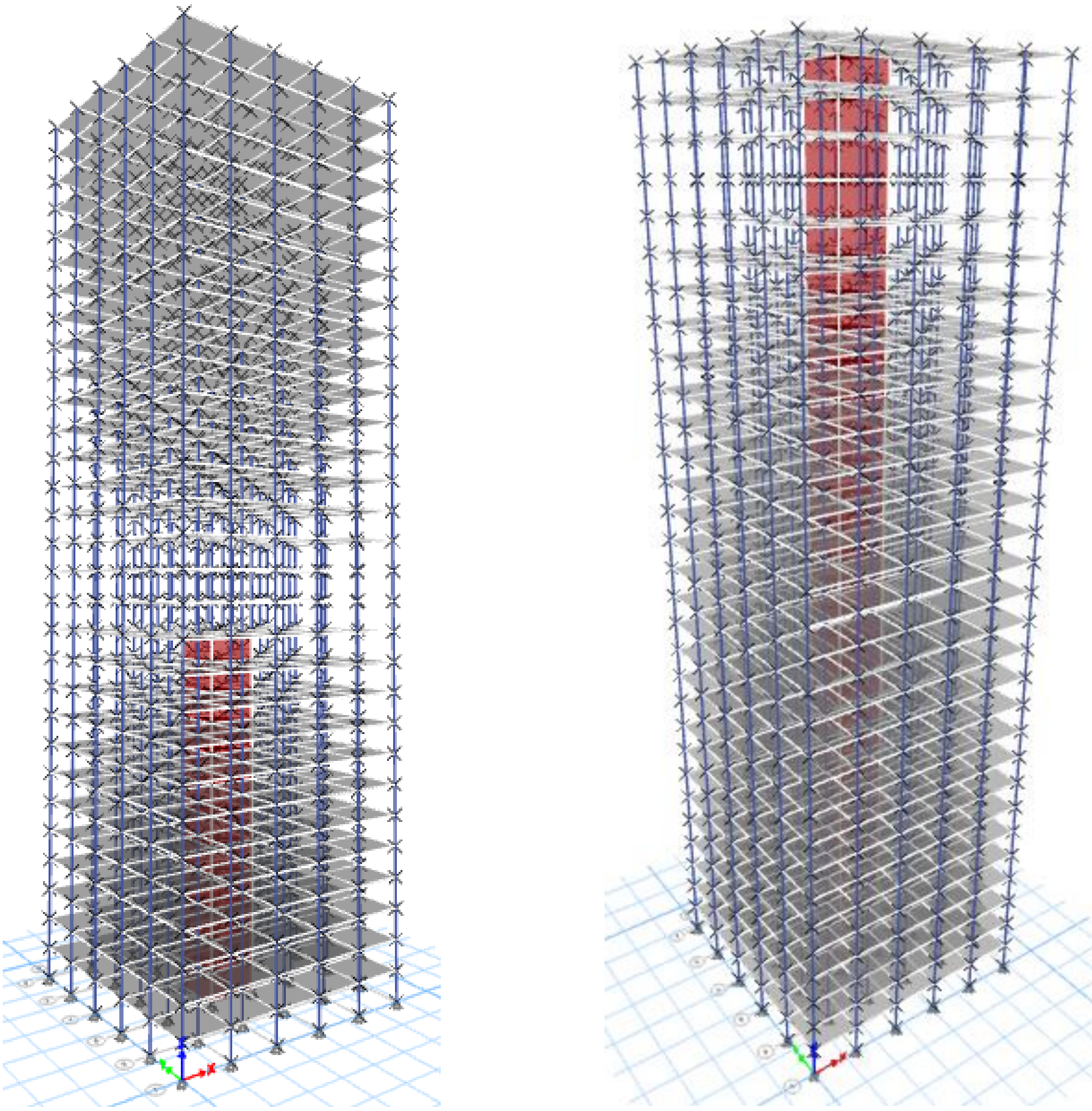
- ✓ Sizes of the members are as follows:

Table 3.10 Design Specification of the 30-storied frame-wall model

SN	Specifications			Size
1	Plan dimensions			25m x 25m
2	Total height of Building			90 m
3	Slab Thickness			200 mm
4	Shear wall thickness			400 mm
5	Beam Size			0.5 m x 0.3 m
6	Column Size		From Ground. - 10th floor	1.4 m x 1.4 m
			From 10th – 18th	1 m x 1 m
			From 19th - 25th	0.8 m x 0.8 m
			From 26th – last	0.5 m x 0.5 m
7	Loads Applied	DL	Dead Load	as per Self Weight
			Floor Finish	1 kN/m2
		LL	Live Load	3 kN/m2
		EQX	Seismic Load (X direction)	as per ES-EN 1998-2015
		EQY	Seismic Load (Y direction)	as per ES-EN 1998-2015

Table.3.11 Building Model Details of the 30-storied frame-wall model

Building Prototype	**Height of Shear Wall**
SW@30	100% height of frame
SW@28	93.33% height of frame
SW@26	86.67% height of frame
SW@24	80% height of frame
SW@22	73.33% height of frame
SW@20	66.67% height of frame
SW@18	60% height of frame
SW@16	53.33% height of frame
SW@14	46.67% height of frame
SW@12	40% height of frame



a. SW 12

b. SW 30

Fig 3.5 Generated sample 3D sub-models of the 30-storied frame-wall model in ETABS

CHAPTER 4: RESULT AND DISCUSSION

4.1 Continuum Analysis Results

4.1.1 Analysis Results of Alex Coull's Method

Alex Coull's method is one of the analytic (continuum) methods used to obtain specially the top displacement of a structure but may have less contribution in determining the effect of shear wall curtailment on top story deflection.

As discussed above in the chapter 3, following all the procedures stated above and determining all the necessary parameters we can get the lateral displacement of a building at different story level using equation 3.2:

$$y(z) = \frac{wH^4}{El}\left\{\frac{1}{(\alpha\,\mathrm{H})^4}\left\{\frac{(\alpha\,\mathrm{H}\sin h\,(\alpha\,\mathrm{H})+1)}{\cos h\,(\alpha\,\mathrm{H})}(\cos h\,(\alpha z)-1)-\alpha\,\mathrm{H}\sinh(\alpha z)+\ (\alpha\mathrm{H})^2\left[\frac{z}{H}-\frac{1}{2}\left[\frac{z}{H}\right]^2\right]\right\}\right\}$$

From the results obtained above the top story displacement of Alex Coull's method i.e., 14.89mm differ from that of the finite element analysis result i.e., 14.32mm by 0.57mm (3.83%). The reason behind is that the continuum analysis (Alex Coull's method) doesn't consider one of the causes for lateral displacement of a building which is the axial deformation of the columns, but as discussed in the literature section the two main factors that cause lateral deformation of a building are the racking shear and the axial deformation of each and every column. So, Alex Coull's method result shows only the racking shear component ignoring the axial deformation of the columns.

- ✓ And then results of the G+10 building model are tabulated and shown in graph below.

Story Level	y(z) (mm)
10	14.89
9	11.36
8	8.44
7	6.07
6	4.17
5	2.70
4	1.60
3	0.83
2	0.34
1	0.08
0	0.00

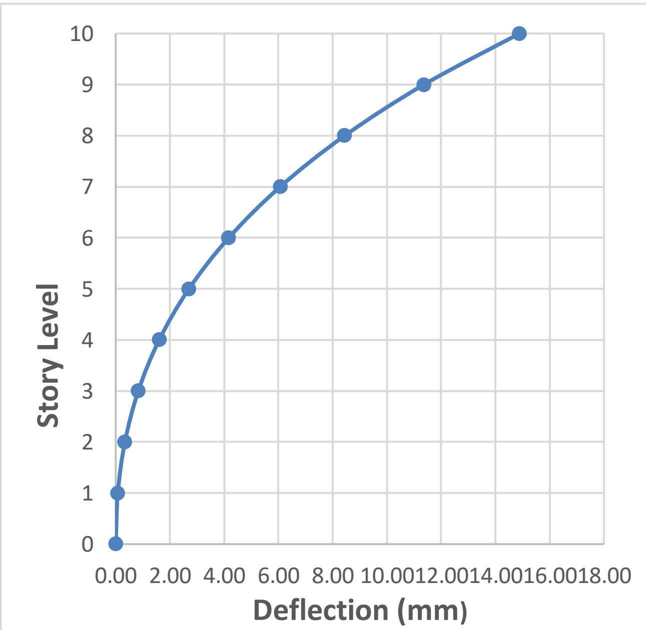


Fig 4.1 Alex Coull's deflection y(z) at different story level of the G+10 building

4.1.2 Analysis Results of Marie-José Nollet's Method

- **Analysis Results of the G+10 building**

$$y_1(H_1) = \frac{wH^4}{EI}\frac{(K^2-1)}{K^2}\left[\frac{{\xi_c}^2}{4} - \frac{{\xi_c}^3}{6} + \frac{{\xi_c}^4}{24} - \frac{\left(1-{\xi_c}^2\right)}{2}\frac{(\cosh k\alpha H_1 - 1)}{(k\alpha H)^2 \cosh k\alpha H_1}\right] + \frac{wH^4}{EI}\frac{I}{K^2}\left[\frac{\left(2\xi_c - {\xi_c}^2\right)}{2(k\alpha H)^2} - \frac{tanhk\alpha H_1}{(k\alpha H)^3} + \frac{(coshk\alpha H_1 - 1)}{(k\alpha H)^4 coshk\alpha H_1}\right]$$

Table 4.1 Deflection results of sub-structure 1 for the G+10 model

Curtailment level (H1)	ξ_c	K^2	k α H	k α H1	tan h (k α H1)	Cos h (k α H1)	$y_1(H_1)$ (mm)
10	1	1	1.43	1.43	0.8917	2.2090	15.01
9	0.9	1	1.43	1.287	0.6126	1.1727	14.44
8	0.8	1	1.43	1.144	0.5383	1.1357	16.87
7	0.7	1	1.43	1.001	0.4662	1.1033	18.20
6	0.6	1	1.43	0.858	0.3961	1.0756	18.48
5	0.5	1	1.43	0.715	0.3276	1.0523	17.73

In order to obtain $y_2(H_2)$ we need first to determine ϕH_1 and $y_1'(H_1)$. So, the tables 4.2 show the results obtained for$y_1'(H_1)$ and ϕH_1 respectively.

$$y_1'(H_1) = \frac{wH^3}{EI}\frac{(K^2-1)}{K^2}\left[\frac{1}{2}\left(\xi_c - {\xi_c}^2 + \frac{{\xi_c}^3}{3}\right) - \frac{(1-\xi_c)^2}{2}\frac{\tanh k\alpha H_1}{k\alpha H}\right] + \frac{wH^3}{EI}\frac{I}{K^2}\left[\frac{(1-\xi_c)}{(k\alpha H)^2} - \frac{1}{(k\alpha H)^2 coshk\alpha H_1} + \frac{(\tanh k\alpha H_1)}{(k\alpha H)^3}\right]$$

Table 4.2 Slope results of sub-structure 1 for the G+10 model

Curtailment level (H1)	ξ_c	k α H	k α H1	tan h (k α H1)	Cos h (k α H1)	$y_1'(H_1)$
10	1	1.43	1.43	0.89	2.21	0.1615
9	0.9	1.43	1.287	0.61	1.17	0.1108
8	0.8	1.43	1.144	0.54	1.14	0.0974
7	0.7	1.43	1.001	0.47	1.10	0.0843
6	0.6	1.43	0.858	0.40	1.08	0.0717
5	0.5	1.43	0.715	0.33	1.05	0.0593

$$\phi H_1 = \frac{wH^3}{EI}\frac{(k^2-1)}{6}[(1-(1-\xi_c)^3)] - (k^2-1)\,y_1'(H_1)$$

Table 4.3 ϕH_1 results of sub-structure 1 for the G+10 model

Curtailment level (H1)	ξ_c	k α H	k α H1	tan h (k α H1)	Cos h (k α H1)	ϕH_1
10	1	1.43	1.43	0.89167	2.20900	0.00067
9	0.9	1.43	1.287	0.61260	1.17272	(0.00127)
8	0.8	1.43	1.144	0.53830	1.13568	(0.00119)
7	0.7	1.43	1.001	0.46622	1.10334	(0.00110)
6	0.6	1.43	0.858	0.39609	1.07559	(0.00099)
5	0.5	1.43	0.715	0.32759	1.05229	(0.00087)

then after obtaining the values of $y_1'(H_1)$ and ϕH_1, we can calculate values for $y_2(H_2)$ using equation 2.16 and results are also shown in table 4.4:

$$y_2(H_2) = \frac{wH^4}{EI}\left[\frac{(1-\xi_c)^2}{2(\alpha H)^2} + \frac{(k^2-1)}{8}(1-\xi_c)^4\right] + \phi H_1((1-\xi_c)H)$$

In which EI, α^2, and k^2 are properties of sub-structure 1

Table 4.4 Deflection results of sub-structure 2 for the G+10 model

Curtailment level (H1)	ξ_c	H1	k α H	k α H1	tan h (k α H1)	Cos h (k α H1)	$y_2(H_2)$ (mm)
10	1	30	1.43	1.43	0.892	2.209	-
9	0.9	27	1.43	1.287	0.613	1.173	(0.109)
8	0.8	24	1.43	1.144	0.538	1.136	0.211
7	0.7	21	1.43	1.001	0.466	1.103	0.971
6	0.6	18	1.43	0.858	0.396	1.076	2.181
5	0.5	15	1.43	0.715	0.328	1.052	3.852

Adding the lateral deflection at the top of Sub-structure 1 and using the above results, the final expression for the top deflection of the curtailed wall $y_c(H)$is:

$$y_c(H) = y_1(H_1) + y_2(H_2)$$

Table 4.5 Top deflections results of the curtailed wall for the G+10 building

Curtailment level (H1)	$y_1(H_1)\ (mm)$	$y_2(H_2)\ (mm)$	$y_c(H)\ (mm)$
10	15.01	-	15.01
9	14.44	(0.109)	14.33
8	16.87	0.211	17.08
7	18.20	0.971	19.18
6	18.48	2.181	20.66
5	17.73	3.852	21.58

Therefore, From the results obtained in table 4.5 above the top story displacement of Marie-José Nollet's method for the G+10 building model i.e., 15.01mm differ from that of the finite element analysis result i.e., 14.32mm by 0.69mm (4.59%).

- **Analysis Results of the G+20 building**

Following the same procedure as that of the above analysis for the G+10 building (i.e. first calculating $y_1(H_1)$ using similar equations used above and then calculating $y_2(H_2)$ which intern depends on ϕH_1 and $y_1'(H_1)$ and finally after determining $y_1(H_1)$ and $y_2(H_2)$ the final expression for the top deflection of the curtailed wall $y_c(H)$ can be obtained by adding $y_1(H_1)$ and $y_2(H_2)$. Therefore, all the parameters are calculated separately and the results are shown in the table 4.6 below:

Table 4.6 Top deflections results of the curtailed wall for the G+20 building

Curtailment level (H1)	$y_1(H_1)\ (mm)$	$y_1'(H_1)$	ϕH_1	$y_2(H_2)\ (mm)$	$y_c(H)\ (mm)$
20	38.01	0.0216	0.00053	-	38.01
18	43.52	0.0133	(0.00122)	(5.82)	37.71
16	44.58	0.0117	(0.00112)	(8.13)	36.45
14	44.12	0.0101	(0.00101)	(6.83)	37.28
12	42.15	0.0086	(0.00089)	(1.80)	40.36
10	38.71	0.0071	(0.00076)	7.11	45.82
8	33.81	0.0056	(0.00063)	20.048	53.86

- **Analysis Results of the G+30 building**

Again, for the G+30 building the parameters $y_1(H_1)$, $y_2(H_2)$ (with its components ϕH_1 and $y_1'(H_1)$) and finally the top deflection of the curtailed wall $y_c(H)$ results are calculated in similar manner with the above sample buildings and tabulated as shown in table 4.7:

Table 4.7 Top deflections results of the curtailed wall for the G+30 building

Curtailment level (H1)	$y_1(H_1)\ (mm)$	$y_1'(H_1)$	ϕH_1	$y_2(H_2)\ (mm)$	$y_c(H)\ (mm)$
30	71.85	0.006663	0.00044	-	71.85
28	71.61	0.006663	(0.00139)	(0.24)	71.37
26	69.65	0.006661	(0.00135)	(0.44)	69.21
24	67.01	0.006656	(0.00129)	(0.60)	66.40
22	63.34	0.006647	(0.00124)	(0.72)	62.62
20	58.66	0.006629	(0.00117)	(0.78)	57.87
18	52.98	0.006597	(0.00110)	(0.795)	52.18
16	46.35	0.006539	(0.00107)	(0.819)	45.53
14	38.89	0.006437	(0.00097)	(0.708)	38.18
12	30.80	0.006259	(0.00087)	(0.526)	30.27

4.1.3 Analysis Result of Component Stiffness Method-Equation C

In this thesis paper, all the models and sub models considered are regular in plan and are mainly plan-symmetric structures subjected to symmetric loading which avoid torsion. And if torsion is not considered, the Component Stiffness Method-Equation C which is simplified method of determining the interaction of frames and shear walls is acceptable.

In this method, like that of Alex Coull's method, column axial deformation contribution for top deflection is less (be around 0.08% of that due to bending) and so neglected. From the results obtained above the top story displacement is 14.72mm which differ from that of the finite element analysis i.e. 14.32mm by 0.40mm (2.72%).

4.2 Finite Element Analysis Results

4.2.1 Finite Element Analysis Results of 10 story Building

Story Displacement

Story displacement is the lateral displacement of the story relative to the base or it is the absolute value of displacement of the story under action of the lateral forces. The lateral force-resisting system can limit the excessive lateral displacement of the building. According to Eurocode the acceptance maximum lateral displacement limit could be taken as H/300. In our case, the change in lateral displacement of building at the top story in contrast to SW@10 becomes 5.77%, 9.97% and 14.84% in SW@9, SW@8 and SW@7 respectively, whereas increases by 64.06% in SW@5. Curtailment of shear wall at 90% of building height shows least displacement as compared to full height of building as shown in Fig. below. This affirms that interruption of shear wall at the optimum level (i.e. above the point of contraflexure (60% of the full height)) eliminates the reverse force and cause minimum deflection change. But as we can see the lateral displacement increment become 64.06% in SW@5 since the shear wall curtailment at the 5^{th} floor is below the point of contraflexure.

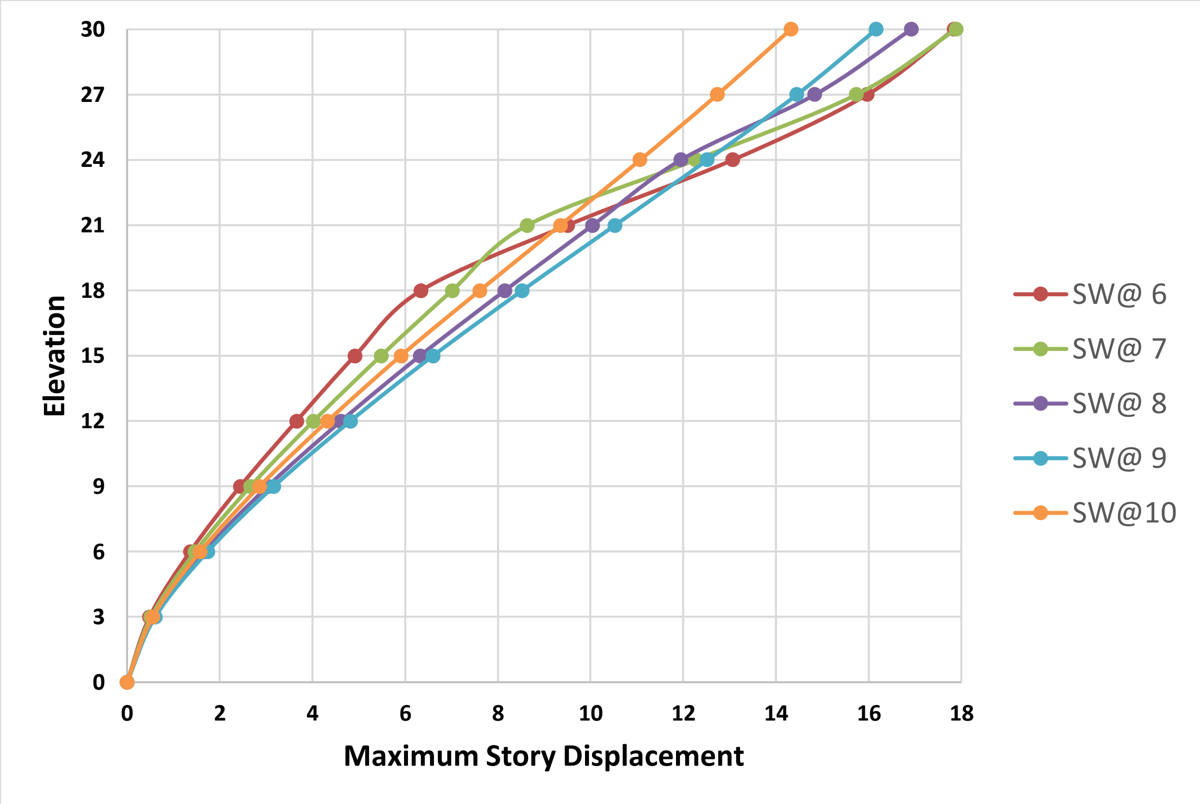


Fig 4.2 Story Displacement of the G+10 building models

Table 4.8 Story Displacement of the G+10 building models

Story level	Elevation	Max story displacement					
	m	Mm					
	X	SW@ 5	SW@ 6	SW@ 7	SW@ 8	SW@ 9	SW@10
1	3	0.884	0.486	0.500	0.549	0.571	0.557
2	6	2.426	1.362	1.390	1.567	1.632	1.579
3	9	4.264	2.441	2.502	2.855	2.976	2.859
4	12	6.257	3.655	3.783	4.340	4.529	4.327
5	15	8.522	4.917	5.151	5.942	6.207	5.907
6	18	14.2	6.345	6.592	7.665	8.015	7.611
7	21	22.694	9.503	8.121	9.444	9.898	9.347
8	24	30.388	13.071	11.530	11.234	11.772	11.072
9	27	36.269	15.972	14.793	13.951	13.588	12.737
10	30	39.843	17.849	16.817	15.907	15.198	14.321

Table 4.9 Change in story Displacement w.r.t SW@10 (in %)

Percent change in story displacement w.r.t SW@10					
Elevation	**SW@ 9**	**SW@ 8**	**SW@ 7**	**SW@ 6**	**SW@ 5**
m	**Δ in top displacement (in %)**				
0	0	0	0	0	0
3	2.38	-1.46	-11.40	-14.61	36.99
6	3.24	-0.77	-13.60	-15.93	34.91
9	3.93	-0.15	-14.26	-17.12	32.95
12	4.46	0.30	-14.39	-18.39	30.85
15	4.83	0.58	-14.67	-20.13	30.69
18	5.05	0.70	-15.45	-19.95	46.40
21	5.57	1.03	-15.10	1.64	58.81
24	5.94	1.44	3.97	15.29	63.56
27	6.26	8.71	13.90	20.25	64.88
30	5.77	9.97	14.84	19.77	64.06

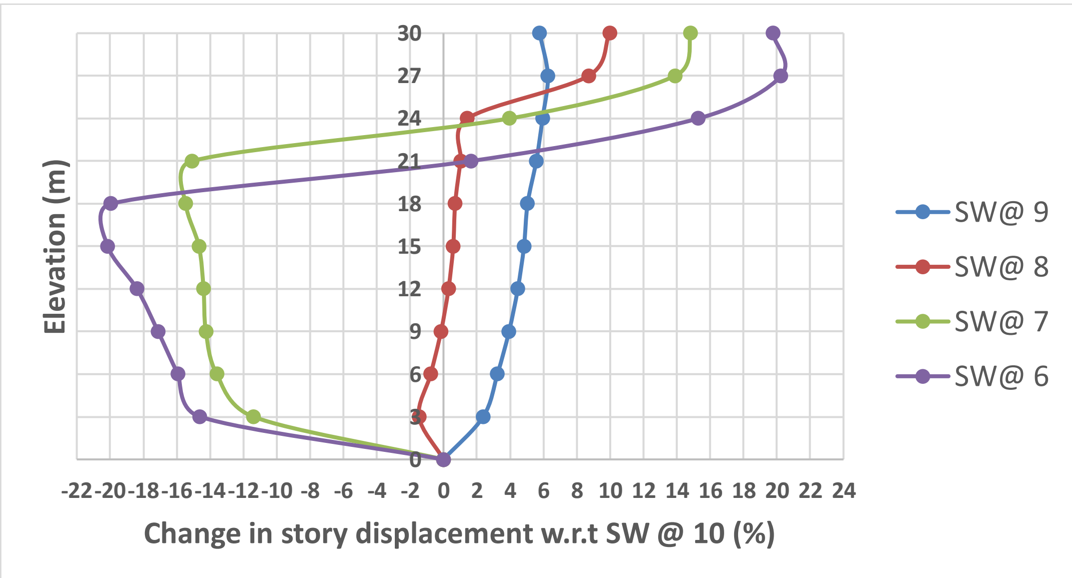


Fig 4.3 Change in Story Displacement w.r.t SW @ 10 (in %)

Story Drift

Story drift is the difference of displacements between two consecutive stories divided by the height of that story. The importance of story drift is in design of partitions/ curtail walls. They must be so designed as to accommodate the story drift; else they will crack.

Inter story drift of each model is calculated as per codal provision which demands the use of un-factor load combination and whose value should not exceed 0.005 times the story height. The values of inter story drift in all the models are within permissible limit. Results show that of all the models (ignoring the maximum story drift of SW@5 since it is below the inflection point) the maximum story drift are 0.00128 and 0.0013 for SW@6 and SW@7 respectively at the elevation near to 24m. The change in maximum inter story drift at top is 0.85%, 14.1%, 18.2% in SW@9, SW@8 and SW@7 respectively as compared with the full height SW@10. Whereas the inter story drift increases with SW@5 by 64.0% as shown in Fig. below. So generally, from the results we can see that the change in maximum inter story drift is very minimum up to the point of contraflexure, especially for SW@9 but increases below that point. Analysis reveals that after the maximum inter-story drift its value decreases with height.

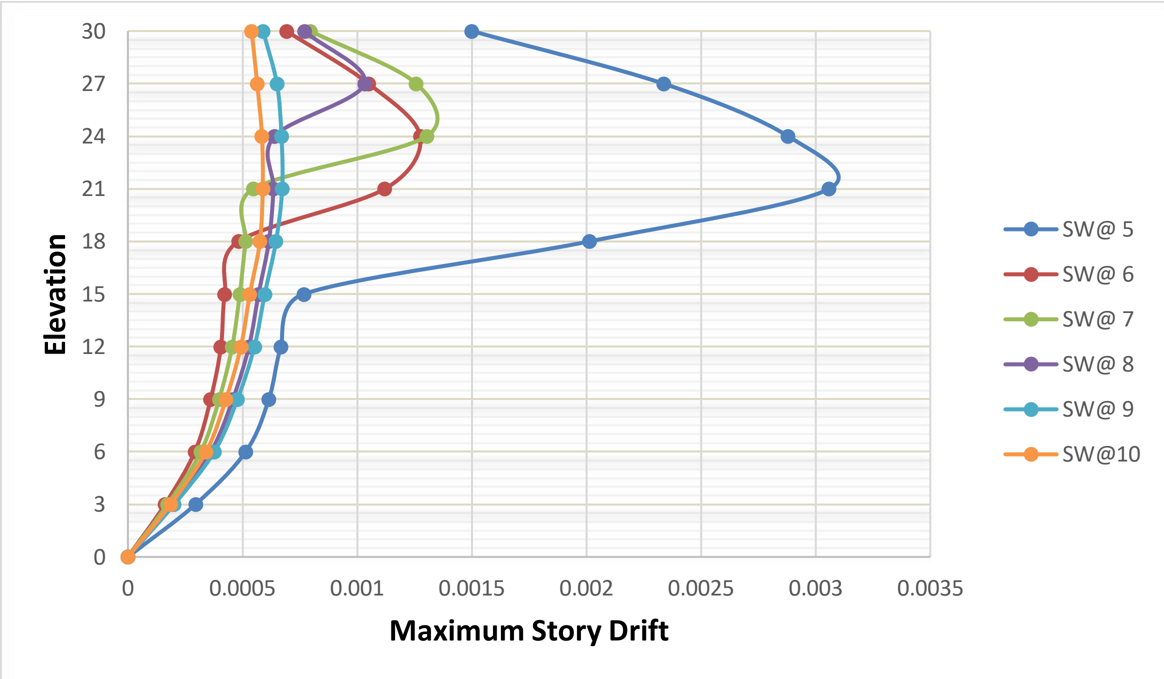


Fig 4.4 Story Drift of the G+10 building models

Table 4.10 Story Drift of the G+10 building models

Elevation	Maximum Story Drifts					
m						
X	SW@ 5	SW@ 6	SW@ 7	SW@ 8	SW@ 9	SW@10
30	0.001501	6.92E-04	7.97E-04	7.70E-04	5.90E-04	5.40E-04
27	0.002337	0.00105	0.001258	1.03E-03	6.50E-04	5.64E-04
24	0.00288	0.001275	0.001303	6.39E-04	6.70E-04	5.85E-04
21	0.003059	0.001121	5.47E-04	6.35E-04	6.73E-04	5.88E-04
18	0.002013	4.82E-04	5.14E-04	6.15E-04	6.46E-04	5.76E-04
15	7.67E-04	4.23E-04	4.88E-04	5.70E-04	5.97E-04	5.32E-04
12	6.68E-04	4.06E-04	4.55E-04	5.28E-04	5.52E-04	4.93E-04
9	6.15E-04	3.61E-04	3.98E-04	4.58E-04	4.78E-04	4.28E-04
6	5.14E-04	2.92E-04	3.18E-04	3.62E-04	3.77E-04	3.41E-04
3	2.95E-04	1.62E-04	1.73E-04	1.95E-04	2.02E-04	1.86E-04
0	0	0	0	0	0	0

In addition to the above point, one thing we need to give attention is that there exists a sudden story drift at each shear wall curtailment location that need to be stiffened, for instance by strong beams, for the respective models but for model SW@ 9 the drift is not significant as compared with the other models, meaning the curtailment don't significantly affect the story drift and we may not need to stiffen the building at the level of curtailment. But for SW@ 5 the story drift become very large.

Story Overturning Moment

Overturning moments are those applied moments, shears, and uplift forces that seek to cause the footing to become unstable and turn over. The moments are obtained by multiplying the story shear by the distance to the center of mass above the elevation considered. For each building, these moment ratios match closely the corresponding shear ratios, giving therefore about the same factor of safety for shear and overturning at the base.

The results show that the change in Story Overturning Moment at the base of SW@6 and SW@7 is 8.8% and 3.2% respectively as compared with the full height SW@10. Whereas the Story Overturning Moment at the base increases with SW@5 by 53.9% as shown in Fig. below. So generally, from the results we can again see that the change in Story Overturning Moment at the base is very minimum up to the point of contraflexure.

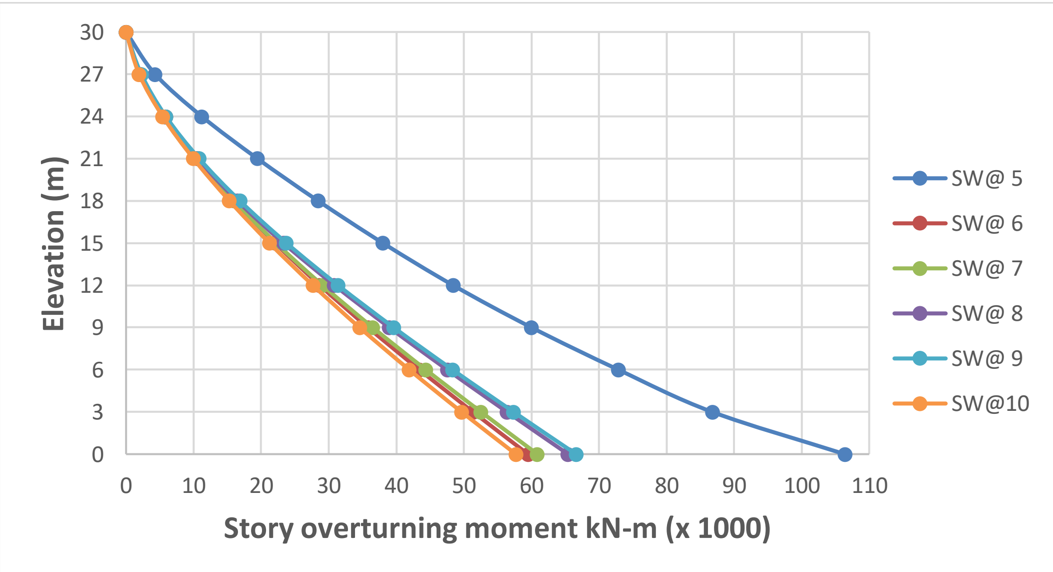


Fig 4.5 Story Overturning moments of the G+10 building models

Table 4.11 Story Overturning moments of the G+10 building models

Elevation	Story overturning moment					
M	kN-m					
X	SW@ 5	SW@ 6	SW@ 7	SW@ 8	SW@ 9	SW@10
0	106167.7	61842.807	65636.9	72960.9	75115.7	67873.6
3	86772.9	51347.2	52543.9	56394.1	57378.8	58307.2
6	72871.3	43415.7	44378.4	47535.8	48335.7	41919.4

9	60016.7	35819.3	36516.9	38979.3	39625.9	34563.4
12	48412.1	28670.9	29084.2	30850.3	31384.4	27655.5
15	38019.6	22073.6	22211.3	23290.8	23745.4	21224.2
18	28426	16064.2	16026.7	16477.2	16846.4	15304.7
21	19454	10553.5	10592.2	10604.8	10830.1	9979.63
24	11206.7	5772.14	5845.62	5848.23	5888.05	5424.62
27	4305.42	2106.7	2165.02	2178.5	2258.38	1942.76
30	0	0	0	0	0	0

Story Stiffness

Story stiffness is the ratio of story shear to story drift. Story stiffness is estimated as the lateral force producing unit translational lateral deformation in that story, with the bottom of the story restrained from moving laterally, i.e., only translational motion of the bottom of the story is restrained while it is free to rotate.

The results show that at the bottom elevations SW@8 and SW@9 have better Story stiffness even when compared with that of the building with full height shear wall SW@10 but near to the top stories SW@9 reveals consistent story stiffness like SW@10. Another point which needs attention is that at the elevation where the shear wall is curtailed the models show a sudden story stiffness change except SW@9 and the full height shear wall.

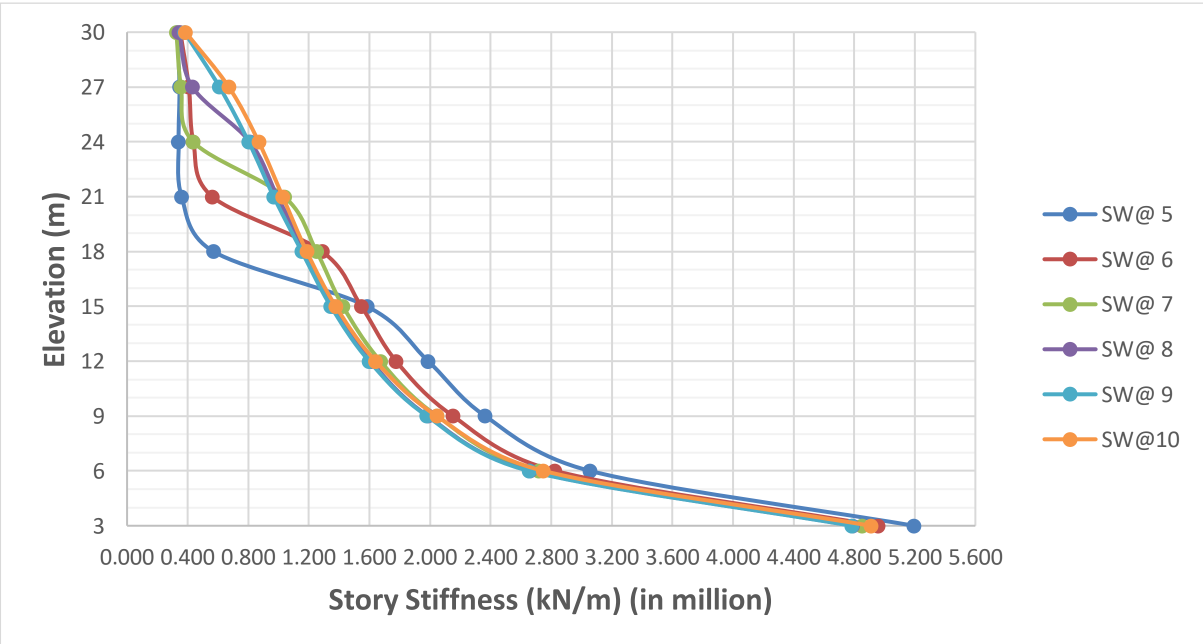


Fig 4.6 Story Stiffness of the G+10 building models

Table 4.12 Story Stiffness of the G+10 building models

Elevation	Story Stiffness					
M	kN/m (in million)					
X	SW@ 5	SW@ 6	SW@ 7	SW@ 8	SW@ 9	SW@10
30	0.3318	0.3566	0.3271	0.3474	0.4305	0.4452
27	0.3459	0.4079	0.3559	0.4314	0.6105	0.6718
24	0.3405	0.4372	0.4371	0.8109	0.8026	0.8691
21	0.3602	0.5632	1.0380	0.9930	0.9681	1.0277
18	0.5709	1.2875	1.2510	1.1768	1.1534	1.1859
15	1.5834	1.5468	1.4253	1.3626	1.3455	1.3776
12	1.9882	1.7764	1.6725	1.6089	1.5988	1.6411
9	2.3647	2.1523	2.0473	1.9860	1.9798	2.0448
6	3.0565	2.8255	2.7195	2.6592	2.6538	2.7487
3	5.1956	4.9567	4.8513	4.7905	4.7849	4.9100
0	0	0	0	0	0	0

Story Shear

In one method of designing a structure to have seismic resistance, the design seismic force is presumed to be applied at each floor level. The floor slab is considered to be very stiff in its own plane, because the large width of the structure. Hence, all floor slabs are presumed to simply move laterally in their own planes under seismic forces. The design seismic force to be applied at each floor level is called story shear. It is a fraction of the total dead load and a part of the live load acting at each floor level.

From the results we can see that the change in Story shear is very minimum up to the point of contraflexure but as soon as it lowers below the point of zero shear (contraflexure point) i.e. SW@5 the story shear deviates and increase significantly (from 714.53kN to 1512.18kN) as shown in the table below.

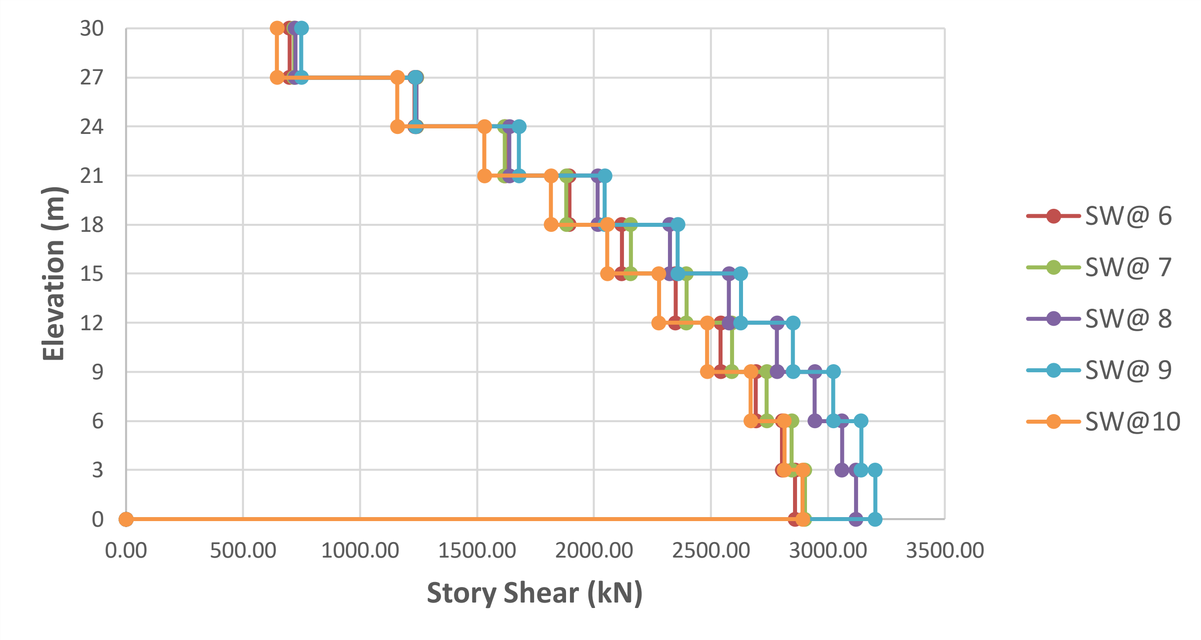


Fig 4.7 Story Shear of the G+10 building models

Table 4.13 Story Shear of the G+10 building models

Story	Elevation	Location	Story Shears					
			kN					
	M		SW@ 5	SW@6	SW@7	SW@8	SW@9	SW@10
Story10	30	Top	**1512.18**	**698.9**	**782.25**	**722.9**	**750.0**	**714.53**
	27	Bottom	1512.18	698.9	782.25	722.9	750.0	714.53
Story9	27	Top	2341.5	1233.4	1241.8	1240.2	1236.7	1159.7
	24	Bottom	2341.5	1233.4	1241.8	1240.2	1236.7	1159.7
Story8	24	Top	2862.9	1621.7	1617.8	1639.5	1679.9	1532.2
	21	Bottom	2862.9	1621.7	1617.8	1639.5	1679.9	1532.2
Story7	21	Top	3257.5	1894.5	1882.7	2016.1	2047.0	1817.2
	18	Bottom	3257.5	1894.5	1882.7	2016.1	2047.0	1817.2
Story6	18	Top	3643.7	2119.0	2158.1	2324.6	2358.9	2057.3
	15	Bottom	3643.7	2119.0	2158.1	2324.6	2358.9	2057.3
Story5	15	Top	4070.1	2348.7	2395.9	2577.2	2628.3	2277.7
	12	Bottom	4070.1	2348.7	2395.9	2577.2	2628.3	2277.7
Story4	12	Top	4547.0	2541.4	2589.4	2782.1	2850.5	2485.1
	9	Bottom	4547.0	2541.4	2589.4	2782.1	2850.5	2485.1
Story3	9	Top	4939.3	2693.0	2738.8	2944.5	3023.3	2670.3
	6	Bottom	4939.3	2693.0	2738.8	2944.5	3023.3	2670.3
Story2	6	Top	5227.8	2804.3	2846.3	3060.7	3142.9	2813.9
	3	Bottom	5227.8	2804.3	2846.3	3060.7	3142.9	2813.9
Story1	3	Top	5376.9	2859.7	2902.2	3120.7	3203.1	2891.6
	0	Bottom	5376.9	2859.7	2902.2	3120.7	3203.1	2891.6
Base	0	Top	0	0	0	0	0	0
	0	Bottom	0	0	0	0	0	0

Fundamental Period

The first-time interval of a periodic signal after which it repeats itself is called a fundamental period. It should be noted that the fundamental period is the first positive value of frequency for which the signal repeats itself. The fundamental time period of SW@9 and SW@8 are less than that of SW@10 for mode 1. Consequently, we can notice that by curtailing the shear wall near the top above point of contraflexure we can minimize the fundamental time period.

Table 4.14 Fundamental Time Period of the G+10 building models

Parameter	SW@10	SW@9	SW@8	SW@7	SW@6	SW@5
Fundamental Period - Mode 1 (sec)	0.884	0.878	0.892	0.924	0.971	1.037
Fundamental Period - Mode 2 (sec)	0.752	0.749	0.757	0.793	0.836	0.951

4.2.2 Finite Element Analysis Results of 20 story Building

Story Displacement

As we can see from the results below, by curtailing the shear wall below the full height the maximum story displacement can be minimized (improved well).

The other very important point is that when story height increases the contribution of the shear wall in lateral load resistance is relatively less as compared with that of the previous 10 storied model building.

From previously discussion in chapter 2, the core (wall) might be ten or more times as stiff as the frame for G+10 building. But this relation will automatically reduce to the core lateral load resistance contribution or stiffness being three times as that of the frame contribution for G+20 building and finally it even become worse for G+50 buildings in which the core stiffness contribution reduces to being only half as stiff as the frame (Alex Coull, 1991).

As shown in figure 4.8 below, the change in lateral story displacement of building at the top story in contrast to SW@20 becomes -1.33%, -1.65% and 0.88% in SW@ 18, SW@ 16 and SW@14 respectively, whereas increases by 11.66% in SW@8. The negative values show that the lateral story displacements are even decreased for the respective shear wall curtailments as compared with that of the building having full height shear wall (i.e. SW@20). Curtailment of shear wall at 80% of building height shows least displacement as compared to full height of building as shown in figure and table above. This affirms that interruption of shear wall at the optimum level (i.e. above the point of contraflexure) eliminates the reverse force and cause minimum deflection change. But as we can see the lateral displacement increment become 11.66% in SW@8 since the shear wall curtailment at the 8th floor is below the point of contraflexure.

Table 4.15 Story Displacement of the G+20 building models

Story Level	Elevation	Max story displacement						
	m	mm						
	x	SW@ 20	SW@ 18	SW@ 16	SW@ 14	SW@ 12	SW@ 10	SW@ 8
11	33	17.745	18.928	18.888	17.317	17.705	17.536	20.338
12	36	19.986	21.188	21.209	19.566	19.830	20.859	23.918
13	39	22.241	23.450	23.529	21.809	22.577	24.127	27.241
14	42	24.49	25.698	25.825	24.048	25.513	27.209	30.293
15	45	26.77	27.969	28.132	27.133	28.734	30.432	33.425
16	48	29.023	30.212	30.391	30.225	31.641	33.289	36.169
17	51	31.249	32.432	32.631	32.971	34.145	35.752	38.525
18	54	33.436	34.605	34.579	35.24	36.176	37.760	40.456
19	57	35.581	36.195	36.086	36.938	37.668	39.241	41.892
20	60	**37.653**	**37.158**	**37.042**	**37.991**	38.592	40.159	42.792

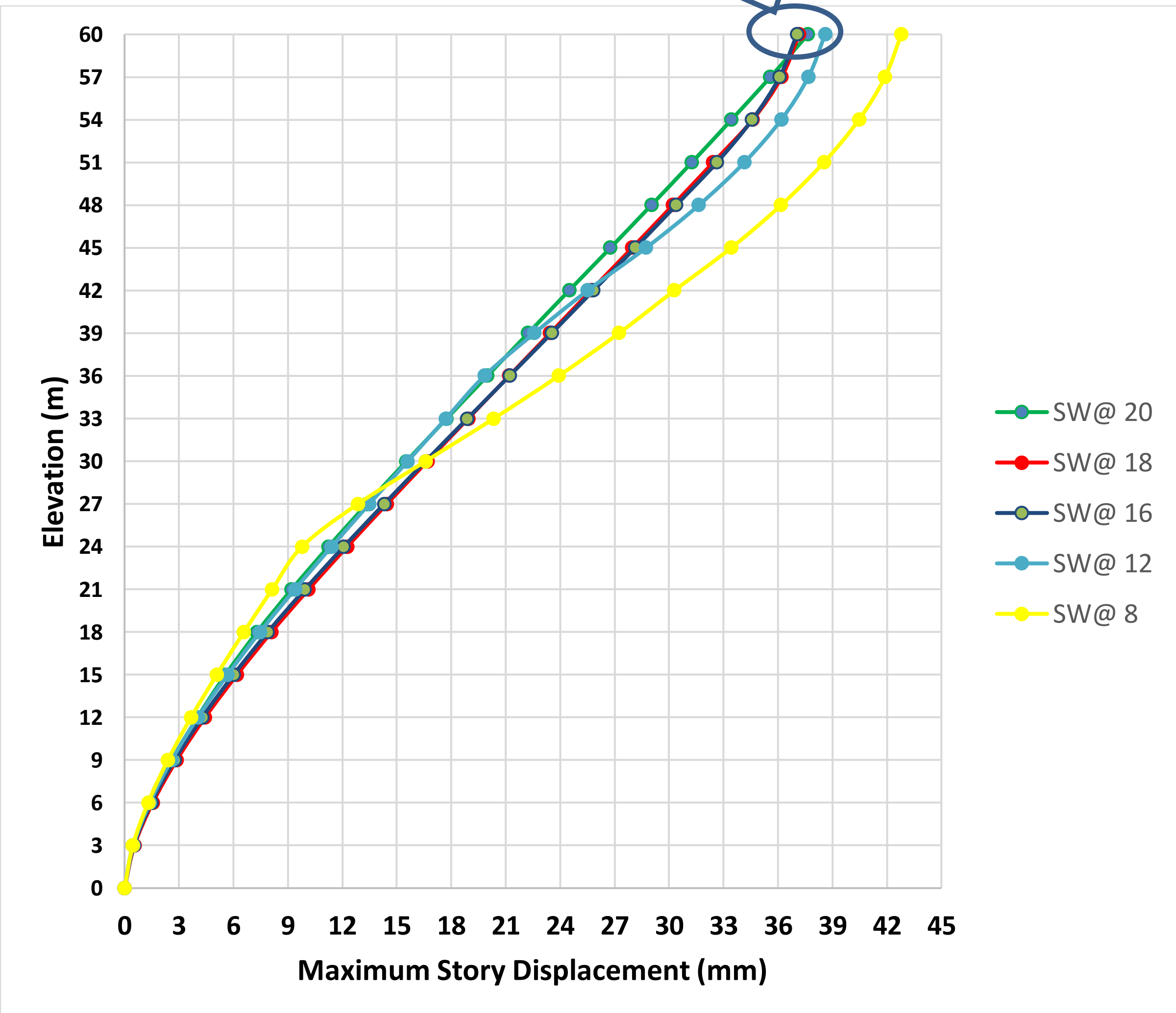


Fig 4.8 Story Displacement of the G+20 building models

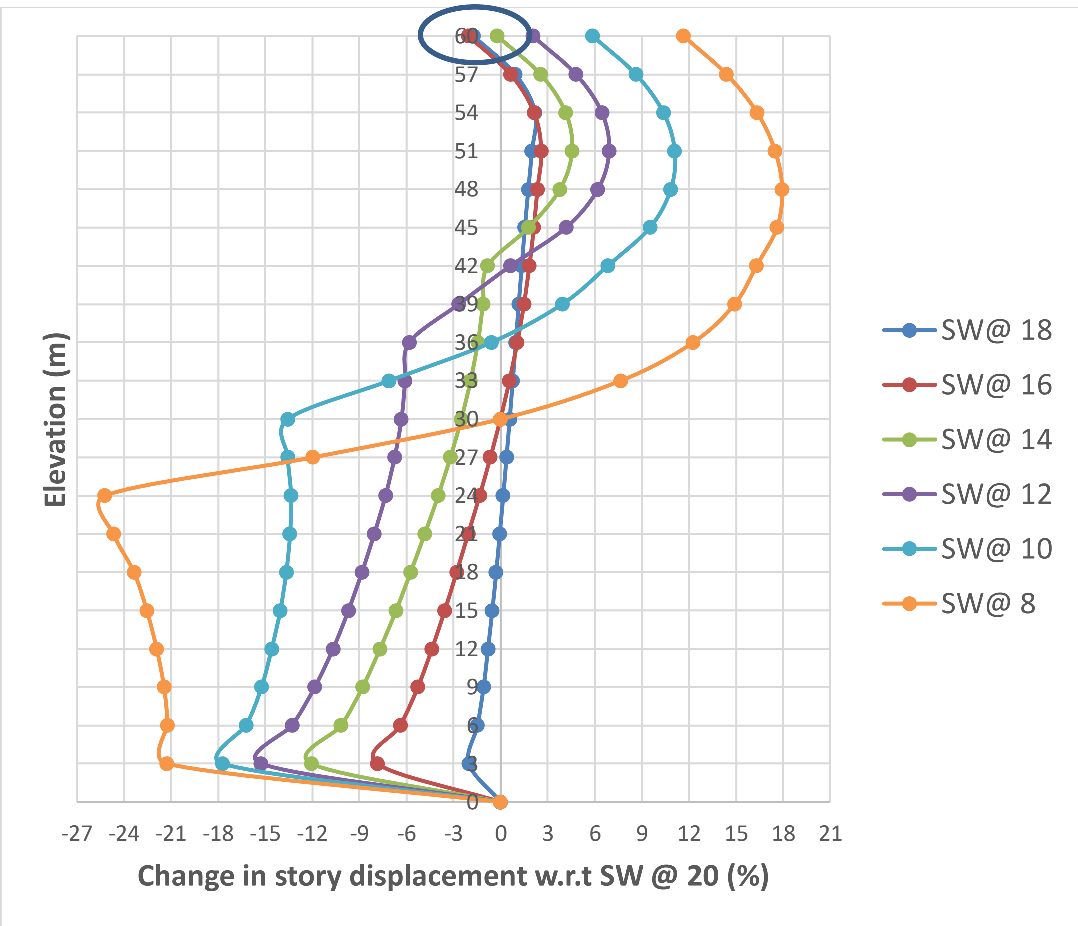


Fig 4.9 Change in Story Displacement w.r.t SW @ 20 (in %)

Table 4.16 Change in story Displacement w.r.t SW@ 20 (in %)

Elevation	Percent change in story displacement w.r.t SW@20					
m	mm					
X	SW@ 18	SW@ 16	SW@ 14	SW@ 12	SW@ 10	SW@ 8
45	1.54271	2.11263	1.77857	4.16547	9.51085	17.6131
48	1.7601	2.34001	3.77606	6.19621	10.8418	17.9402
51	1.99421	2.59107	4.55023	6.91114	11.096	17.4949
54	2.20131	2.12845	4.14321	6.44932	10.3742	16.346
57	0.9303	0.63227	2.54538	4.80657	8.62073	14.4043
60	**-1.3324**	**-1.6495**	**0.8896**	2.4335	6.2400	12.0091

Story Drift

Story drift which is the difference of displacements between two consecutive stories divided by the height of that story is also shown in the figure below.

Results show that of all the models (ignoring the maximum story drift of SW@8-SW@12 since they are below the inflection point) the maximum story drift is 0.001297 for SW@14 at the elevation near to 48m. The change in maximum inter story drift at the top is 19.92%, 21.75%, 29.23% in SW@18, SW@16 and SW@14 respectively as compared with the full height SW@20. Whereas the inter story drift increases with SW@5 by 58.2% as shown in Fig. below. So generally, from the results we can see that the change in maximum inter story drift increases while the curtailment level decreases. Analysis result reveals that after the maximum inter-story drift its value decreases with increasing height.

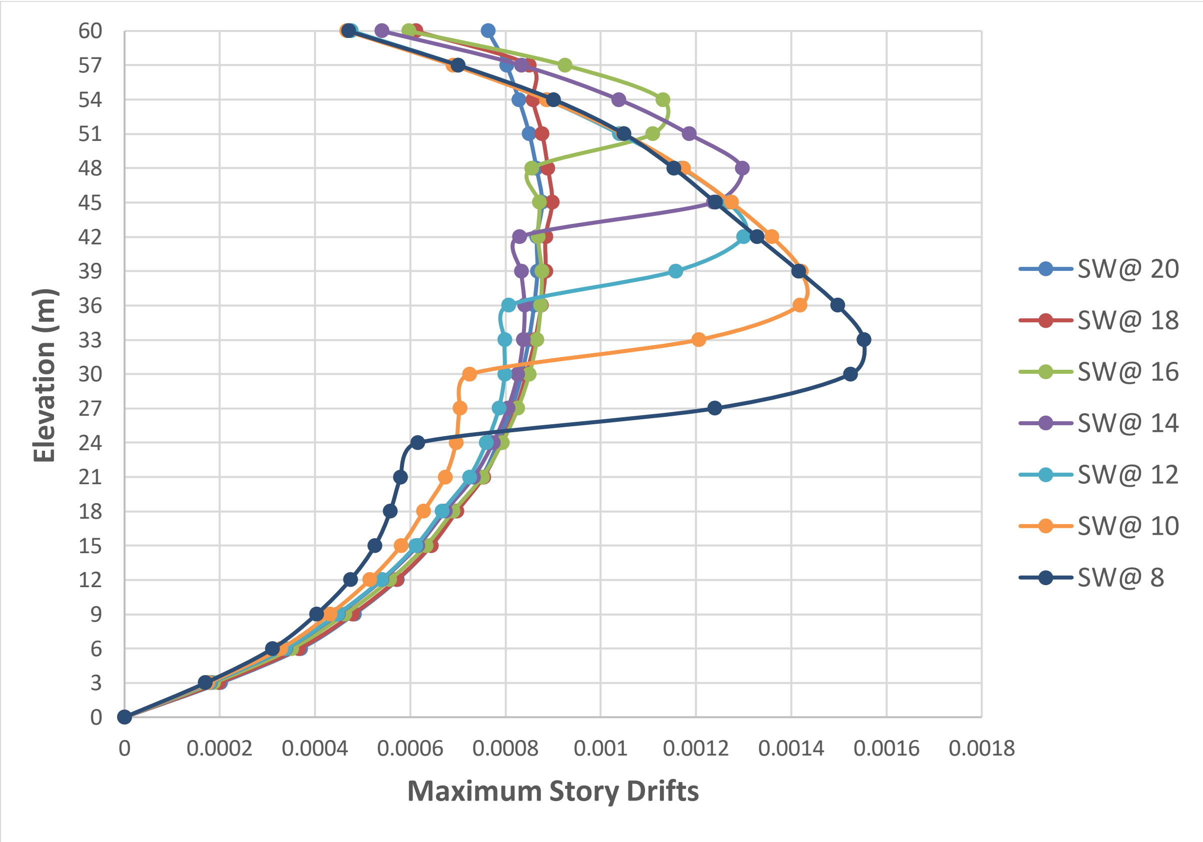


Fig 4.10 Story Drift of the G+20 building models

Additionally, as shown in the figure above and tried to show also in the table below underlined, there exists a sudden story drift at each shear wall curtailment location which needs a special concern (like stiffening that specific story by stiff beams) for the respective models but its effect lowers as the curtailment level decrease from SW@ 18 to SW@ 8.

Table 4.17 Story Drift of the G+20 building models

Elevation	Maximum Story Drifts						
m							
x	SW@ 20	SW@ 18	SW@ 16	SW@ 14	SW@ 12	SW@ 10	SW@ 8
60	0.00076	0.00061	0.000597	0.00054	0.000476	0.000466	0.00047
57	0.0008	0.00085	0.000925	0.000833	0.0007	0.00069	0.000701
54	0.00083	0.00086	0.00113	0.001038	0.000887	0.000886	0.000901
51	0.00085	0.00088	***0.001109***	0.001185	0.001039	0.001047	0.001049
48	0.00087	0.00089	***0.000855***	***0.001297***	0.00117	0.001173	0.001153
45	0.00088	0.0009	0.000871	***0.001237***	0.001269	0.001274	0.001241
42	0.00087	0.00088	0.000868	***0.000829***	**0.0013**	0.001359	0.001328
39	0.00087	0.00088	0.000876	0.000833	**0.001157**	0.001421	0.001416
36	0.00086	0.00088	0.000874	0.00084	**0.000806**	**0.001418**	0.001498
33	0.00085	0.00086	0.000866	0.000837	0.000798	**0.001206**	0.001552
30	0.00083	0.00084	0.00085	0.000826	0.000798	**0.000725**	0.001525
27	0.00081	0.00082	0.000826	0.000805	0.000786	0.000704	**0.00124**
24	0.00078	0.00079	0.000793	0.000774	0.00076	0.000696	**0.000616**

4.2.3 Finite Element Analysis Results of 30 story Building

Lateral Displacement

As shown in the figure below, the change in lateral story displacement of building at the top story in contrast to SW@30 becomes -0.905%, -2.04%, -2.81% and -3.17% in SW@ 28, SW@ 26, SW@24 and SW@22 respectively. The negative values show that the lateral story displacements are even decreased for the respective shear wall curtailments as compared with the full height shear wall (i.e. SW@30). Curtailment of shear wall at 73.33% of building height shows least displacement as shown in figure and table below.

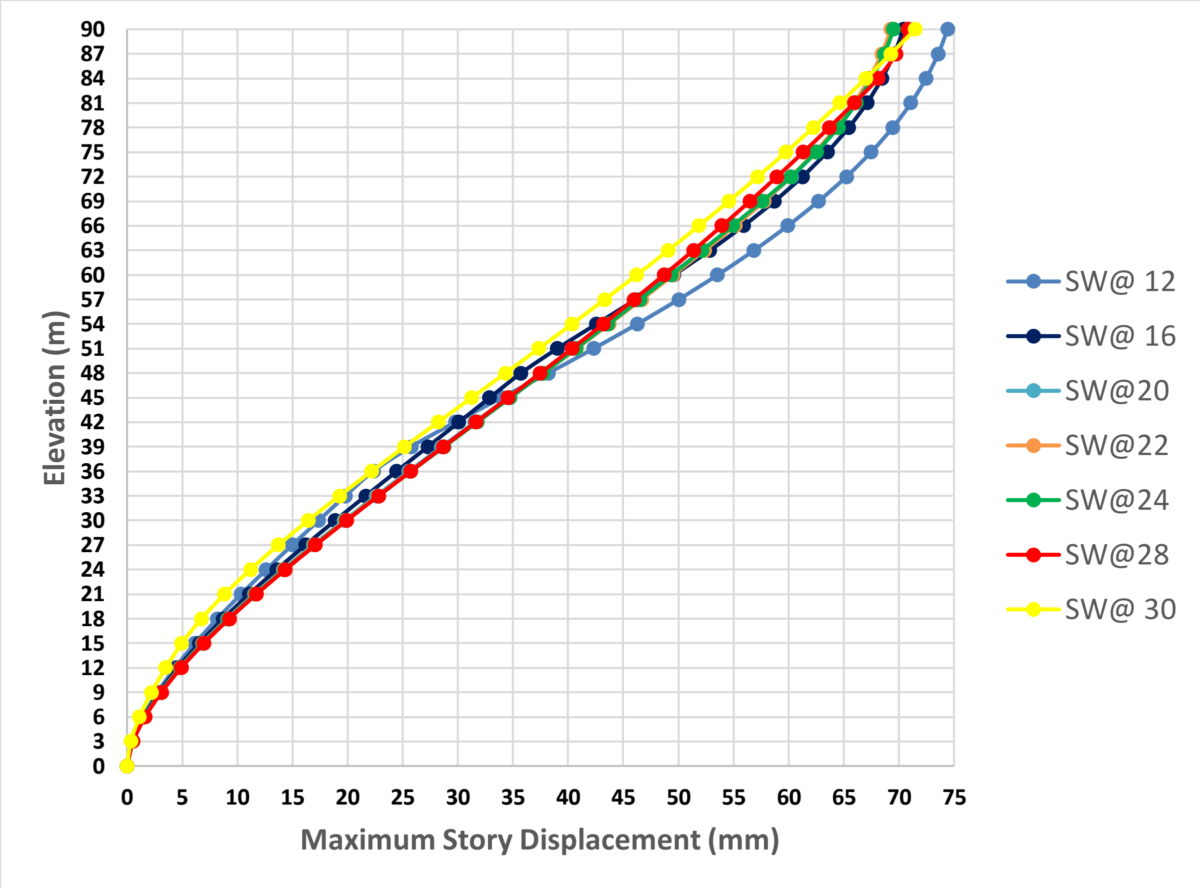


Fig 4.11 Story Displacement of the G+30 building models

Table 4.18 Story Displacement of the G+30 building models

S.L	Elevation	Maximum Story Displacement							
	m	Mm							
	X	SW@ 12	SW@ 16	SW@20	SW@22	SW@24	SW@ 26	SW@28	SW@ 30
30	90	74.417	**70.419**	**69.35544**	**69.285**	**69.531**	**70.053**	**70.845**	71.484
29	87	73.567	69.573	68.50712	68.431	68.666	69.158	69.738	69.264
28	84	72.461	68.468	67.39832	67.316	67.537	67.969	68.101	66.986
27	81	71.093	67.102	66.03432	65.947	66.151	66.458	65.945	64.645
26	78	69.436	65.446	64.38696	64.303	64.482	64.508	63.668	62.237
25	75	67.484	63.494	62.45712	62.386	62.512	62.106	61.338	59.757
24	72	65.241	61.246	60.25096	60.209	60.208	59.597	58.944	57.201

Table 4.19 Change in story Displacement w.r.t SW@ 30 (in %)

Elevation	Percent change in story displacement w.r.t SW@ 30						
m	Mm						
x	SW@ 12	SW@ 16	SW@20	SW@22	SW@24	SW@ 26	SW@28
72	12.3229	6.6048	5.0621	4.9955	4.9941	4.0205	2.9573
75	11.4497	4.04147	2.44878	2.33733	2.53393	1.89727	0.66856
78	10.3683	3.4812	1.89293	1.76538	2.03753	2.07629	0.78507
81	9.0703	2.62944	1.05545	0.92474	1.23051	1.68565	0.92077
84	7.5557	1.49478	-0.0679	-0.1909	0.13812	0.77294	0.96527
87	5.8492	0.11384	-1.44	-1.5535	-1.206	-0.4848	0.3508
90	3.9415	**-1.5118**	**-3.0690**	**-3.1737**	**-2.8081**	**-2.0423**	**-0.9015**

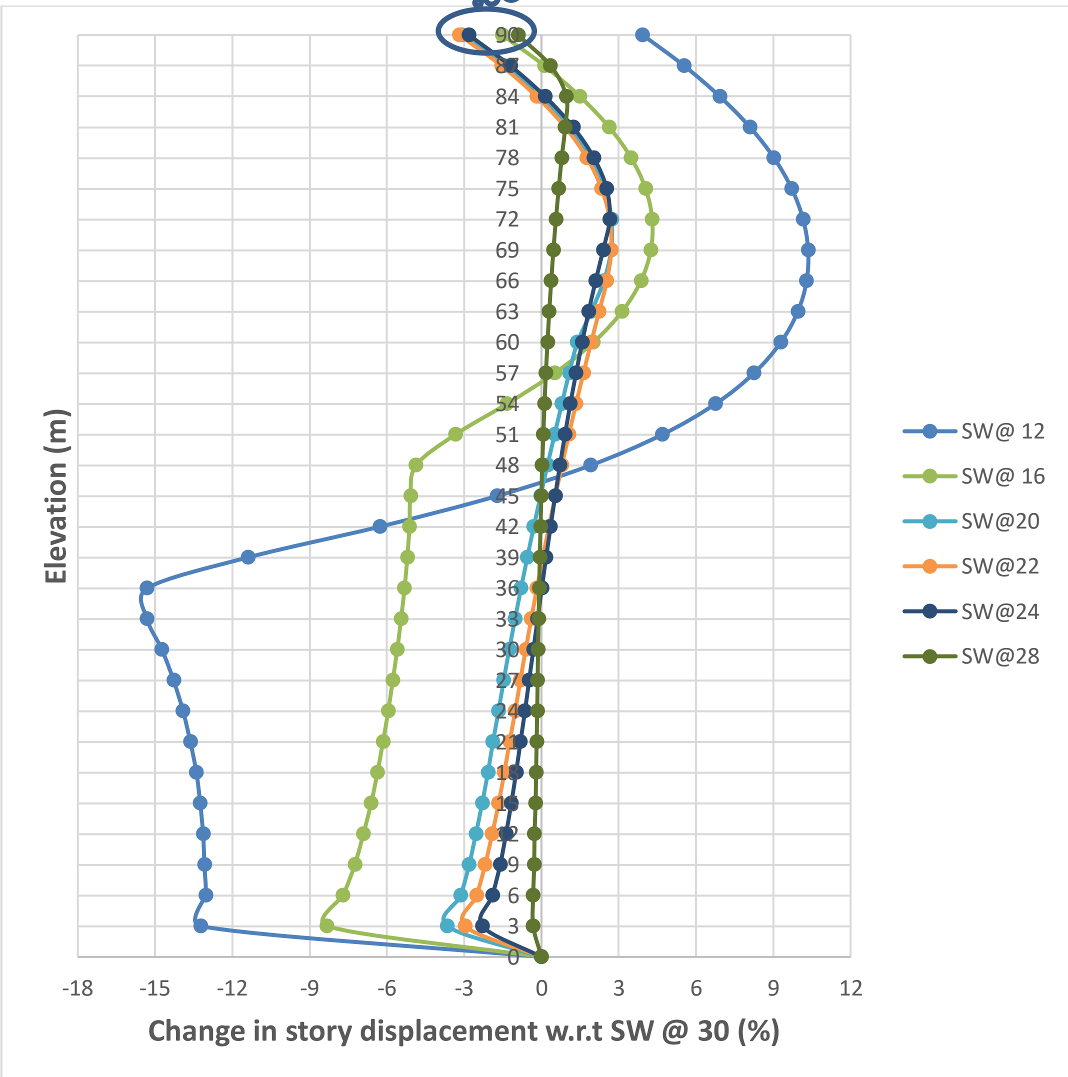


Fig 4.12 Change in story Displacement w.r.t SW@ 30 (in %)

Story Drift

From the story drift shown in the figure below we can see that of all the models (ignoring the maximum story drift of SW@8-SW@12 since they are below the inflection point) the maximum story drift is 0.0012 for SW@20 at the elevation near to 66m. The change in maximum inter story drift is 39.5%, 37.03%, 35.8%, 35% in SW@22, SW@24, SW@26 and SW@28 respectively as compared with the full height SW@20. So generally, from the results we can see that the change in maximum inter story drift increases while the curtailment level decreases. Analysis result reveals that after the maximum inter-story drift its value decreases with increasing height.

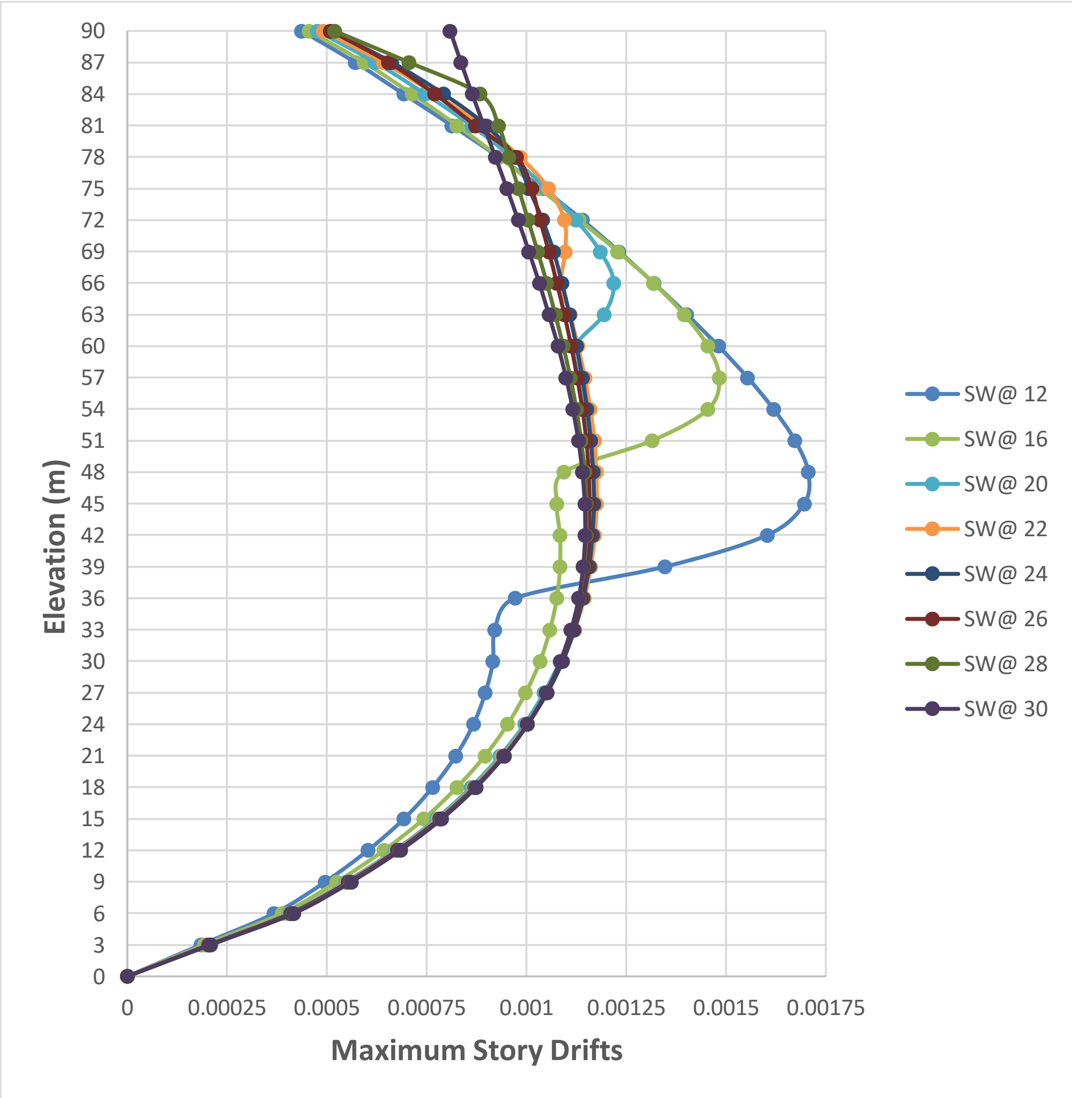


Fig 4.13 Story Drift of the G+30 building models

Table 4.20 Story Drift of the G+30 building models

Elev.	Maximum Story Drifts							
m								
X	SW@ 12	SW@ 16	SW@ 20	SW@ 22	SW@ 24	SW@ 26	SW@ 28	SW@ 30
90	0.0004	0.0005	0.0005	0.0005	0.0005	0.0005	0.0005	0.0008
87	0.0006	0.0006	0.0006	0.0006	0.0007	0.0007	0.0007	0.0008
84	0.0007	0.0007	0.0007	0.0008	0.0008	0.0008	0.0009	0.0009
81	0.0008	0.0008	0.0009	0.0009	0.0009	0.0009	0.0009	0.0009
78	0.0009	0.0009	0.0010	0.0010	0.0010	0.0010	0.0010	0.0009
75	0.0010	0.0010	0.0010	0.0011	0.0010	0.0010	0.0010	0.0010
72	0.0011	0.0011	0.0011	0.0011	0.0010	0.0010	0.0010	0.0010
69	0.0012	0.0012	0.0012	0.0011	0.0011	0.0011	0.0010	0.0010
66	0.0013	0.0013	0.0012	0.0011	0.0011	0.0011	0.0011	0.0010

Generally, 70 - 80% of the total building height that are found above the recommended contra-flexural level (60%) have proven to be effective in reducing and maintaining the least top story (roof) displacement with respect to that of the full height shear wall-frame models.

The curtailment of shear wall above 9th, 14th and 16th floors of the G+10, G+20 and G+30 building model respectively shows a reduced or a very small increment in top story (roof) displacement (i.e., +3.73%, -0.22% and -1.52% top story (roof) displacement difference with respect to that of the full height shear wall-frame models in the respective story height considered). Consequently, we can see that, when building height increases, the stiffer the frames will be and the lesser the contribution of the shear wall become. So, if we curtail the shear wall for the increased building height between 53.33% - 60% of the full height the curtailment become safe.

4.3 Comparison of the Continuum Analysis and FEA Results

The accuracy of the proposed continuum analysis and FEA method has been investigated by considering different structural arrangements and geometrical and stiffness characteristics (A. Carpinteri, M. Corrado, G. Lacidogna and S. Cammarano, 2012). For geometrical characteristics, a single thin-walled cantilever beam of total height H, having closed square cross-section with side L and thickness s, is considered. The investigated variables are the overall slenderness, defined by the ratio H/L, and the cross-section slenderness, defined as s/L. The comparisons are carried out in terms of displacements of the top floor (horizontal displacement). The results for horizontal forces applied to the center of each floor are shown in Fig. 4.14. The relative error is defined as $(\Delta_{an} - \Delta_{FE})/\Delta_{an}$, where the subscript "an" stands for "analytical method" and FE for Finite element method.

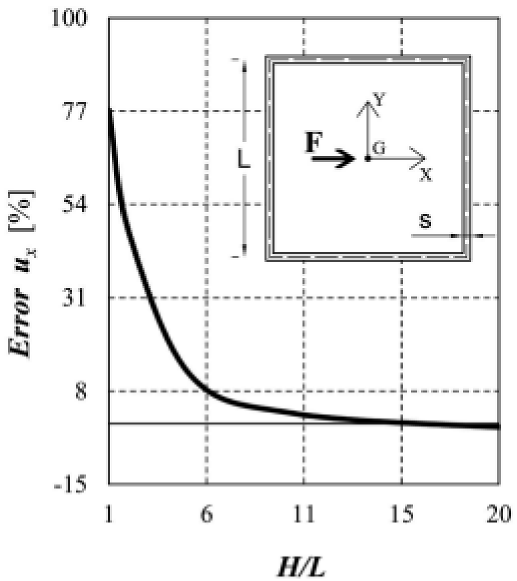


Fig 4.14 Relative error for the top (roof) displacement between the continuum method and FE analysis

For the top (roof) displacement, the relative error is about 80% for very short shear walls – frame systems, due to the large shear deformations disregarded by the analytical model, whereas it decreases up to zero for very slender dual systems. Also in this case, the cross-section slenderness ratio (s/L) has a limited influence on the comparison. The same results are obtained for the three considered values of s/L (1:100, 1:50 and 1:20).

i. For the G+10 building model

For the G+10 building, the total height H = 30m and length of the shear wall is L = 5m, so, the overall slenderness, defined by the ratio H/L is 6. Then according to A. Carpinteri, M. Corrado, G. Lacidogna and S. Cammarano, the error can possibly reach up to 8% but in our case the error is 4.81% which is within the allowable range. Similarly, the errors above 90% of curtailment level show an acceptable result (i.e., the overall slenderness ratio H/L is 27m/5m is 5.4, which allows more than 8% error, while our result show 5.71%).

Table 4.21 Comparison between FEA Vs M.J. Nollet's Continuum Analysis Method of the G+10 building

Curtailment level	FEA Results	Continuum Analysis Results	Δ (%)
SW@10	14.321	15.01	4.81
SW@ 9	15.198	14.33	5.71
SW@ 8	15.907	17.08	7.37
SW@ 7	16.817	19.18	14.05
SW@ 6	17.849	20.66	15.75
SW@ 5	39.843	21.58	45.84

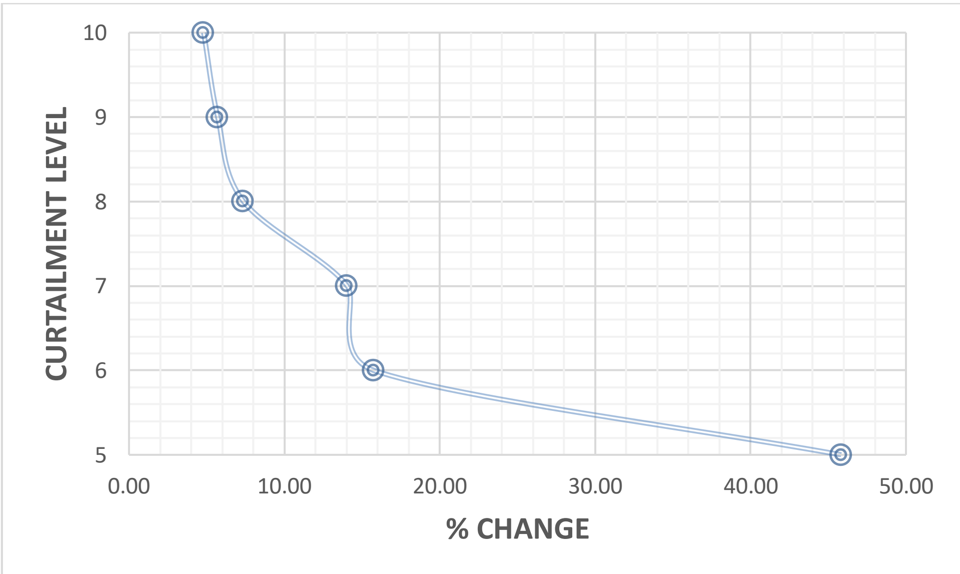


Fig 4.15 Percent change in top story displacement of FEA and M.J. Nollet's Continuum Analysis Method of the G+10 building

ii. For the G+20 building model

For the G+20 building, the total height H = 60m and length of the shear wall is L = 5m, so, the overall slenderness, defined by the ratio H/L is 12. Then according to fig 4.14, the allowable error can possibly reach up to approximately 1.0% which is near to the result in table 4.22 for SW@20. For the rest of the varying curtailment levels also the errors are approximately with in the allowable range.

Table 4.22 Comparison between FEA Vs M.J. Nollet's Continuum Analysis Method of the G+20 building

Curtailment level	FEA Results	Continuum Analysis Results	Δ (%)
SW@20	37.653	38.01	0.95
SW@ 18	37.158	37.71	1.47
SW@ 16	37.042	36.45	1.60
SW@ 14	37.991	37.28	1.86
SW@ 12	38.592	40.36	4.57
SW@ 10	40.159	45.82	14.10
SW@ 8	42.792	53.86	25.86

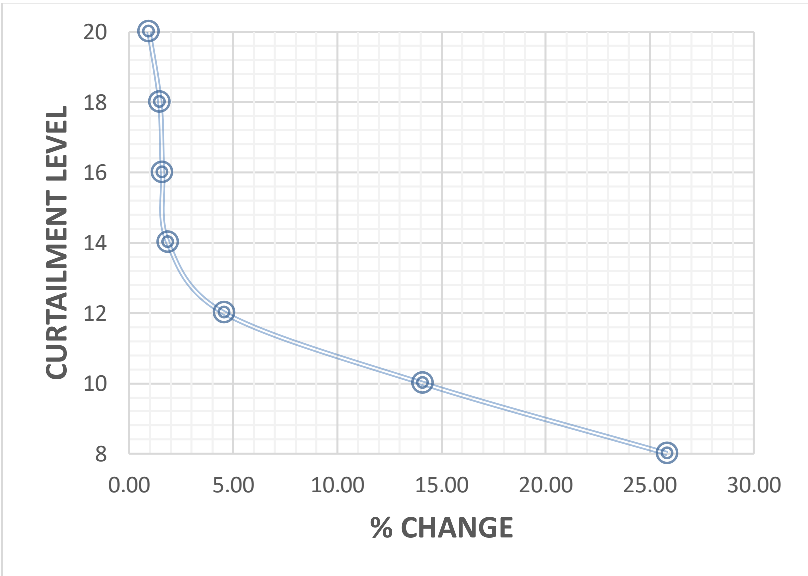


Fig 4.16 Percent change in top story displacement of FEA and M.J. Nollet's Continuum Analysis Method of the G+20 building

iii. For the G+30 building model

For the G+30 building, the total height H = 90m and length of the shear wall is L = 5m, so, the overall slenderness, defined by the ratio H/L is 18. Then accordingly, the allowable error is almost zero but the result in table 4.23 for SW@30 which is -0.17 is a bit beyond the allowable limit. But at lower curtailment levels the error enter with in the range.

Table 4.23 Comparison between FEA Vs M.J. Nollet's Continuum Analysis Method of the G+30 building

Curtailment level	Finite Element Analysis Results	Continuum Analysis Results	Δ (in %)
SW@30	71.48	71.85	0.52
SW@ 28	70.85	71.37	0.73
SW@ 26	70.05	69.21	1.21
SW@ 24	69.53	66.40	4.50
SW@ 22	69.29	62.62	9.62
SW@ 20	69.36	57.87	16.56
SW@ 18	69.75	52.18	25.19
SW@ 16	70.42	45.53	35.34
SW@ 14	71.97	38.18	46.95
SW@ 12	74.42	30.27	59.33

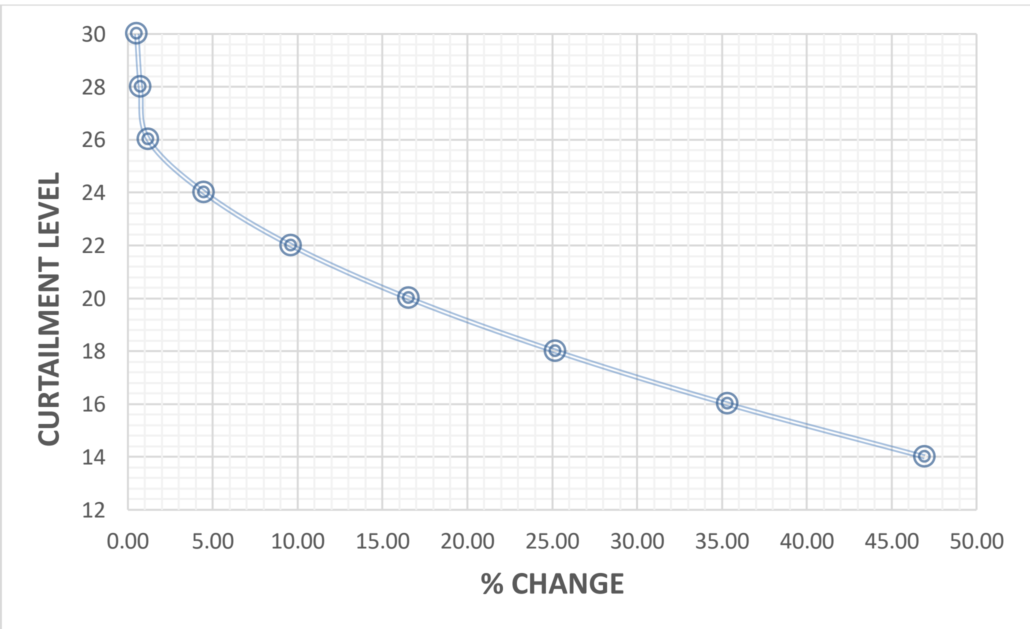


Fig 4.17 Percent change in top story displacement of FEA and M.J. Nollet's Continuum Analysis Method of the G+30 building

The Alex Coull's continuum analysis method can also be compared with that of the finite analysis method and checked if the errors are within the allowable range provided in fig 4.14. For the G+10 building, the total height H = 30m and length of the shear wall is L = 5m, so, the overall slenderness ratio H/L is 6. Then according to fig 4.14, the allowable error can possibly reach up to approximately 8%. So, we can see that the top (roof) displacement result obtained by Alex Coull's (i.e., -0.04%) is within the allowable range.

Table 4.24 Comparison between FEA Vs A. Coull's Continuum Analysis Method of the G+10 building

Story Level	Continuum Analysis Results	FE Analysis Results	Δ (in %)
10	14.89	14.321	-0.040
9	11.36	12.737	0.108
8	8.44	11.072	0.238
7	6.07	9.347	0.351
6	4.17	7.611	0.452
5	2.70	5.907	0.543
4	1.60	4.327	0.630
3	0.83	2.859	0.710
2	0.34	1.579	0.785
1	0.08	0.557	0.856
0	0.00	0	0

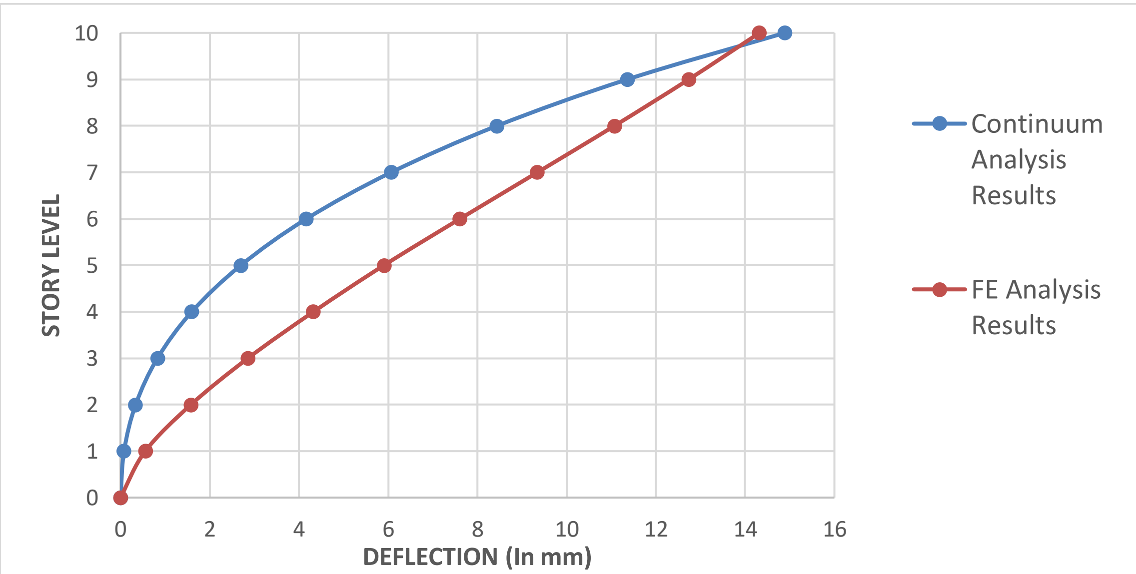


Fig 4.18 Change in top story displacement of FEA and A. Coull's Continuum Analysis Method of the G+10 building

- Generally, as we can see from analysis results of all the cases i.e., the G+10, G+20, G+30 buildings, percentile change in maximum top story displacement computed by finite element analysis method and that of continuum analysis method, decrease as the story height increase for each of the models considered. Moreover, from the three cases considered the maximum story displacement of the G+30 model which is 0.52% is less than that of 0.95% for the G+20 model which again is less than 4.81% of the G+10 model.

4.4 Comparison of FEA Results for Different Ground Accelerations

As discussed on the third chapter, ground acceleration ratio, ag = 0.1g, 0.15g and 0.2g were used during defining the response spectrum function. So, the results shown below try to prove that the curtailment levels which reveal relatively better result are also applicable for the different ground acceleration ratios (ag = 0.1g, 0.15g and 0.2g) on the respective models and sub-models investigated above.

i. For the G+10 building model

The change in lateral displacement of building at the top story in contrast to SW@10 becomes 5.77% and 9.97% in SW@9 and SW@8 respectively for ground acceleration ratios ag = 0.1g and around 8% and 10.5% in SW@9 and SW@8 respectively for both ground acceleration ratios 0.15g and 0.2g. So, curtailment of shear wall at 90% of building height shows least displacement as compared to full height of building for all the three ground acceleration ratios.

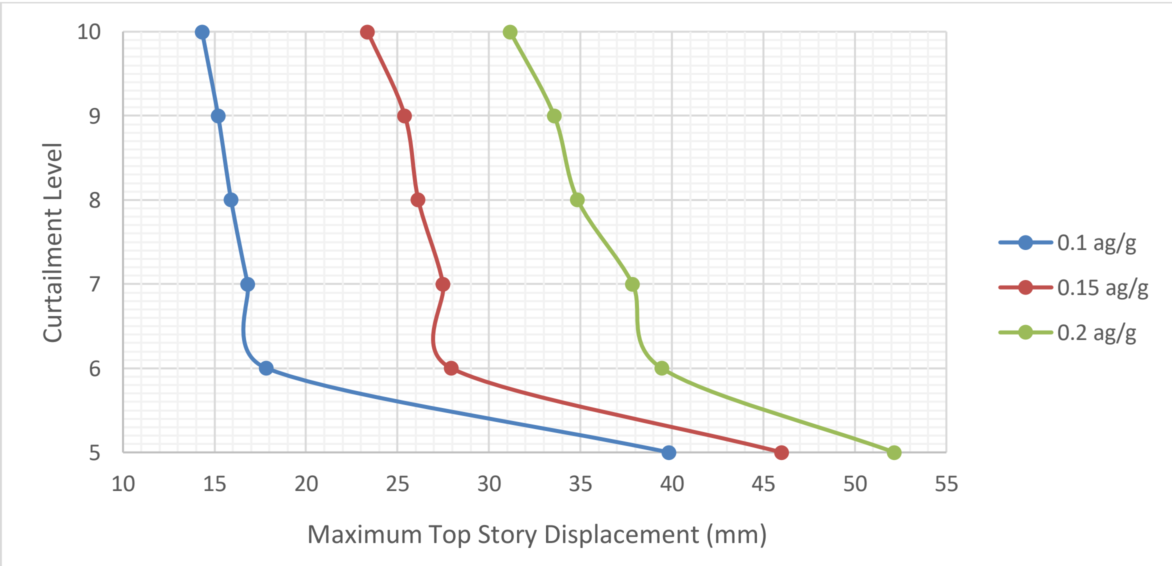


Fig 4.19 Change in top story displacement of FEA for different ground accelerations of the G+10 building

Table 4.25 Comparison of FEA Results for different ground accelerations of the G+10 building

ag/g	Maximum Top Story Displacement					
	5	6	7	8	9	10
0.1	39.843	17.849	16.817	15.907	15.198	14.321
0.15	46.006	27.946	27.493	26.141	25.407	23.372
0.2	52.169	39.451	37.853	34.855	33.572	31.163

ii. For the G+20 building model

Similarly, the results of the G+20 model prove that the curtailment of shear wall at 14th floor show 0.89% variation with respect to the full height model (SW@20) for all of the three ground acceleration ratios.

Table 4.26 Comparison of FEA Results for different ground accelerations of the G+20 building

ag/g	Maximum Top Story Displacement		Change (%)
	SW@14	SW@20	
0.1 ag/g	37.991	37.653	0.8896844
0.15 ag/g	56.986	56.479	0.8896922
0.2 ag/g	75.981	75.305	0.8896961

iii. For the G+30 building model

The results of the G+30 model also prove that SW@22 show 3.08% variation with respect to the SW@30 for all of the three ground acceleration ratios.

Table 4.27 Comparison of FEA Results for different ground accelerations of the G+30 building

ag/g	Maximum Top Story Displacement		Change (%)
	SW@22	SW@30	
0.1 ag/g	69.284	71.484	3.0776118
0.15 ag/g	103.926	107.225	3.0767079
0.2 ag/g	138.568	142.967	3.0769338

CHAPTER 5: CONCLUSION AND RECOMMENDATION

5.1 CONCLUSION

In this thesis work, the effect of shear wall curtailment for medium and high-rise buildings in seismic region is studied using both continuum methods and finite element method of analysis. The effectiveness of wall-frame structure depends first on the amount of horizontal interaction which is governed by the relative stiffness of the walls and frames and second on height of the structure. While accounting for the wall and frame interaction it would even become safer if we curtail the wall above the contra flexural point. So, in order to prove that different building models having variable story height and also with varying shear wall curtailment level are considered. After computations of the different parameters and comparison of the results the two methods of analysis for all models and sub-models, the following conclusions are drawn:

- The curtailment of shear wall at 8th, 14th and 22nd floors of the G+10, G+20 and G+30 building model respectively (i.e., 70 - 80% of the total building height that are found above the recommended contra-flexural level (60%) have proven to be effective in reducing and maintaining the least top story (roof) displacement with respect to that of the full height shear wall-frame models.
- The change in the maximum story drift is very small up to the point of the recommended contra-flexural level (60% of the total building height), but increase significantly above that level. Specially, the story drift above 70% of the total building height of all the G+10, G+20 and G+30 building models give better result compared to that of the full height models. in addition, all the story drift values above 60% height are within the code specified story drift deformation limit.
- The models also displayed acceptable performance in terms of story overturning moment, story shear and story stiffness when the shear wall were curtailed up to 60% of the height of the original full height structure. The story stiffness reveals less than 2% difference for the 90% curtailment height of all the G+10, G+20 and G+30 building models.
- For all the parameters considered in the FEA, the respective values show highly deviated results below the contra flexural point i.e., below 50 - 60% of the height of all the G+10, G+20 and G+30 building models.
- The percentile changes in maximum top (roof) story displacement computed by finite element analysis method and that of continuum analysis method shows less than 5% difference for the respective G+10, G+20 and G+30 building models. So that the

maximum top (roof) story displacement results obtained from the FEA was cross checked with the continuum analysis results.

- The results of FEA for the different ground acceleration ratios (ag = 0.1g, 0.15g and 0.2g) also prove that the curtailment at 8^{th}, 14^{th} and 22^{nd} floors of the G+10, G+20 and G+30 building model respectively reveal the least top story displacement with respect to that of the full height shear wall-frame models.

Finally, all the above findings supported our initial hypothesis that the curtailment of the shear walls in the top floors is not necessarily detrimental to the performance of the structure even if ES-EN 1998: 2015 recommend shear walls to continue up to the roof level for the sake of regularity. Additionally, when building height increases, the stiffer the frames will be, the greater the interaction become between wall and frame and the lesser the contribution of the shear wall. So, if we curtail the shear wall for the increased building height (high rise), since the shear wall contribution is minimized, the curtailment become safe.

5.2 RECOMMENDATION

Recommendations and suggestions for future studies can be:

- All the models and sub-models considered in this study are regular in plan and are mainly plan-symmetric structures subjected to symmetric loading, so future works can consider structures that are asymmetric about the axis of loading which inevitably subjected to a torsion.
- Due to the unavailability of testing machines (cyclic loading testing machine) and economic constraints, experimental investigation required for this particular study is not covered. So, future studies can reinforce the investigation by including experimental investigation.
- The two main factors that cause lateral deformation of a building are the racking shear and the axial deformation of each and every column. But, in this paper Alex Coull's method don't consider the axial deformation of the columns which can be included in future works.
- Future studies can further investigate effect of curtailment by considering openings in the shear walls for all sub-models considered.

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