OPTIMUM LOCATION OF CURTAILED SHEAR WALLS IN SMRF

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CERTIFICATE

This is to certify that the project report entitled **"Optimum Location of Curtailed Shear Walls in SMRF",** is submitted by **Swasti Saxena (111710)** in partial fulfilment for the award of degree of Bachelor of Technology in Civil Engineering from Jaypee University of Information Technology, Waknaghat, Solan, and has been carried out under my supervision.

This work has not been submitted partially or fully to any other University or Institute for the award of this or any other degree or diploma.

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ABSTRACT

Earthquake is a force of nature that can neither be precluded nor predicted before its mechanism is already set in motion. What man can do is mitigate the damage and minimize casualties when earthquakes do hit the surface. The central idea of Earthquake Engineering is to first understand the origin of seismic waves that are causative of the destruction and havoc and then learn various methods and procedures that can enable structures to withstand earthquake forces and maintain operation after the natural calamity has passed.

Reinforced cement concrete (RCC) framed structures combined with shear walls have been widely used to resist lateral forces caused by wind load and earthquakes in tall buildings. Shear walls impart greater strength as well stiffness in their own plane and hence are generally provided for full height of the frames. Under lateral load, the shear wall deflects essentially in flexural shape and the frame deflects in shear shape. For this reason, these components are forced to interact horizontally through the floor slabs. Consequently, the upper part of the shear wall could play a negative role and may lead to unreasonable design by introducing additional internal forces to the system. Hence arises the need to curtail shear walls to reduce these forces, so that the advantages of shear walls can be reaped without their inconveniences.

The scope of the current work was to determine optimum location of RC shear walls in two models, plan 1 - 7x3 bays and plan 2 - 9x3 bays respectively, for 20 storey RCC-framed buildings, with shear walls being curtailed at various heights. Response spectrum analysis has been performed using standard package SAP2000, along with P-delta effects to more precisely capture the behaviour of the dual system. A comparison of different parameters like deflection and shear has been made and a polynomial expression deduced using Regression Analysis in MS Excel. The expression not only explains the current behaviour, but can even be used to roughly predict the behaviour of taller buildings.

1. EARTHQUAKE RESISTANT DESIGN OF RCC BUILDINGS

1.1. INTRODUCTION

Earthquakes are, fundamentally, vibrations caused by seismic waves. Tectonic plates in the lithosphere layer of earth float due to convection currents which cause faults in the crust of the earth and the subsequent release of energy (according to Elastic Rebound Theory) is the source of seismic waves which cause earthquakes.

Key responses to different intensities of earthquakes vary from imperceptible shaking, to awakening of households, to movement of furniture, to damage to chimneys, to catastrophic destruction like building shifted off foundations, ground cracked conspicuously underground pipes broken, rails bent, bridges destroyed etc.

In some countries, greater importance to the community of some types of facility is recognized by regulatory requirements, like various public buildings are designed for higher earthquake forces than other buildings. Some of the most vital facilities to remain functional after destructive earthquakes are dams, hospitals, fire and police stations, government offices, bridges, radio and telephone services, schools, energy sources, or, in short, anything vitally concerned with preventing major loss of life in the first instance and with the operation of emergency services afterwards.

Most countries have various codal provisions to govern earthquake resistant design of structures. Although no standard can preclude damage during earthquake of all magnitudes, endeavours are made to ensure that, as far as possible, structures are able to respond, without structural damage to shocks of moderate intensities and without total collapse to shocks of heavy intensities.

Four procedures are presented for seismic analysis of buildings: two linear procedures, and two nonlinear procedures. The two linear procedures are termed the Linear Static Procedure (LSP) and the Linear Dynamic Procedure (LDP). The two nonlinear procedures are termed the Nonlinear Static Procedure (NSP) and Nonlinear Dynamic Procedure (NDP).

1.2 SEISMIC DESIGN PHILOSOPHY

Construction of an earthquake resistant building is a costly business for the need of larger sections of structural members, extra ductility and/or additional bracings. Hence the question arises whether design of buildings for all earthquakes must be done away with altogether or to make them earthquake resistant for the greatest intensity but rarest of earthquakes. While the first option is capable of worst imaginable destruction, the second option is wildly uneconomic. So as a negotiation, following is the summary of an earthquake design philosophy that is the objective of earthquake codes worldwide:

i) Under minor but frequent shaking, the vertical and horizontal load carrying members should not be damaged; however building parts that do not carry load may sustain repairable damage.
ii) Under moderate but occasional shaking, the main members may sustain repairable damage, while the other parts of the building may be damaged enough to require replacement after the earthquake; and
iii) Under strong but rare shaking, the main members may sustain severe (even irreparable) damage, but the building should not collapse.

Such a design philosophy is best supported by the latest prodigy of seismic engineering, i.e. Performance based seismic engineering, or Performance based seismic design. The promise of performance-based seismic engineering (PBSE) is to produce structures with predictable seismic performance.

Work by the Structural Engineers Association of California (SEAOC) resulted in the publication of tentative design guidelines on this subject. Most basic stage in this design process is the selection of performance objectives. A performance objective is a coupling of the expected performance level with expected levels of earthquake ground motions.

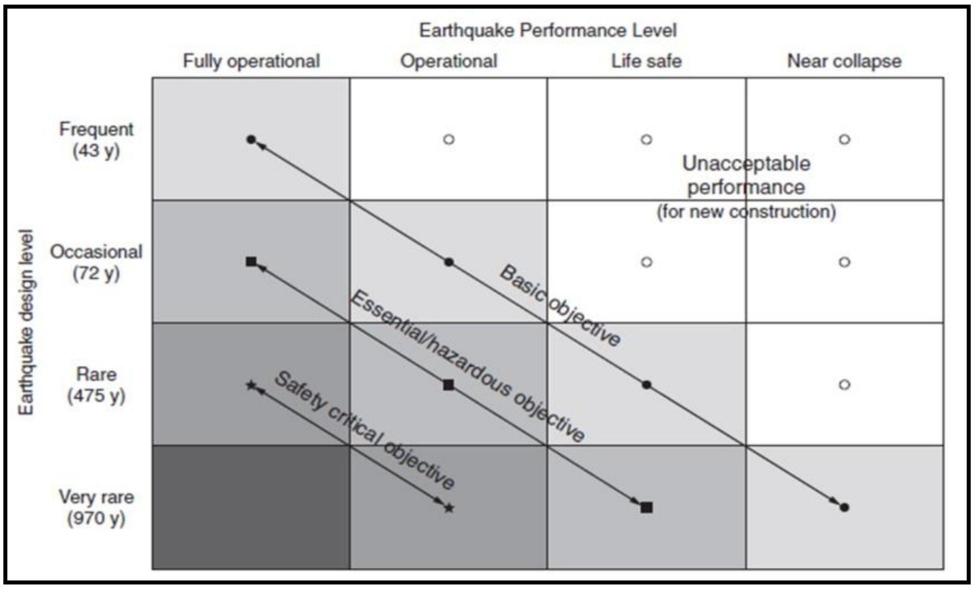


Figure 1: Seismic performance objectives for buildings recommended by SEAOC (1999)

From Figure1 it can be seen that less damage extent is allowed for an earthquake of high probability or for a structure of significant importance (like hospitals or power plants). On the other hand, more damage is acceptable for severe earthquakes of rare occurrence or for a less critical or temporary facilities. Accordingly, a building would be expected to suffer more damage if it were subjected to a more severe, less likely earthquake. Also, a more critical building would be expected to have less damage for the same earthquake probability.

In order to make a comprehensive understanding of Seismic performance objectives for buildings recommended by SEAOC (1999), let us first review various criteria that demarcate Occupancy Classifications, Performance States, Earthquake Classification, and Limit State Damage to steel frames (Table 1 and Table 2). There are four performance (i.e. limit) states and three occupancy types:

- *Safety Critical Facilities*

  - Large quantities of hazardous materials such as toxins, radioactive materials, or explosives with significant external effects of damage to building
- *Essential/Hazardous Facilities*

  - Critical post-earthquake facilities such as hospitals, communications centers, police, fire stations, etc

  - Hazardous materials with limited impact outside of immediate vicinity of building such as refineries, etc
- *Basic Facilities*

  - All other structures

Table 1: Description of damage for the four performance (i.e. limit) states

Limit State	Damage Description
Fully operational	Negligible
Operational	Minor local yielding at a few places. No observable fractures. Minor buckling or observable permanent distortion of members.
Life Safe	Hinges form. Local buckling of some beam elements. Severe joint distortion. Isolated connection fractures. A few elements may experience fracture.
Near Collapse	Extensive distortion of beams and column panels. Many fractures in connections.

Table 2: Description of functions of buildings for the four performance (i.e. limit) states

Limit State	Damage Description Damage Description
Fully operational	Continuous service. Negligible structural and non-structural damage.
Operational	Most operations and functions can resume immediately. Structure safe for occupancy. Essential operations protected, non-essential operations disrupted. Repair required to restore some non-essential services. Damage is light.
Life Safe	Damage is moderate, but structure remains stable. Selected building systems, features, or contents may be protected from damage. Life safety is generally protected. Building may be evacuated following earthquake. Repair possible, but may be economically impractical.
Near Collapse	Damage severe, but structural collapse prevented. Nonstructural elements may fall. Repair generally not possible.

This type of design philosophy strikes a perfect bargain between an economical yet safe structure under severe but rare earthquakes, and results in least damaged in case of frequent earthquakes. The earthquake intensity is described quantitatively in probabilistic terms in Table 3.

Table 3: Classification of earthquakes based on their probability of occurrence

Earthquake Classification	Recurrence Interval	Probability of Occurrence
Frequent	43 years	50% in 30 years
Occasional	72 years	50% in 50 years
Rare	475 years	10% in 50 years
Very Rare	970 years*	10% in 100 years

* need not exceed mean + 1 standard deviation for the maximum deterministic event

Above is the basic deign philosophy of performance based design, which allows one to decipher the level of rigor with which a building must be reinforced with seismic reisistance. Let us explore performance based design in detail as described by Federal Emergency Management Authority (FEMA) 356, which clearly identifies the isolated Building Performance Levels.

1.2.1. Target Building Performance Levels

Building performance is a combination of the performance of both structural and non-structural components. Building performance in FEMA 356 is expressed in terms of target Building Performance Levels. These target Building Performance Levels are discrete damage states selected from among the infinite spectrum of possible damage states that buildings could experience during an earthquake. The particular damage states identified as target Building Performance Levels in this standard have been selected because they have readily identifiable consequences associated with the post-earthquake disposition of the building that are meaningful to the building community. These include the ability to resume normal functions within the building, the advisability of post-earthquake occupancy, and the risk to life safety.

1.2.2. Structural Performance Levels and Ranges

The Structural Performance Level of a building shall be selected from four discrete Structural Performance Levels and two intermediate Structural Performance Ranges defined in this section (Figure 2).

The discrete Structural Performance Levels are Immediate Occupancy (S-1), Life Safety (S-3), Collapse Prevention (S-5), and Not Considered (S-6). The intermediate Structural Performance Ranges are the Damage Control Range (S-2) and the Limited Safety Range (S-4). Acceptance criteria for performance within the Damage Control Structural Performance Range shall be obtained by interpolating the acceptance criteria provided for the Immediate Occupancy and Life Safety Structural Performance Levels. Acceptance criteria for performance within the Limited Safety Structural Performance Range shall be obtained by interpolating the acceptance criteria provided for the Life Safety and Collapse Prevention Structural Performance Levels.

a) *Immediate Occupancy Structural Performance Level (S-1)*

Structural Performance Level S-1, Immediate Occupancy, means the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate, these would generally not be required prior to re-occupancy.

b) *Damage Control Structural Performance Range (S-2)*

Design for the Damage Control Structural Performance Range may be desirable to minimize repair time and operation interruption, as a partial means of protecting valuable equipment and contents, or to preserve important historic features when the cost of design for immediate occupancy is excessive.

c) Life Safety Structural Performance Level (S-3)

Structural Performance Level S-3, Life Safety, means the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, the overall risk of life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to re-occupancy.

d) Limited Safety Structural Performance Range (S-4)

Structural Performance Range S-4, Limited Safety, shall be defined as the continuous range of damage states between the Life Safety Structural Performance Level (S-3) and the Collapse Prevention Structural Performance Level (S-5).

e) Collapse Prevention Structural Performance Level (S-5)

Structural Performance Level S-5, Collapse Prevention, means the post-earthquake damage state in which the building is on the verge of partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral force resisting system, large permanent lateral deformation of the structure, and—to a more limited extent— degradation in vertical-load-carrying capacity. However, all significant components of the gravity load- resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for re-occupancy, as aftershock activity could induce collapse.

f) Structural Performance Not Considered (S-6)

Some owners may desire to address certain non-structural vulnerabilities in a rehabilitation program—for example, bracing parapets, or anchoring hazardous materials storage containers—without addressing the performance of the structure itself. Such rehabilitation programs are sometimes attractive because they can permit a significant reduction in seismic risk at relatively low cost.

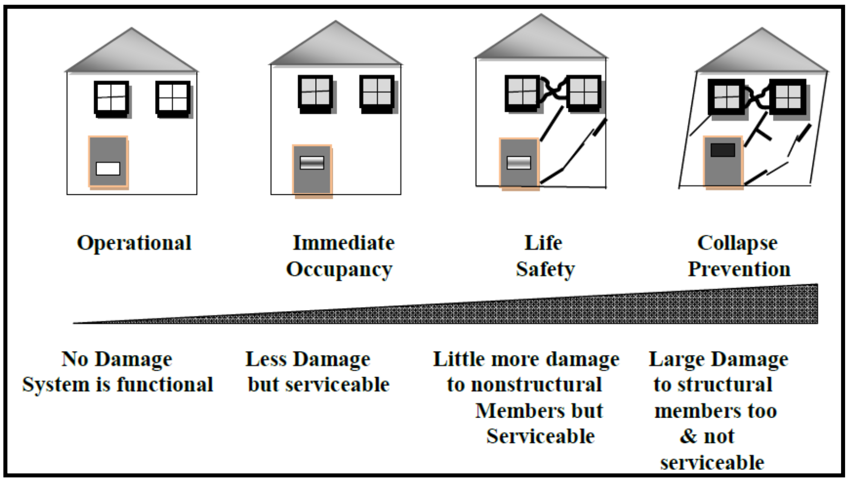


Figure 2: Structural Performance levels and damage Functions

1.3. PRINCIPLES OF RELIABLE SEISMIC BEHAVIOUR-FORM, MATERIAL AND FAILURE MODES

In seeking the optimum of the proposed, construction designers should choose forms and materials that give the best failure modes in earthquakes with functional and cost requirements.

1.3.1. FORM OF THE BUILDING

In order to achieve reliable earthquake resistance, the form of construction should be decided from the consideration of the following factors:

1) Simplicity and symmetry
2) Length in plan
3) Shape in elevation
4) Uniformity and Continuity
5) Stiffness
6) Failure modes
7) Foundation conditions

1.3.1.1. Simplicity and symmetry

Earthquakes repeatedly demonstrate that the simplest structures have the greatest chance of survival. There are three main reasons for this. First, our ability to understand the overall behaviour of a simple structure is markedly greater than it is for a complex one – for example, torsional effects are particularly hard to predict on an irregular structure. Secondly, our ability to understand simple structural details is considerably greater than it is for complicated ones. Thirdly, simple structures are likely to be more buildable than complex ones.

Symmetry is desirable for much the same reasons. It is worth pointing out that symmetry is important in both directions in plan, and helps in elevation as well. Lack of symmetry produces torsional effects which are sometimes difficult to assess, and can be very destructive. The introduction of deep re-entrant angles into the facades of buildings (Figure 3) introduces complexities into the analysis which makes them potentially less reliable than simple forms. Buildings of H-, L-, T- and Y-shape in plan have often been severely damaged in earthquakes.

1.3.1.2. Length in plan

Structures which are long in plan naturally experience greater variations in ground movement and soil conditions over their length than short ones (Figure 3). These variations may be due to out-of-phase effects or to differences in geological conditions, which are likely to be most pronounced along long bridges where depth to bedrock may change from zero to very large. The effects on structure will differ greatly, depending on whether the foundation structure is continuous, or a series of isolated footings, and whether the superstructure is continuous or not.

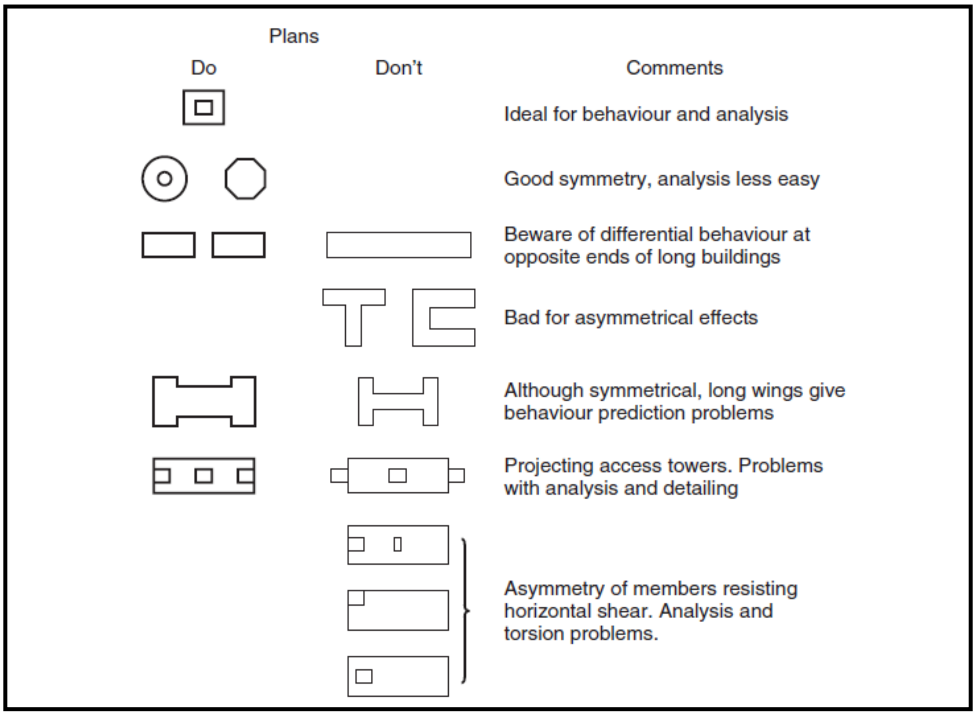


Figure 3: Simple rules for plan layouts

1.3.1.3. Shape in elevation

As indicated in Figure 4, very slender structures and those with sudden changes in width should be avoided in strong earthquake areas. Very slender buildings have high column forces, and foundation stability may be difficult to achieve. Also higher mode contributions may add significantly to the seismic response of the superstructure. Height–width ratios in excess of about 4 lead to less economical structures and require dynamic analysis for proper evaluation of seismic responses.

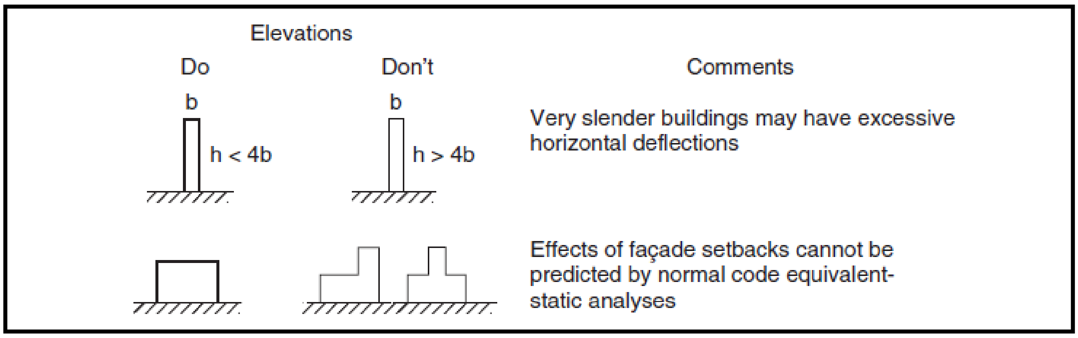


Figure 4: Simple rules for elevation shapes of seismic buildings

1.3.1.4. Uniform and continuous distribution of strength, stiffness and mass

This concept is closely related to that of simplicity and symmetry. The structure will have the maximum chance of surviving an earthquake if the following conditions are satisfied:

1) The load-bearing members are uniformly distributed.
2) All columns and walls are continuous and without offsets from roof to foundation.
3) All beams are free of offsets.
4) Columns and beams are coaxial.
5) Reinforced concrete columns and beams are nearly the same width.
6) No principal members change section suddenly.
7) The structure is as continuous (redundant) and monolithic as possible.
8) There are no irregular or asymmetric large concentrations of mass.

Sudden changes in lateral stiffness up a building are *not* wise (Figure 5), first because even with the most sophisticated and expensive computer analysis the earthquake stresses cannot be determined well, and secondly because the demands on effective structural detailing become very high. Severe damage and collapse of buildings with sudden big changes in vertical structure have occurred in many earthquakes. Sometimes such severe effects are caused by the failure of infill in framed structures, leading to the unintended creation of a soft (weak) storey.

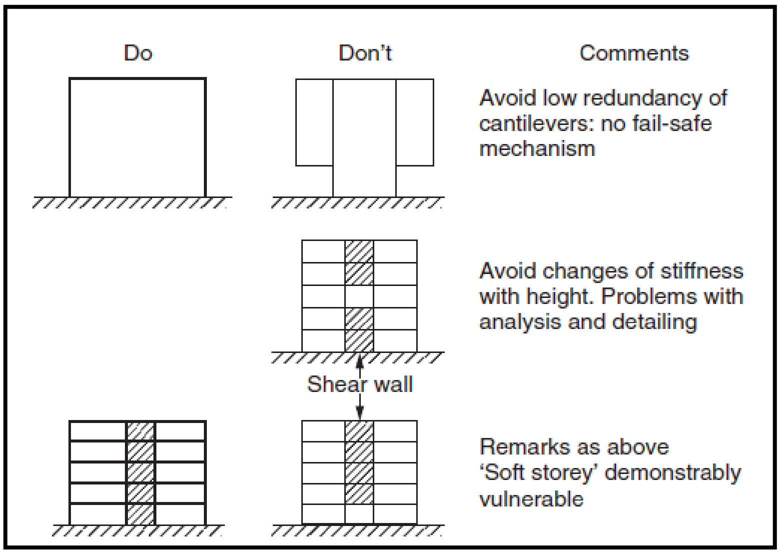


Figure 5: Simple rules for vertical frames in seismic buildings

The earthquake resistance of an economically designed structure depends on its capacity to absorb apparently excessive energy input, mainly in repeated plastic deformations of its members. Hence, the more continuous and monolithic a structure is made, the more plastic hinges and shear and thrust routes are available for energy dissipation.

1.3.1.5. Appropriate stiffness

The criteria for the stiffness of a structure fall into three categories, i.e. the stiffness is required:

1) to create desired vibrational characteristics of the structure (to reduce seismic response, or to suit equipment or function);
2) to control deformations (to protect structure, cladding, partitions, services);
3) to influence failure modes.

1.3.1.5.1. Stiffness to suit required vibrational characteristics

With regard to vibrational characteristics, we note first that it would be desirable in general to avoid resonance of the structure with the dominant period of the site as indicated by the peak in the response spectrum (Figure 6). This is particularly true for flexible longer-period structures, while shorter-period structures with ample structural walls can be made to work on any kind of site.
In the case of sites where the soil is soft and deep enough to amplify the lower frequencies, resonance with longer-period structures may occur, and high frequencies may be largely filtered out.

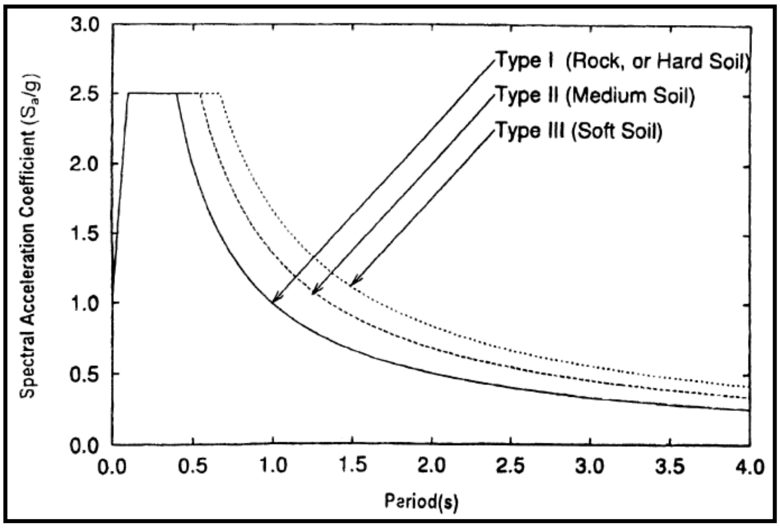


Figure 6: Response Spectra for rock and soil sites for 5% damping (IS 1893: 2002 Part 1)

1.3.1.5.2. Stiffness to control deformation

The stiffness levels required to control damaging interaction between structure, cladding, partitions and equipment vary widely, depending upon the nature of components and the function of the construction, but stiff construction is obviously better than flexible in this regard. The seismic deformations of conventional construction can be greatly reduced by the use of seismic isolation so that relatively flexible moment resisting frames may be able to satisfy the design deformation criteria, and *P*-delta column moments will be greatly reduced.

1.3.1.5.3. Stiffness affects failure modes

Different levels of stiffness can be created by such widely differing structural configurations such that wide differences in potential failure modes arise. In general, stiffer construction implies the existence of less favourable failure modes from an earthquake design point of view, and this needs special design attention.

1.3.1.5.4. Stiff structures versus flexible structures

Key distinctions between stiff structures and flexible structures are given in Table 4. To overcome the difficulties imposed by the deformability of more flexible construction over the years, there has been a trend to avoid using traditional moment resisting frames by various means such as shear walls (various forms), bracing (various forms), base isolation, and energy absorbing devices. These will:

• reduce lateral drift;

• reduce reinforced concrete joint detailing problems;

• help to ensure that plasticity develops uniformly over the structure;

• prevent column failure in sway due to the *P*-delta effect.

In conclusion, it can be said that in many situations either a stiff or a flexible structure can be made to work, but the advantages of the two forms need careful consideration when choosing between them.

Table 4: Comparative merits of stiff and flexible construction (not seismically isolated)

	Advantages	Disadvantages
Flexible structures	(1) Specially suitable for short-period sites, for buildings with long periods (2) Ductility arguably easier to achieve (3) Non-structure may invalidate analysis	(1) Higher response on long-period sites (2) Flexible framed reinforced concrete is difficult to reinforce (3) More amenable to analysis (4) Non-structure difficult to detail
Stiff structures	(1) Suitable for long-period sites (2) Easier to reinforce stiff reinforced concrete (i.e. with shear wall) (3) Non-structure easier to detail	(1) Higher response on short-period sites (2) Appropriate ductility not easy to knowingly achieve (3) Less amenable to analysis

1.3.2. CHOICE OF CONSTRUCTION MATERIALS

Purely in terms of earthquake resistance, the best materials have the following properties:

i. high ductility;

ii. high strength–weight ratio;

iii. homogeneity;

iv. orthotropy;

v. ease in making full strength connections.

Generally, the larger the structure, the more important the above properties are. The choice of construction material is important in relation to the desirable stiffness. It is worth bearing in mind while choosing materials that if a flexible structure is required then some materials, such as masonry, are not suitable. On the other hand, steelwork is used essentially to obtain flexible structures, although if greater stiffness is desired diagonal bracing or reinforced concrete shear panels may sometimes be incorporated into steel frames. Concrete, of course, can readily be used to achieve almost any degree of stiffness.

A word of warning should be given here about the effect of non-structural materials on the structural response of buildings. The non-structure, mainly in the form of partitions, may greatly stiffen an otherwise flexible structure and hence must be allowed for in the structural analysis.

1.3.3. FAILURE MODE CONTROL

Good design not only seeks to keep the overall probability of failure below a given level but also arranges the system such that less desirable modes of failure are less likely to happen than others. This increases the reliability of the design by decreasing the potential for damage and increasing the overall safety. The less desirable modes of failure for structures are:

i) those resulting in total collapse of the structure (notably through failure of vertical load-carrying members); and
ii) those involving sudden failure (e.g. brittle or buckling modes).

The above principle is particularly important for moderate to strong earthquake loading, because such loading generally involves stress incursions well into the post-elastic range in the parts of the structure. It is therefore highly desirable to control both the location and the manner of the post-elastic behaviour, i.e. to design for failure mode control. To reduce the probability of occurrence of failure modes (i) and (ii) above, earthquake codes commonly have requirements that give added strength

i) to vertical load-carrying elements and
ii) to members carrying significant shear or compressive loads.

Figure 7 illustrates alternative failure modes for a multi-storey moment resisting frame. Clearly, the column side-sway mechanism is less desirable than the beam side-sway mechanism, as the former will lead to earlier total collapse than the latter. However, while it is possible and desirable to design so that plastic hinges form in beams rather than columns, it is not possible to eliminate plastic hinges from vertical structure completely. A number of potential plastic hinge zones are generally required in the lowest level of columns or walls even in the preferred failure mode, as in Figure 7(b). The number of possible failure modes is substantially reduced by suppressing the chances of occurrence of undesirable failure mechanisms.

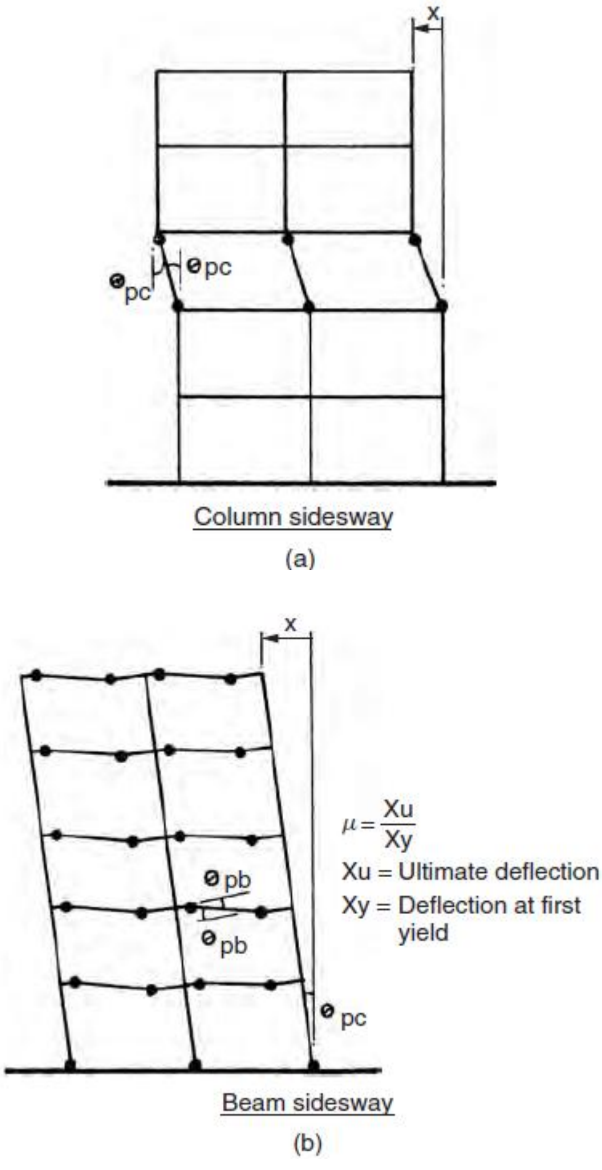


Figure 7: Alternative plastic hinge mechanisms for a typical multi-storey frame

Some items which are normally non-structural become structurally very responsive in earthquakes. This means anything which will interfere with the free deformations of the structure during an earthquake. Where these elements are made of very flexible materials, they will not affect the structure significantly. However, very often it will be desirable for non-structural reasons to construct them of stiff materials such as precast concrete, or concrete blocks, or bricks. Such elements can have a significant effect on the behaviour and safety of the structure. Although these elements may be carrying little vertical load, they can act as shear walls in an earthquake with the following important negative or positive effects. They may:

i) reduce the natural period of vibration of the structure, hence changing the intake of seismic energy and changing the seismic stresses of the 'official' structure;

ii) redistribute the lateral stiffness of the structure, hence changing the stress distribution, sometimes creating large asymmetries;

iii) cause premature failure of the structure usually in shear or by pounding;

iv) suffer excessive damage themselves, due to shear forces or pounding;

v) prevent failure of otherwise inadequate moment resisting frames.

1.4. SEISMIC METHODS OF ANALYSIS

Methods of analysis are broadly classified into 4 types:

1) Linear Static Procedure (LSP)
2) Linear Dynamic Procedure (LDP)
3) Non-linear Static Procedure (NSP)
4) Non-linear Dynamic Procedure (NDP)

1.4.1. Linear Static Procedure

The term "linear" in linear analysis procedures implies "linearly elastic" material, i.e. the stress-strain curve of the material is a straight line and failure of material is assumed at the yield point. The analysis procedure, however, may include geometric nonlinearity of gravity loads acting through lateral displacements and implicit material nonlinearity of concrete and masonry components using properties of cracked sections, as in *P-delta* method of analysis.

One such example is *equivalent lateral force method*, as defined in *IS 1893:2002 Part I*, where the base shear which is the total horizontal force on the structure is calculated on the basis of the structure's mass, its fundamental period of vibration, and corresponding shape. The base end shear is distributed along the height of the structure, in terms of lateral forces, according to the code formula. It assumes that the building responds in its fundamental mode. For this to be true, the building must be low-rise and must not twist significantly when the ground moves.

1.4.2. Linear Dynamic Procedure

Static procedures are appropriate when higher mode effects are not significant. This is generally true for short, regular buildings. Therefore, for tall buildings, buildings with torsional irregularities, or non-orthogonal systems, a dynamic procedure is required. In the linear dynamic procedure, the building is modelled as a multi-degree-of-freedom (MDOF) system with a linear elastic stiffness matrix and an equivalent viscous damping matrix.

The seismic input is modelled using either response spectrum analysis or time history analysis but in both cases, the corresponding internal forces and displacements are determined using linear elastic analysis. The advantage of these linear dynamic procedures with respect to linear static procedures is that higher modes can be considered. However, they are based on linear elastic response and hence the applicability decreases with increasing nonlinear behaviour, which is approximated by global force reduction factors.

Response spectrum analysis is the representation of the maximum response of idealized single degree of freedom systems having certain period and damping, during earthquake ground motion. The maximum response is plotted against the undamped natural period and for various damping values, and can be expressed in terms of maximum absolute acceleration, maximum relative velocity, or maximum relative displacement.

1.4.3. Non-linear Static Procedure

In general, linear procedures are applicable when the structure is expected to remain nearly elastic for the level of ground motion or when the design results in nearly uniform distribution of nonlinear response throughout the structure. As the performance objective of the structure implies greater inelastic demands, the uncertainty with linear procedures increases to a point that requires a high level of conservatism in demand assumptions and acceptability criteria to avoid unintended performance. Therefore, procedures incorporating inelastic analysis can reduce the uncertainty and conservatism.

Pushover analysis is an example of non-linear static procedure in which the magnitude of the structural loading along the lateral direction of the structure is incrementally increased in accordance with a certain pre-defined pattern. It is generally assumed that the behaviour of the structure is controlled by its fundamental mode and the predefined pattern is expressed either in terms of storey shear or in terms of fundamental mode shape. With the increase in magnitude of lateral loading, the progressive non-linear behaviour of various structural elements is captured, and weak links and failure modes of the structure are identified.

However, these procedures are not exact, and cannot accurately account for changes in dynamic response as the structure degrades in stiffness or account for higher mode effects. When the NSP is utilized on a structure that has significant higher mode response, the LDP is also employed to verify the adequacy of the design.

1.4.4. Non-linear Dynamic Procedure

Nonlinear dynamic analysis utilizes the combination of ground motion records with a detailed structural model, and therefore, is capable of producing results with relatively low uncertainty. In nonlinear dynamic analyses, the detailed structural model subjected to a ground-motion record produces estimates of component deformations for each degree of freedom in the model and the modal responses are combined using schemes such as the square-root-sum-of-squares.

In non-linear dynamic analysis, the non-linear properties of the structure are considered as part of a time domain analysis. This approach is the most rigorous, and is required by some building codes for buildings of unusual configuration or of special importance. However, the calculated response can be very sensitive to the characteristics of the individual ground motion used as seismic input; therefore, several analyses are required using different ground motion records to achieve a reliable estimation of the probabilistic distribution of structural response. Time history analysis is an example NSP.

2. SHEAR WALL-FRAME DUAL SYSTEM

2.1. LATERAL LOAD RESISTING SYSTEMS

The load resisting system must be of closed loops, so that it is able to transfer all the forces acting either vertically or horizontally to the ground. The horizontal structural elements are usually diaphragms, such as floor slab, and horizontal bracing in special floors; and the vertical structural elements are the shear walls, braced frame, and moment-resisting frames. The earthquake forces developed at different floor levels in a building are brought down along the height to the ground through the shortest path.

Bureau of Indian Standards (BIS) has approved three major types of lateral force resisting system in the code IS 1893 (Part 1) : 2002, which are as follows:

2.1.1. Moment Resisting Frame

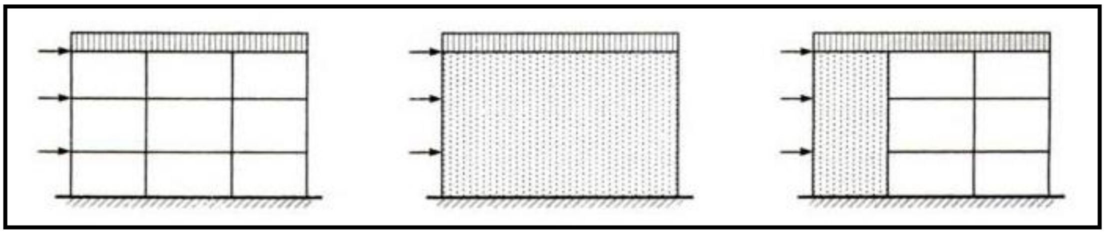
These are the frames in which the beams, columns, and joints resist earthquake forces, primarily by flexure. These frames, when subjected to lateral forces, exhibit zero moments at mid-height of the columns, shear distribution proportional to the moments of inertia of the columns, and relative displacements (or inter-storey drifts) proportional to the shear forces. This is the reason why sometimes these frames are referred to as shear systems. The continuity of the frame also assists in resisting gravity loading more efficiently by reducing positive moments in the centre span of girders. These are preferred because of least obstruction to access. However, this system is recommended only up to thirty-storeys due to a limitation on the drift.

2.1.2. Bearing Wall or Shear Wall System

The walls are load-bearing walls. Some of the bearing walls maybe shear walls. The system is designed for gravity as well as for lateral loads. Under lateral loads, the walls act like vertical cantilever beams. The shear distribution is proportional to the moments of inertia of the cross-sections of the walls. The relative displacements of the floors result from bending deformation of the walls.

2.1.3. Dual System

These consist of moment-resisting frames either braced or with shear walls. The coupling of the above two systems completely alters the moment and shear diagrams of both the walls and the frame. The characteristic of this combination is that in the lower floors the wall retains the frame, while in the upper floors the frame inhibits large displacements of the wall. As a result, the frame exhibits a small variation in storey shear between the first and the last floors. The two systems maybe designed to resist the total design force in proportion to their lateral stiffness.



(1) Moment resisting frames (2) Bearing wall system (3) Building with dual system

Figure 8: Different lateral load resisting systems

2.2. SHEAR WALL- FRAME DUAL SYSTEM

2.2.1. Introduction

The use of shear walls or their equivalent becomes imperative in certain high-rise buildings, if inter-storey deflections caused by lateral loadings are to be controlled. Well designed shear walls not only provide adequate safety, but also give a great measure of protection against costly non-structural damage during moderate seismic disturbances.

Shear walls may be added solely to resist horizontal force, or concrete walls enclosing stairways, elevated shafts, and utility cores may serve as shear walls. Shear walls not only have a very large in-plane stiffness and strength, resisting lateral load and control deflection very efficiently, but may also help to ensure development of all available plastic hinge locations.

Shear walls also provide lateral stiffness to prevent the roof or floor above from excessive side-sway. When shear walls are stiff enough, they will prevent floor and roof framing members from moving off their supports. Also, buildings that are sufficiently stiff will usually suffer less non-structural damage. Shear walls provide stiffness in large part by the ratio of their height to width. Long short walls are stiffer than narrow ones. For a wall of constant height, the stiffness will grow exponentially as the wall length increases.

A slender shear wall, in a high-rise building, when subjected to lateral force has predominantly moment deflections and only very insignificant shear distortions. The term shear wall, in such a case, becomes a misnomer.

More often than not, shear walls are pierced by numerous openings. Such shear walls are called coupled shear walls. The walls on both sides of the openings are interconnected by short, often deep, beams forming part of the wall, or floor slab, or both of these.

Figure 9(a) shows a building with the lateral force represented by arrows acting on the edge of each floor or roof. The horizontal surfaces act as deep beams to transmit loads to vertical load-resisting elements – the shear walls A and B [Figure 9(b)]. These walls, in turn, act as cantilever beams fixed at their base and transfer loads to the foundation. For the building plan shown in Figure 9(a), additional shear walls C and D are provided to resist the lateral loads that mey act in the orthogonal direction [Figure 9(c)]. Shear walls are subjected to the following loads:

i. A variable shear which reaches a maximum at the base;
ii. A bending (overturning) moment which tends to cause vertical tension near the loaded edge and compression at the far edge;
iii. A vertical compression due to ordinary gravity loading from the structure;

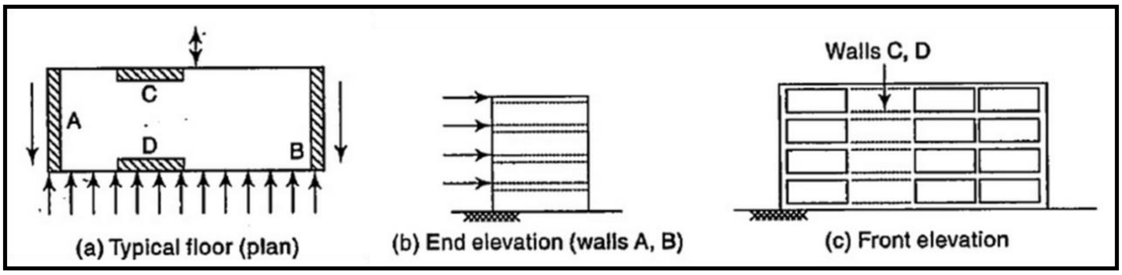


Figure 9: Building with shear walls subjected to horizontal loads

2.2.2. Literature Review

Yoshimura and Inoue (1977) analyzed shear wall frames and concluded that the manner of arrangement of shear walls remarkably affected the maximum base shear caused by earthquakes. Ashraf *et al* (2008) carried out a study to determine the optimum configuration in location of shear walls (lift core) in multi-storey buildings and concluded that shear walls should be placed at a point by coinciding the centre of mass and centre of rigidity of the building.

Ishac and Heidebrecht (1977) concluded that the dynamic analysis of high-rise buildings should be a prime essential because dynamic coupling amplifies the torsional response, and static analysis would not adequately determine stresses and deformations. Frank *et al* (1997) carried out experiments on wood shear walls and found that walls with oversized large panels resisted more load. Wen and Song (2003) investigated the redundancies of SMRF and dual systems. The factors considered were structural configuration (number of bays and shear walls), ductility capacity, uncertainty in demand and capacity, interaction between walls and moment frames, and three-dimensional (3-D) motions. They concluded that in a dual system the number of shear walls had a small effect.

Nollet and Smith investigated deflection of tall wall–frame structures using two dimensional models, in which shear walls were reduced in size or terminated entirely at intermediate heights. It was shown that curtailment of walls was not necessarily detrimental to the performance of the structures.

2.2.2.1. Wall-frame interaction

When a wall-frame structure is loaded laterally, the lower part of the structure deflects in a flexural configuration, i.e. concavity downwind, and the upper part in a shear configuration, i.e. concavity upwind, with a point of inflection at the transition (Figure 10)The greater the racking shear rigidity of the frames relative to flexural rigidity of the walls, the lower the level of the point of inflection.

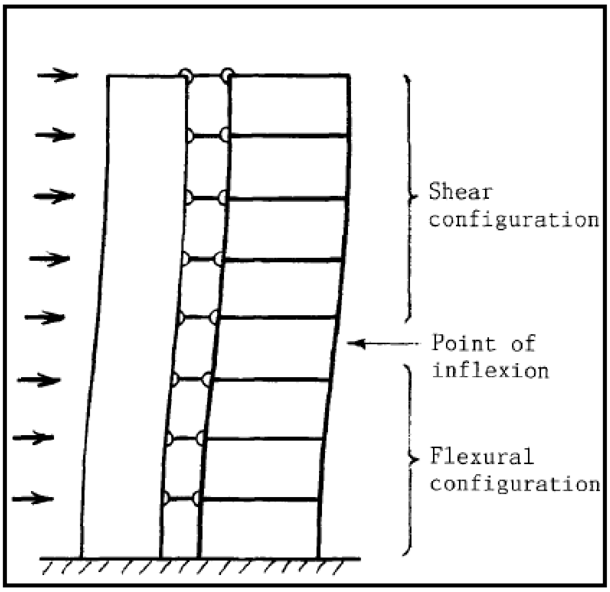


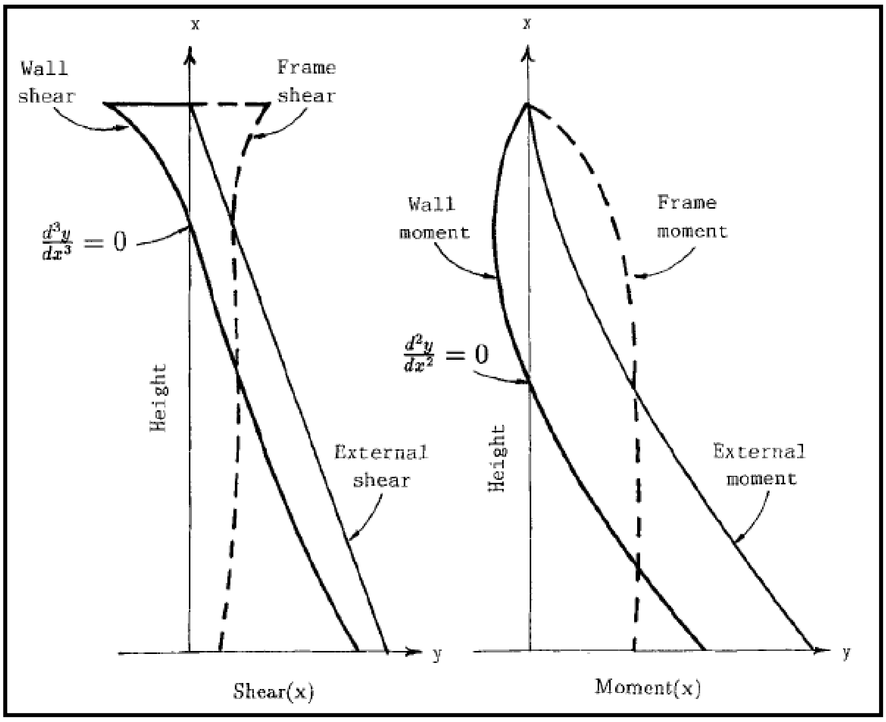
Figure 10: In-plane deflected shape of a wall-frame structure

2.2.2.2. Behavior of curtailed wall-frame structure

The behavior of wall-frame structures having curtailed walls is not obvious. An understanding is made easier, however, by first reviewing the known behavior of the corresponding full-height wall-frame structure.

Referring to the distribution of bending moment for the full-height-wall structure [Figure 11(a)] 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.

Similarly, for the distribution of the shear force in the full-height-wall structure [Figure 11(b)] the shear wall 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 frames exceeds 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



(a) (b)

Figure 11: Typical distribution of Shear Force (left) and Moment (right) in Shear wall and Frame

An inspection of Figures 10 and 11, shows that if the wall were curtailed between the points of zero shear and inflection, the shear in the frame above the curtailment level would be increased by a small amount while the moment in the frame above that level would be reduced. If the wall were curtailed below the point of contra flexure, both the shear and the moment in the frame would increase.

3. MODELLING AND ANALYSIS

3.1. Parametric details of the models

- 20 storey regular buildings with long plan.
- 2 models- (i) 7x3 bays and (ii) 9x3 bays (Figure 12).
- Length of each bay = 5 m, in both directions of the plan.
- Buildings assumed to be located in seismic zone IV.
- All supports of the columns are assumed to be fixed.
- Floors are assumed to act as rigid diaphragms.
- The effect of infill walls in resisting the earthquake forces has been ignored.
- Symmetric RC frames with shear walls curtailed at various storey levels.
- Thickness of shear wall = 250 mm .
- Height of each storey = 3 m, height of plinth level from ground = 1.2 m
- All beams are of 0.35 × 0.65 m sections
- The size of the columns taken is shown in Table 5.

Table 5: Size of columns at various storey levels

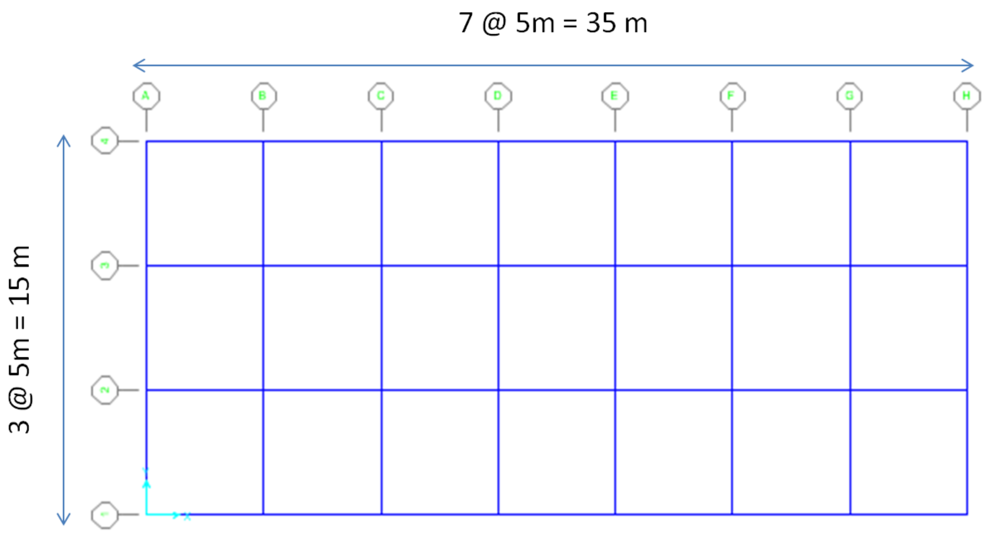
STOREY LEVEL	SIZE OF COLUMNS
1-7	750X750
8-14	600X600
15-20	450X450

3.2. Loads acting on the buildings

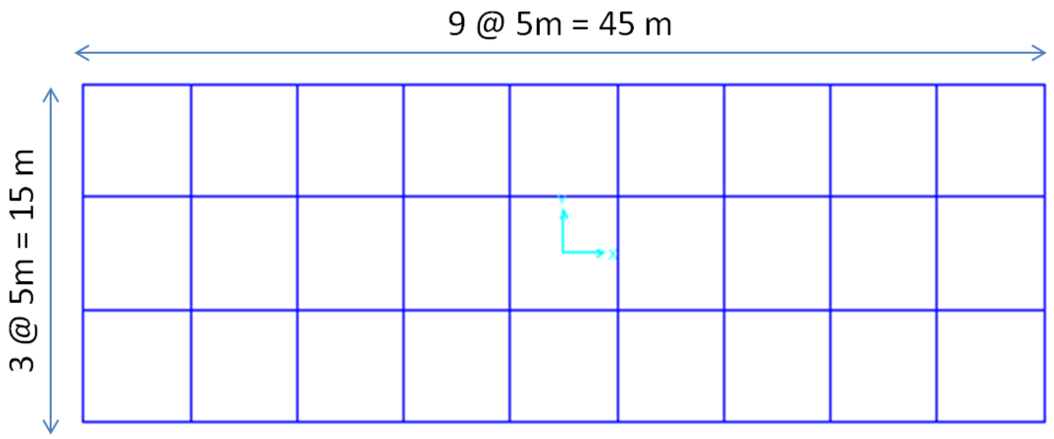
- Dead load intensity at all floor levels is taken as 6 kN/m^2.
- Live load as 3 kN/m^2 for all floors.
- For calculation of seismic weight no live load is considered at the roof level.

3.3. Method of Analysis

- The analysis of the buildings has been done using 3-D modeling in SAP2000 and as per IS-1893: 2002 (Part-1)
- Related factors taken are :
    - seismic Zone factor 0.24,
    - Response reduction factor 5,
    - Importance factor 1.5,
    - Damping 0.05, and
    - Foundation Soil type medium.



(a)

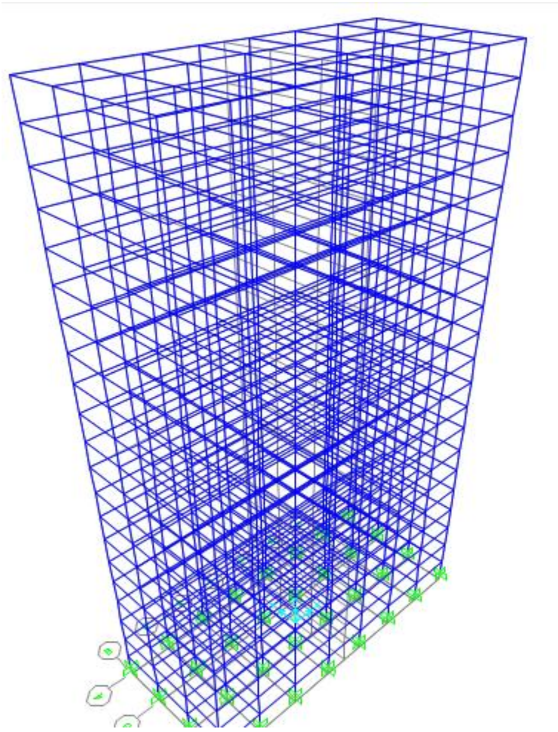


(b)

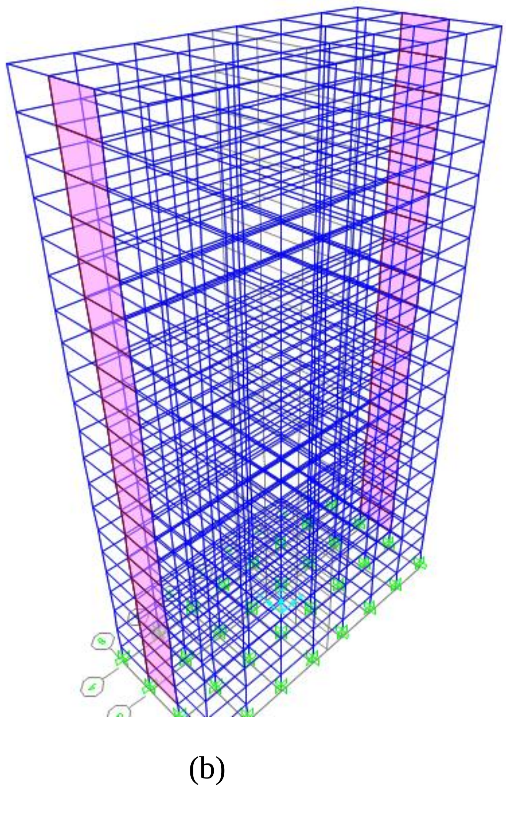
Figure 12: Plans for (a) Model 1 - 7x3 bays and (b) Model 2 – 9x3 bays

3.4. Modelling in the software

Based on above specifications, models were prepared, firstly, for no shear wall [Figure 13] and then for different heights of shear walls at various locations [Figure 14].



(a)



(b)

Figure 13: Three dimensional models (a) without shear walls; (b) with full-height shear walls

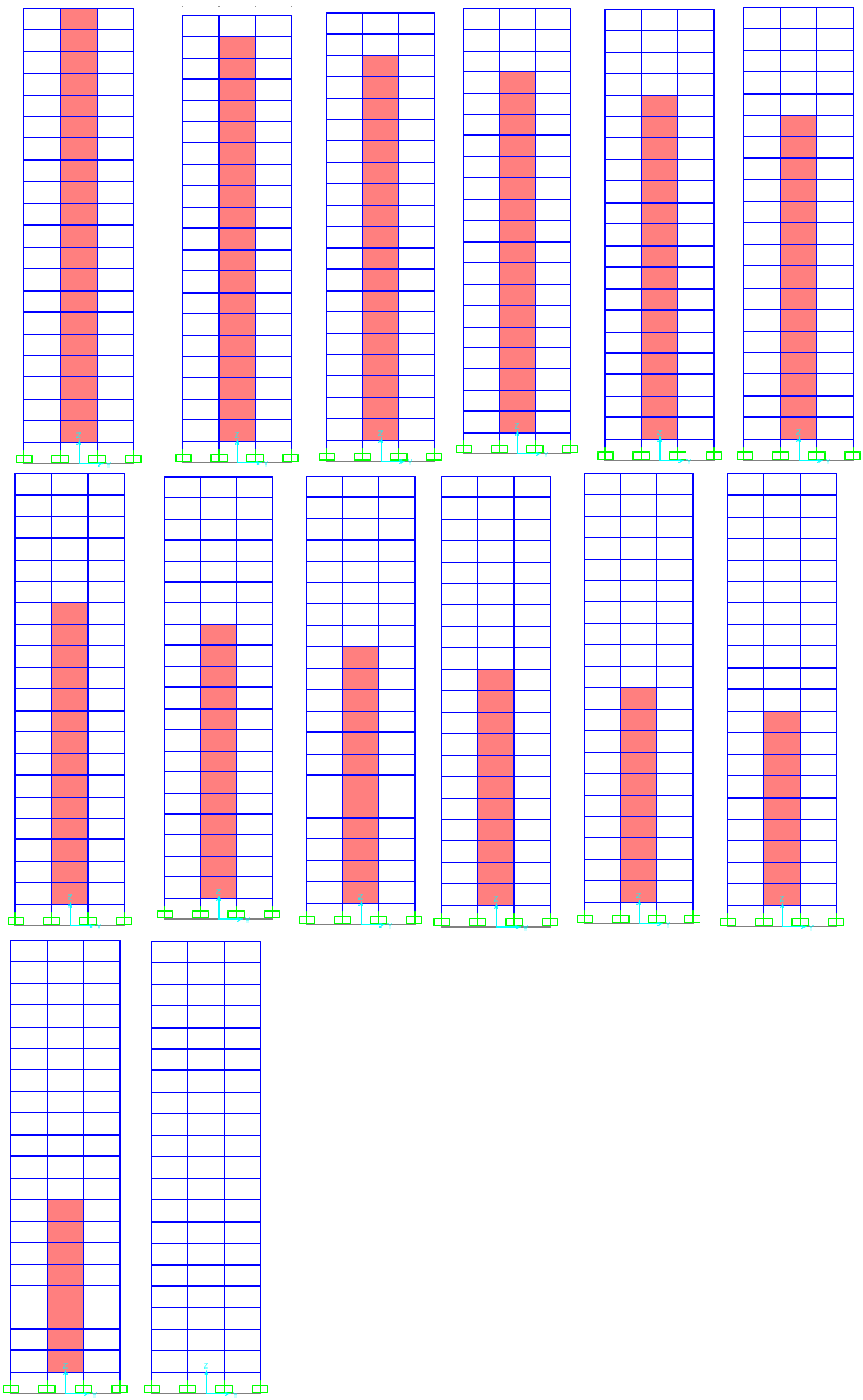
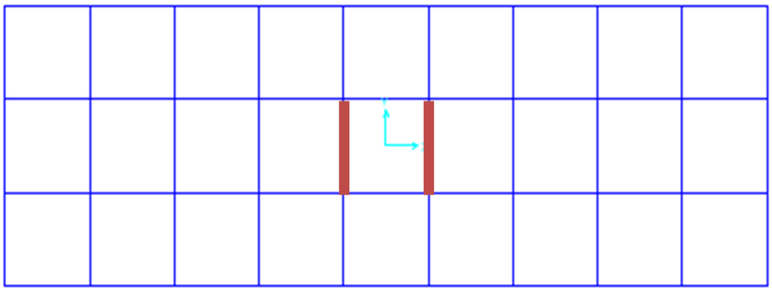


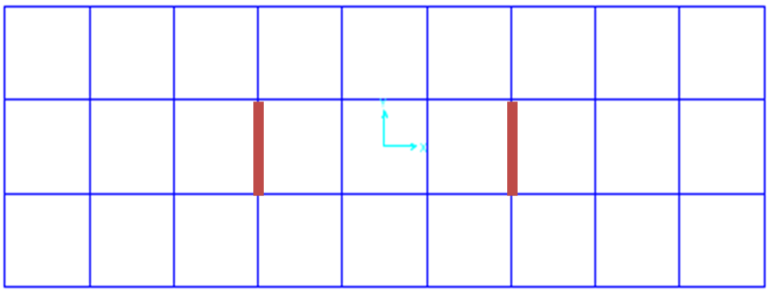
Figure 14: Shear wall curtailed at various storey levels (from 20 up to 8 storey height) and ultimately, building with no shear wall

3.5. Notations

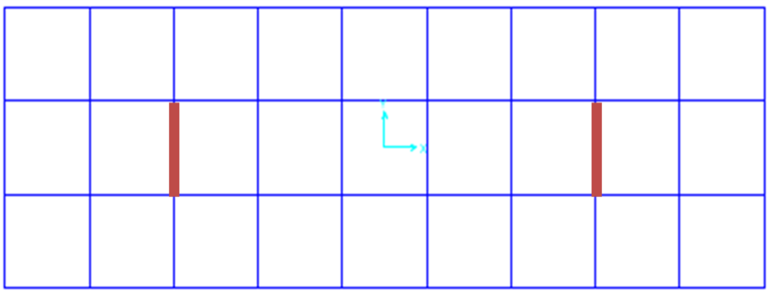
For the purpose of ease in modeling, analysis and most importantly, results interpretation, notations have been introduced to refer to different positions of shear walls in SMRF panels. Beginning with the P1 position in core of the building, shear wall is gradually brought to exterior of the building at the P4 position and P5 position, for Model 1 and Model 2 respectively. For better understanding, the various positions of shear walls are demonstrated in Figure 15.



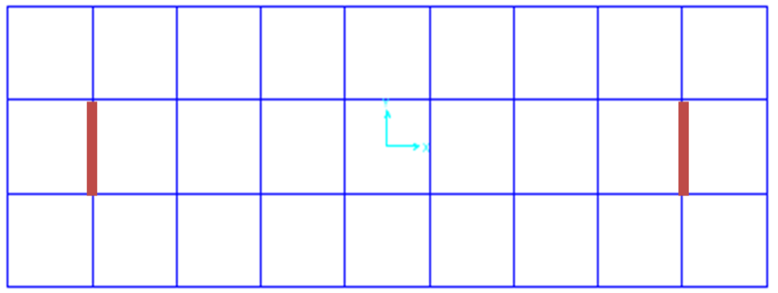
(a) P1 position



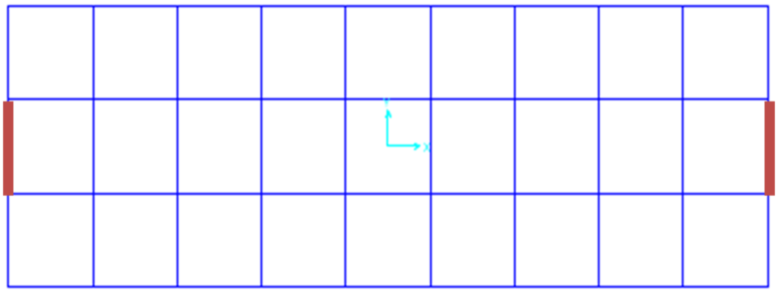
(b) P2 position



(c) P3 position



(d) P4 position



(e) P5 position

Figure 15: Various positions of Shear wall in the SMRF panels of Model 2

4. RESULTS AND INTERPRETATION

4.1. RESULTS FROM RESPONSE SPECTRUM ANALYSIS

Following tables present the results obtained from response spectrum analysis of both models, in terms of rooftop displacement for both out-of-plane (x-direction) and in-plane (y-direction) of shear walls. Base shear has also been noted for different positions and heights of shear walls.

4.1.1. Model 1

In the absence of shear walls:

- Rooftop displacement in x-direction – 71.05 mm
- Rooftop displacement in y-direction – 89.26 mm
- Base shear – 7644.7 kN (both x- and y- directions)

For various positions and heights of shear walls, maximum and minimum rooftop displacements are:

- Maximum = 86.278 mm, in x-dir for P3 position, full shear wall height
- Maximum = 63.482 mm, in y-dir for P4 position, full shear wall height
- Minimum = 70.887 mm, in x-dir for P4 position, 8-storeys shear wall height
- Minimum = 54.369 mm, in y-dir for P3 position, 12-storeys shear wall height
- Base shear values are constant in all positions for each level of shear wall height, and consistently reduce as the height of shear wall reduces.

Percentage of increase in maximum rooftop displacement in x-direction, with respect to that in absence of shear walls = 21.43%

Percentage of decrease in maximum rooftop displacement in y-direction, with respect to that in absence of shear walls = 39.09%

Table 6: Rooftop displacements in x-direction

Height of shear wall (in storeys)	Rooftop Displacement (in mm)			
	P1	P2	P3	P4
20	85.087	85.566	86.278	84.339
19	83.222	83.644	84.267	82.391
18	81.54	81.908	82.45	80.598
17	80.066	80.377	80.838	78.976
16	78.801	79.054	79.434	77.534
15	77.732	77.922	78.225	76.269
14	77.009	77.135	77.354	75.303
13	75.969	76.077	76.263	74.243
12	75.038	75.131	75.292	73.314
11	74.225	74.302	74.439	72.51
10	73.525	73.587	73.61	71.835
9	72.94	72.987	72.99	71.295
8	72.94	72.489	72.476	70.887

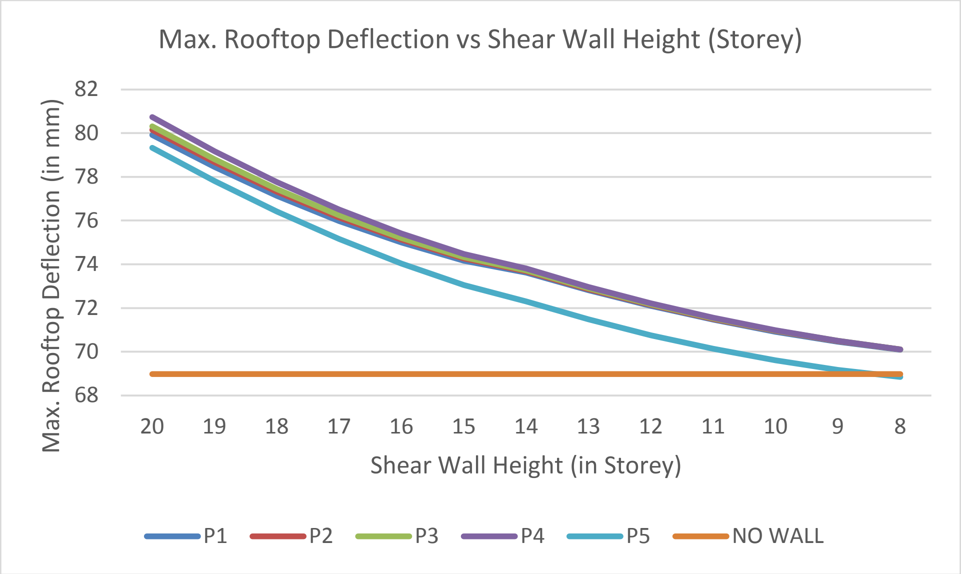


Figure 16: Comparison of Rooftop displacement vs Height of Shear walls in x-direction

Table 7: Rooftop displacements in y-direction

Height of shear wall (in storeys)	Rooftop Displacement (in mm)			
	P1	P2	P3	P4
20	63.476	61.593	61.602	63.482
19	61.45	59.631	59.638	61.459
18	59.519	57.802	57.806	59.529
17	57.972	56.369	56.365	57.962
16	56.856	55.388	55.379	56.83
15	56.179	54.87	54.859	56.153
14	56.203	55.05	55.038	56.178
13	55.589	54.549	54.535	55.564
12	55.297	54.382	54.369	55.271
11	55.385	54.602	54.589	55.36
10	55.89	55.239	55.236	55.866
9	56.862	56.337	56.333	56.839
8	58.307	57.9	57.896	58.286

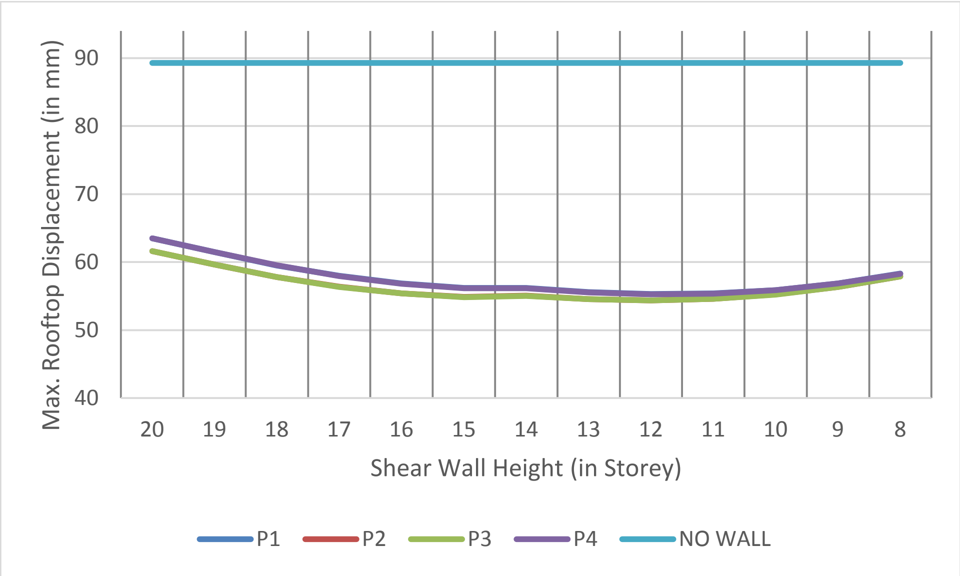


Figure 17: Comparison of Rooftop displacement vs Height of Shear walls in y-direction

(Overlapping curves are P1 and P4; P2 and P3)

Table 8: Base shear (equal values in both x- and y-directions)

Height of shear wall (in storeys)	Base Shear (in kN)			
	P1	P2	P3	P4
20	9410.743	9410.743	9415.093	9415.093
19	9322.441	9322.441	9326.791	9326.791
18	9234.139	9234.139	9238.489	9238.489
17	9145.837	9145.837	9150.187	9150.187
16	9057.536	9057.536	9061.885	9061.885
15	8969.234	8969.234	8973.584	8973.584
14	8880.932	8880.932	8885.282	8885.282
13	8792.63	8792.63	8796.98	8796.98
12	8704.329	8704.329	8708.678	8708.678
11	8616.027	8616.027	8620.376	8620.376
10	8527.725	8527.725	8527.725	8532.075
9	8439.423	8439.423	8439.423	8443.773
8	8351.121	8351.121	8351.121	8355.471

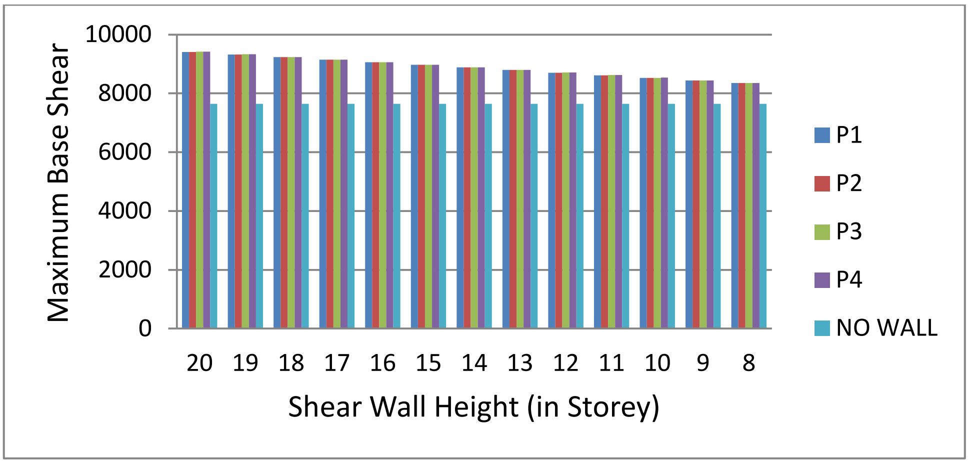


Figure 18: Comparison of Base shear vs Height of Shear walls

4.1.2. Model 2

In the absence of shear walls:

- Rooftop displacement in x-direction – 68.98 mm
- Rooftop displacement in y-direction – 90.21 mm
- Base shear – 9652.665 kN (both x- and y- directions)

For various positions and heights of shear walls, maximum and minimum rooftop displacements are:

- Maximum = 80.737 mm, in x-dir for P4 position, full shear wall height
- Maximum = 66.931 mm, in y-dir for P5 position, full shear wall height
- Minimum = 68.851 mm, in x-dir for P5 position, 8-storeys shear wall height
- Minimum = 58.331 mm, in y-dir for P3 position, 12-storeys shear wall height
- Base shear values are constant in all positions for each level of shear wall height, and consistently reduce as the height of shear wall reduces.

Percentage of increase in maximum rooftop displacement in x-direction, with respect to that in absence of shear walls = 17.04%

Percentage of decrease in maximum rooftop displacement in y-direction, with respect to that in absence of shear walls = 35.34%

Table 9: Rooftop displacements in x-direction

Height of shear wall (in storeys)	Rooftop Displacement (in mm)				
	P1	P2	P3	P4	P5
20	79.911	80.153	80.307	80.737	79.333
19	78.448	78.663	78.8	79.171	77.813
18	77.132	77.322	77.441	77.757	76.416
17	75.979	76.141	76.242	76.503	75.152
16	74.994	75.124	75.207	75.412	74.028
15	74.161	74.256	74.322	74.472	73.043
14	73.621	73.679	73.726	73.812	72.301
13	72.813	72.864	72.904	72.967	71.477
12	72.092	72.137	72.17	72.216	70.755
11	71.461	71.498	71.526	71.554	70.127
10	70.917	70.947	70.968	70.982	69.6
9	70.461	70.485	70.5	70.5	69.175
8	70.084	70.1	70.111	70.099	68.851

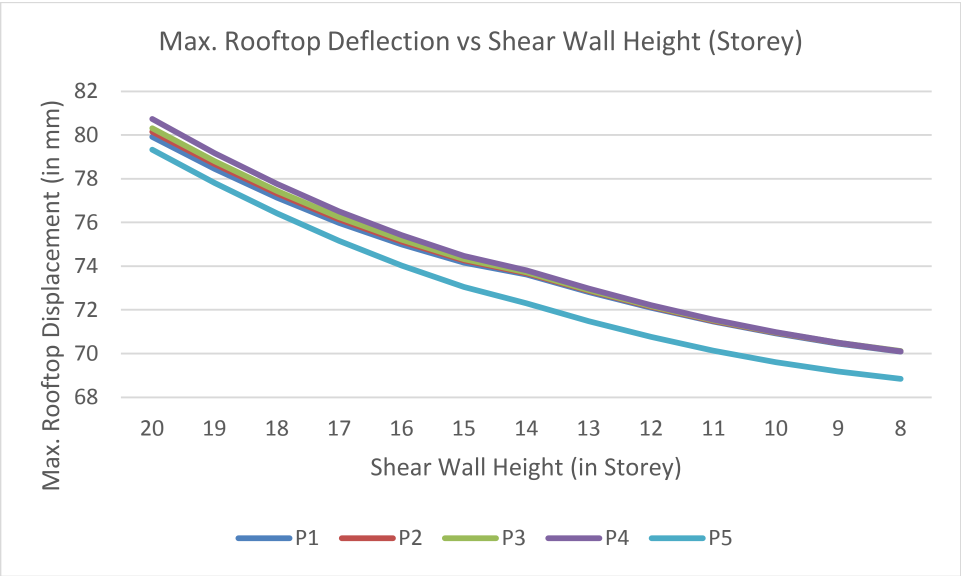


Figure 19: Comparison of Rooftop displacements vs Height of Shear walls in x-direction

Table 10: Rooftop displacements in y-direction

Height of shear wall (in storeys)	Rooftop Displacement (in mm)				
	P1	P2	P3	P4	P5
20	66.924	65.187	65.166	65.208	66.931
19	65.139	63.442	63.423	63.463	65.149
18	63.392	61.772	61.754	61.791	63.402
17	61.948	60.433	60.416	60.449	61.958
16	60.881	59.492	59.477	59.506	60.89
15	60.215	58.967	58.954	58.978	60.223
14	60.215	59.108	59.097	59.116	60.222
13	59.594	58.583	58.573	58.59	59.601
12	59.24	58.34	58.331	58.346	59.246
11	59.21	58.429	58.422	58.434	59.215
10	59.544	58.885	58.88	58.889	59.547
9	60.291	59.754	59.75	59.756	60.293
8	61.466	61.044	61.041	61.046	61.468

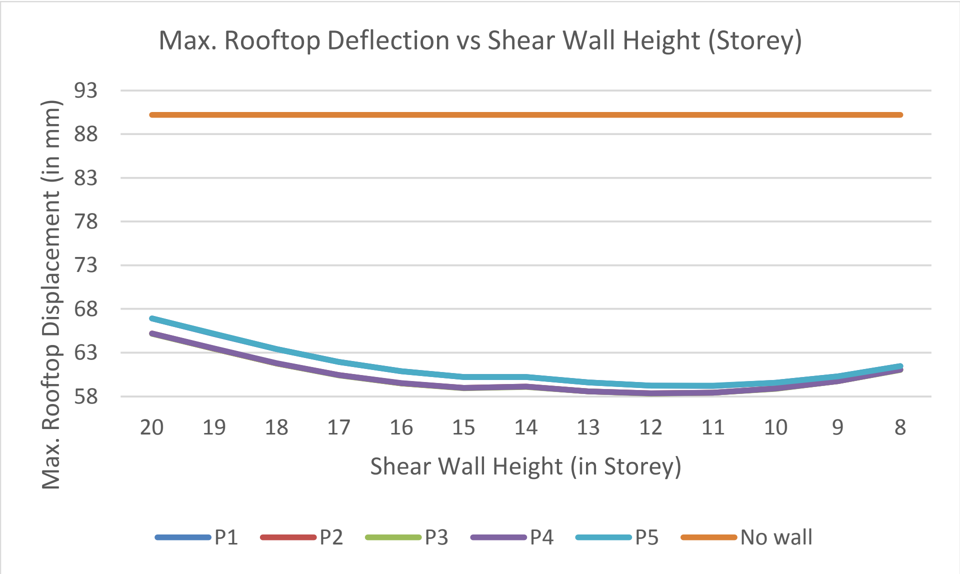


Figure 20: Comparison of Rooftop displacements vs Height of Shear walls in y-direction

(Overlapping curves are P1 and P5; P2, P3 and P4)

Table 11: Base shear (equal values in both x- and y-directions)

Height of shear wall (in storeys)	Base Shear (in kN)				
	P1	P2	P3	P4	P5
20	11418.7	11418.7	11418.7	11418.7	11418.7
19	11330.4	11330.4	11330.4	11330.4	11330.4
18	11242.1	11242.1	11242.1	11242.1	11242.1
17	11153.8	11153.8	11153.8	11153.8	11153.8
16	11065.49	11065.49	11065.49	11065.49	11065.49
15	10977.19	10977.19	10977.19	10977.19	10977.19
14	10888.89	10888.89	10888.89	10888.89	10888.89
13	10800.59	10800.59	10800.59	10800.59	10800.59
12	10712.29	10712.29	10712.29	10712.29	10712.29
11	10623.99	10623.99	10623.99	10623.99	10623.99
10	10535.68	10535.68	10535.68	10535.68	10535.68
9	10447.38	10447.38	10447.38	10447.38	10447.38
8	10359.08	10359.08	10359.08	10359.08	10359.08

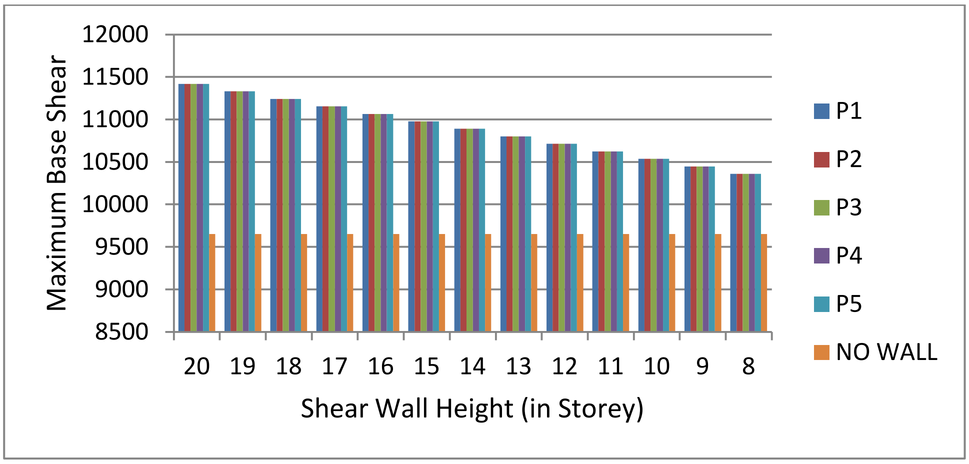


Figure 21: Comparison of Rooftop displacements vs Height of Shear walls in y-direction

4.2. INTERPRETATION OF RESULTS

Results obtained above are interpreted to determine a relationship between the maximum rooftop displacements and the storey height of shear walls, using curve fitting technique via regression analysis in MS Excel. Since the displacements in x-direction and base shear have a consistently declining trend, displacements only in the y-direction are interpreted. It can also be observed from Figure 20, that the displacement trend remains more or less similar for all positions of shear walls. Hence one position, P1 position, is interpreted to identify the model that best describes the displacement curve, for varying height of shear wall (Figure 22).

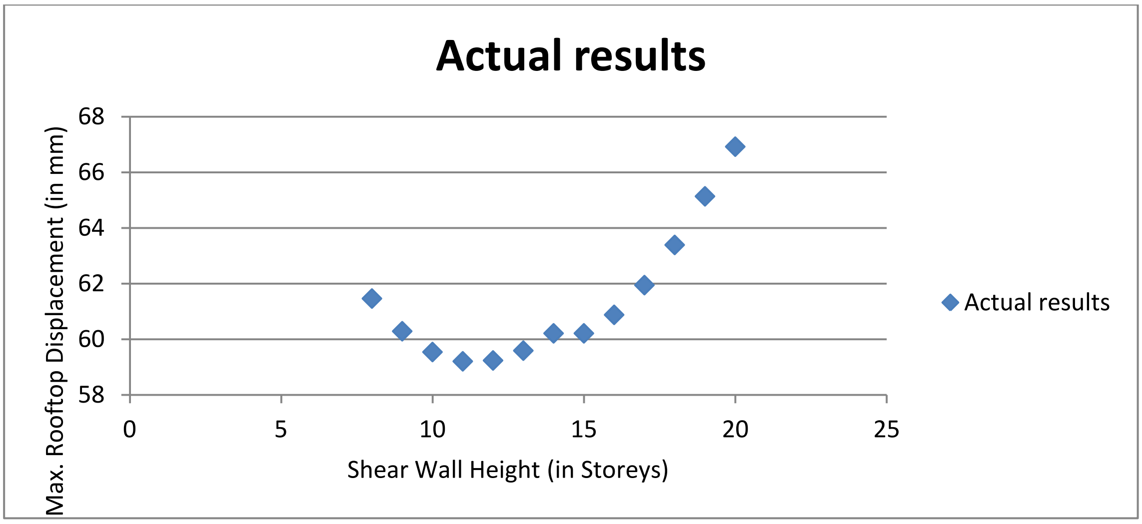


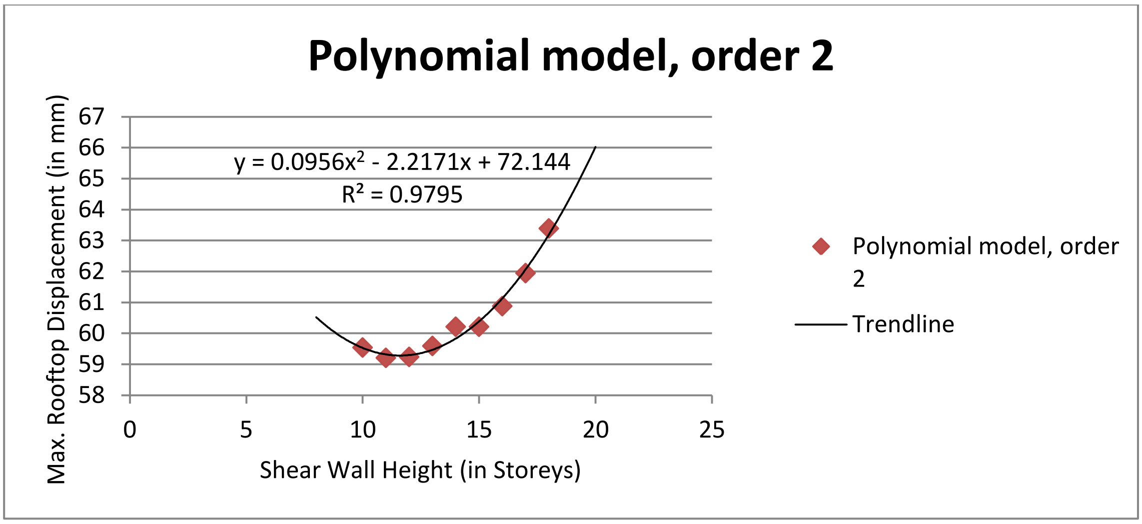
Figure 22: Actual displacement results plotted as data points

4.2.1. Curve fitting

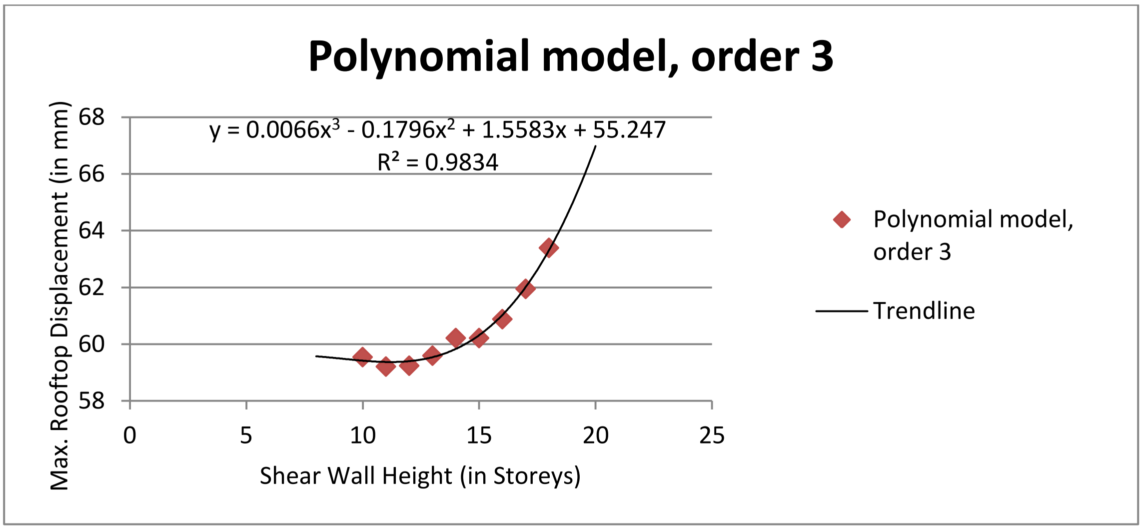
The model that best identifies with the data is the one that fulfills following two requirements:

a) It must explain the trend with least deviations, i.e. it must have the highest value of R^2 (where R is percent of the response variable variation that is explained by the model), and
b) It must be able to predict values with least deviation from actual results.

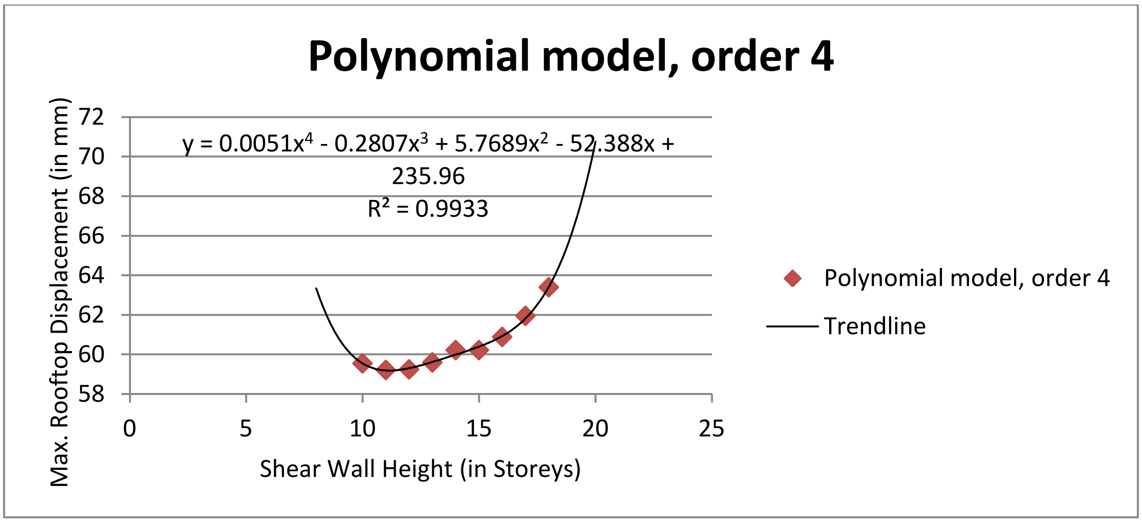
The curves that best fit the displacement trend are polynomial in nature. Polynomial models have the most flexibility and they can smoothly fit the data with excellent precision. Objective (b) is achieved by considering only 9 of the 13 data points; 2 data points of the end and 2 of the beginning are removed from the curve to determine best of the predictions made by the polynomial models. From Figure 23, it is apparent that while the polynomials of higher degrees, even though being better fits to the curve accomplishing objective (a), will give poor predictions, failing in objective (b).


Polynomial model, order 2
y = 0.0956x² - 2.2171x + 72.144
R² = 0.9795
Max. Rooftop Displacement (in mm)
67
66
65
64
63
62
61
60
59
58
0
5
10
15
20
25
Shear Wall Height (in Storeys)
Polynomial model, order 2
Trendline

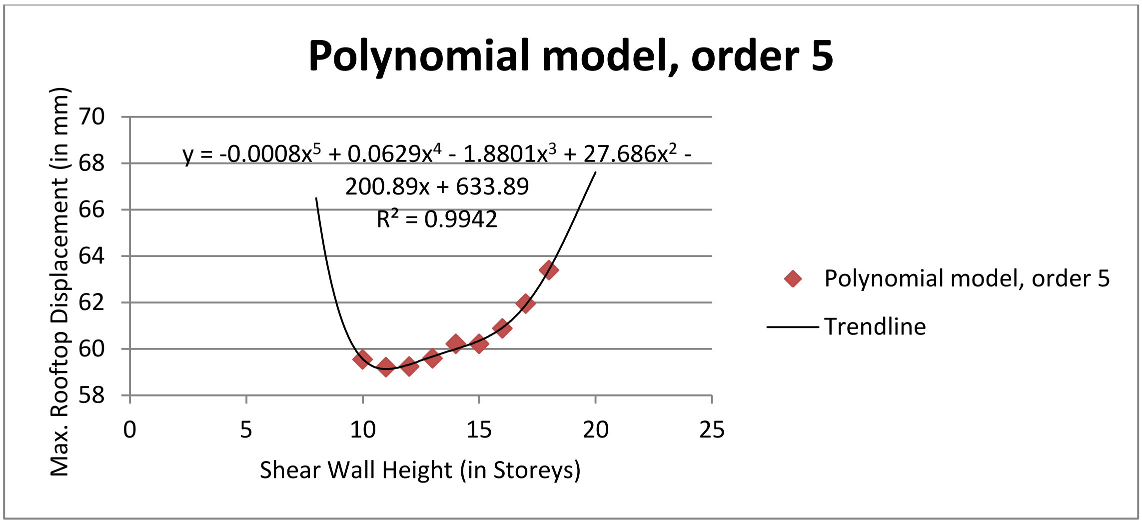
(a)


Polynomial model, order 3
y = 0.0066x³ - 0.1796x² + 1.5583x + 55.247
R² = 0.9834
Max. Rooftop Displacement (in mm)
68
66
64
62
60
58
0
5
10
15
20
25
Shear Wall Height (in Storeys)
Polynomial model, order 3
Trendline

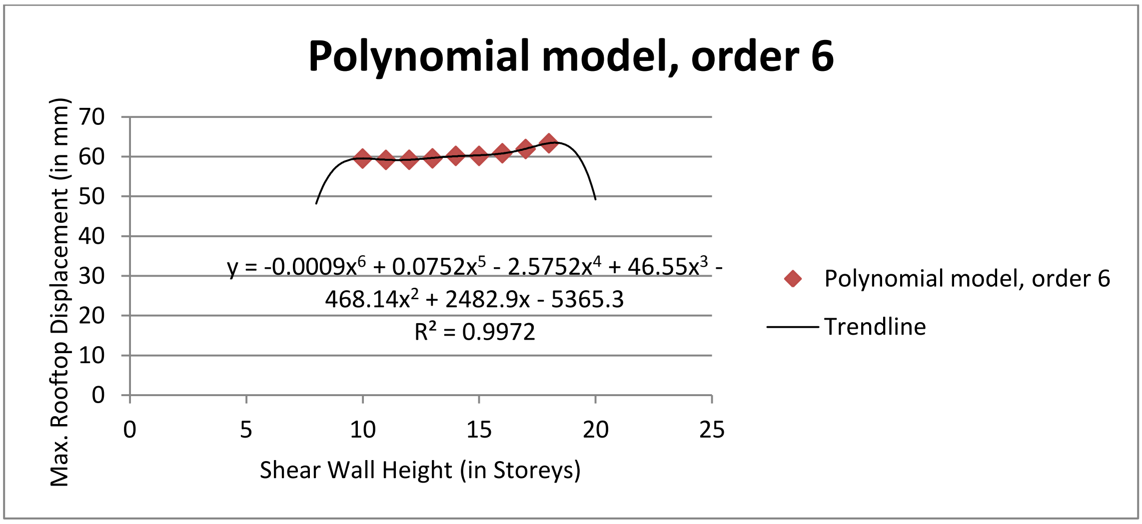
(b)


Polynomial model, order 4
y = 0.0051x⁴ - 0.2807x³ + 5.7689x² - 52.388x + 235.96
R² = 0.9933
Max. Rooftop Displacement (in mm)
72
70
68
66
64
62
60
58
0
5
10
15
20
25
Shear Wall Height (in Storeys)
Polynomial model, order 4
Trendline

(c)



(d)



(e)

Figure 23: Curve fitting and prediction through polynomial models

R^2 values are higher for higher degree polynomials (Table 12), but the predictions are far from the actual results. Since only 2nd, 3rd and 4th order polynomials follow a trend similar to the data, they are noted (Table 13) and for their accuracy is determined. The model with least deviations (standard deviation) is selected as most appropriate to represent maximum displacement of roof.

Table 12: R-squared values for different polynomial models

	Order 2	Order 3	Order 4	Order 5	Order 6
R-squared values (in %)	97.95	98.34	99.33	99.42	99.72

Table 13: Comparison of predicted values

Storey	Original result	Predicted values		
		order 2	order 3	order 4
8	61.466	60.5256	59.5982	63.2368
9	60.291	59.9337	59.5355	60.5797
19	65.139	64.5307	65.2885	62.4767
20	66.924	66.042	67.373	66.16

From the above comparison it can be concluded that for a varying height of shear wall, rooftop displacement follows a trend that can be best represented by a quadratic equation, given by

$$y = 0.0956x^2 - 2.2171x + 72.144$$

where, x is the height of shear wall in storey.

5. DISCUSSIONS AND CONCLUSIONS

A study was carried out to optimize height and location of shear walls in a 20 storeyed 7x3 bays and 9x3 bays RCC buildings with SMRFs to obtain least rooftop displacement. Comparing the results of analyses following can be concluded about the effects of shear wall curtailment at varying storey levels for different positions of them in the SMRFs. Change in position of shear walls does not have a significant effect on displacements, but for discussions' sake, we will study the minute variations:

- In the absence of shear walls, rooftop displacements in y-direction are more than those in x-direction. This is because larger number of bays led to greater stiffness of the building in that direction.
- When 2 bays are added to Model 1, displacements in x-direction are reduced, with and without shear walls, as expected, since the added bays increase stiffness in the direction of their addition.
- Introduction of shear walls reduces rooftop displacements in y-direction (i.e. in the plane of walls) as compared to displacements in their absence, but increases them in x-direction. This is because addition of shear walls increases stiffness and strength in their own plane but primarily increases the seismic weight when it comes to their effect out-of their plane.
- Curtailment of shear walls progressively reduces displacements in x-direction, for all positions of shear walls, following the previous explanation.
- In both models, shear walls in the outermost position (P4 and P5) lead to minimum displacements in x-direction, at all curtailment levels. This behaviour can be explained by the integrity of the structure that shear walls introduce when they are placed at the perimeter. The whole structure acts like a box with maximum stiffness at its perimeter.
- In the y-direction, i.e. in their own plane, shear walls greatly reduce the lateral drift, up to almost 40% (in Model 1).

- Displacements in y-direction show a similar trend for both building plans, i.e. almost equal displacements for extreme interior & exterior positions; and similar displacements for intermediate positions of shear walls.
- Displacements in y-direction are smaller for intermediate positions; reason being more uniform distribution of load on either side of the shear wall, when in these positions.
- For all plans and position of shear walls, the optimum curtailment level works out to be around the 12th storey (60% height).
- Difference in displacements is more pronounced for different positions of shear walls in both x- as well as y-direction, when the height of shear wall is larger. For all positions, displacements finally converge to a single value as the height of shear wall decreases. In x-direction, this value is equal to the displacement that occurs in absence of shear walls.
- The relationship between maximum rooftop displacements in y-direction, and the height of shear wall is a quadratic one.

From the above discussions, it can be concluded that a symmetric geometry leads to best performance of a structure. Hence, when the plan of a building is long, it must be separated into two regular ones (probably by an expansion joint). If that is not feasible, then shear walls must be provided in both the directions of the plan of the building.

6. SCOPE OF FUTUTRE WORK

After all the analyses and discussions carried out, there is still a lot of scope of work that can be done in this project in the future:

- There exists a consistent aberration in the displacement trend, at the 14^{th} storey height of shear wall that needs to be further investigated.
- Only 2 models were analysed in the current study, which can be extended to include more kinds of plan irregularities.
- We can predict from current study that shear walls provided in both directions of the plan will be more effective. This claim can be substantiated by introduction of walls in both directions, and a study of different combinations of their positions.
- More variables can be included in the expression for rooftop displacements, like height of the building, storey height, shear rigidity, size of members, etc.

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