ANALYSIS AND DESIGN OF SHEAR WALL USING SAP 2000

A PROJECT

Submitted in partial fulfillment of the requirements for the award of the degree of

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Under the supervision of

Mr. Anil Kumar Dhiman

By

Aditya Shukla (111666)
Shivam Arora (111640)

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

WAKNAGHAT SOLAN – 173 234

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CERTIFICATE

This is to certify that the work which is being presented in the project title "**ANALYSIS AND DESIGN OF SHEAR WALL USING SAP 2000**" in partial fulfillment of the requirements for the award of the degree of Bachelor of technology and submitted in Civil Engineering Department, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out By Aditya Shukla & Shivam Arora during a period from August 2014 to May 2015 under the supervision of Mr. Anil Dhiman Assistant Professor, Civil Engineering Department, Jaypee University of Information Technology, Waknaghat.

The above statement made is correct to the best of my knowledge.

Date: /5/2015

Prof. Dr. Ashok Kumar Gupta	**Anil Dhiman**	
Professor & Head of Department	Assistant Professor	External Examiner
Civil Engineering Department	Civil Engineering Department	
JUIT Waknaghat	JUIT Waknaghat	

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"EXPRESSION OF FEELINGS BY WORDS MAKES THEM LESS SIGNIFICANT WHEN IT COMES TO STATEMENT OF GRATITUDE"

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Date:

(Shivam Arora) **(Aditya Shukla)**

ABSTRACT

The reinforced concrete shear wall is one of the most commonly used lateral load resisting in high rise building. The reinforced concrete shear wall building is high in plane stiffness and strength which can be used to simultaneously resist large horizontal load and support gravity load. The scope of the present work was to study seismic responses of the five, six, seven, & eight storey RC shear wall building using standard package SAP2000. Developing model and analyzing the reinforced concrete shear wall building by using different non-linear method (time history).Non-linear static procedure is developed with the aim of overcoming the insufficiency and limitations of linear methods, whilst at the same time maintaining a relatively simple application. All procedures incorporate performance-based concepts paying more attention to damage control. The comparison of these models for different parameters like displacement, moment and shear has been presented by RC shear wall building with opening (core of lift) without opening (i.e. exterior). The experimental results have also been compared and found to be in agreement with the software solutions. Reason for study is many reinforced concrete (RC) buildings have either collapsed or experienced different levels of damage during past earthquakes, hence to reduce such destruction.

Keywords: Response spectrum, seismic engineering, concrete structure.

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CHAPTER1

INTRODUCTION

1.1INTRODUCTION

Shear wall is a structural system composed of braced panels to counter the effects of lateral load acting on a structure. Wind and seismic loads are the most common loads that shear walls are designed to carry. Under several building codes, including the International Building Code and Uniform Building Code, all exterior wall lines in wood or steel frame construction must be braced. Depending on the size of the building some interior walls must be braced as well.A typical shear wall is shown in Fig. 1.1.1

Shear wall is to create braced panels in the wall line using structural plywood sheathing with specific nailing at the edges and supporting framing of the panel. Shear walls resist in-plane loads that are applied along its height. The applied load is generally transferred to the wall by a diaphragm or collector or drag member. They are built in wood, concrete. Plywood is the conventional material used in shear walls, but with advances in technology and modern building methods, other prefabricated options have made it possible to inject shear assemblies into narrow walls that fall at either side of an opening. Sheet steel and steel-backed shear panels in the place of structural plywood in shear walls has proved to provide stronger seismic resistance.

Shear walls should be located on each level of the structure including the crawl space. To form an effective box structure, equal length shear walls should be placed symmetrically on all four exterior walls of the building. Shear walls should be added to the building interior when the exterior walls cannot provide sufficient strength and stiffness or when the allowable span-width ratio for the floor or roof diaphragm is exceeded. For subfloors with conventional diagonal sheathing, the span-width ratio is 3:1. This means that a 25-foot wide building with this subfloor will not require interior shear walls until its length exceeds 75 feet unless the strength or stiffness of the exterior shear walls are inadequate .

Shear walls are most efficient when they align vertically and are supported on foundation walls or footings. When shear walls do not align, other parts of the building will need additional strengthening. Consider the common case of an interior wall supported by a subfloor over a crawl space and there is no continuous footing beneath the wall. For this wall to be used as shear wall, the subfloor and its connections will have to be strengthened near the wall. For new construction, thicker plywood or extra nailing and connections can be added. For retrofit work, existing floor construction is not easily changed. That's the reason why most retrofit work uses walls with continuous footings underneath them as shear walls.

Another type of alignment problem occurs when the ends of shear walls do not align from story to story. This condition creates the need for extra framing members and connections in the walls for holdown devices. Holdown devices must transfer the uplift from the shear wall to framing members that can resist it. When full height studs are not available, special connections must be added. These connections must assemble enough of the structure's framing to resist the uplift.

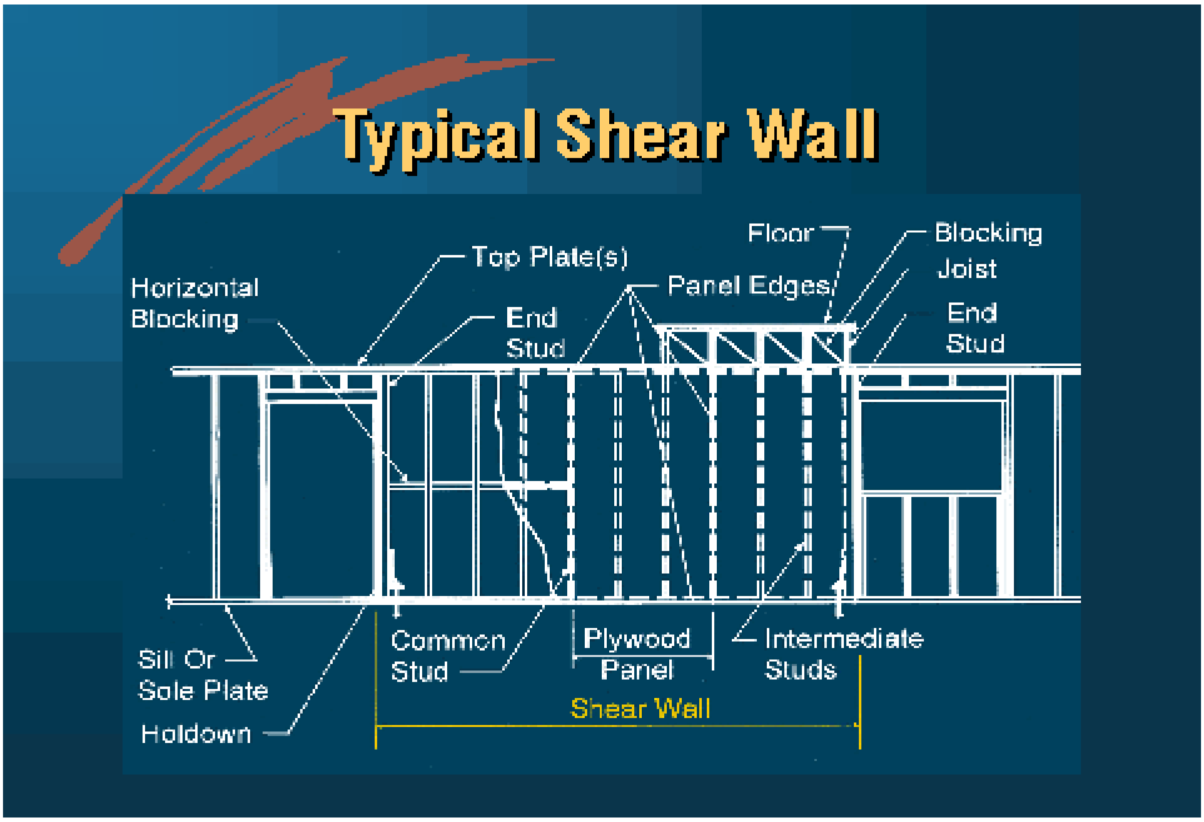


Fig. 1.1.1 Typical Shear Wall

1.2 ARCHITECTURAL ASPECT OF SHEAR WALL

Most RC buildings with shear walls also have columns; these columns primarily carry gravity loads (those due to self-weight and contents of building). Shear walls provide large strength and stiffness to buildings in the direction of their orientation, which significantly reduces lateral sway of the building and thereby reduces damage to structure and its contents. Since shear walls carry large horizontal earthquake forces, the overturning effects on them are large. Thus, design of their foundations requires special attention. Shear walls should be provided along preferably both length and width. However, if they are provided along only one direction, a proper grid of beams and columns in the vertical plane (called a moment-resistant frame) must be provided along the other direction to resist strong earthquake effects.

Door or window openings can be provided in shear walls, but their size must be small to ensure least interruption to force flow through walls. Moreover, openings should be symmetrically located. Special design checks are required to ensure that the net cross-sectional area of a wall at an opening is sufficient to carry the horizontal earthquake force. Shear walls in buildings must be symmetrically located in plan to reduce ill-effects of twist in buildings. They could be placed symmetrically along one or both directions in plan as per Fig. 1.2.1. Shear walls are more effective when located along exterior perimeter of the building such a layout increases resistance of the building to twisting.

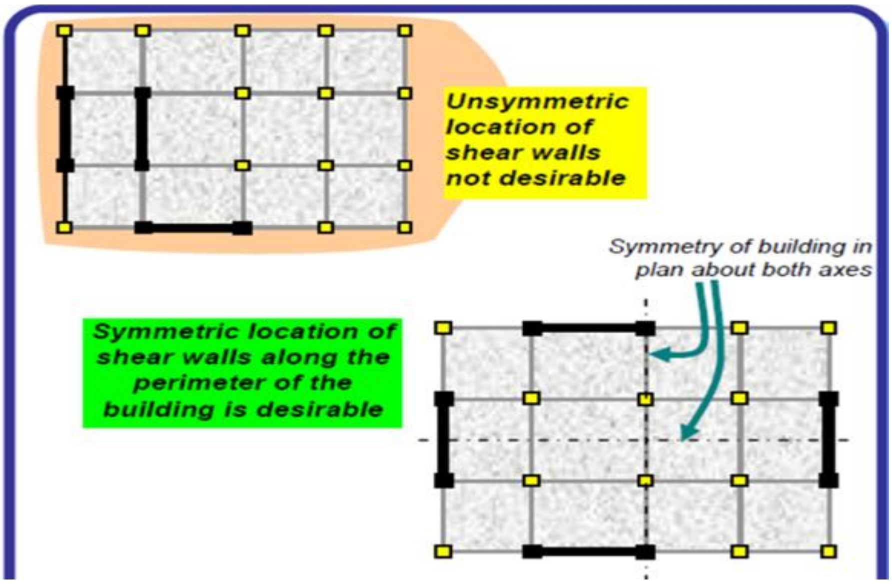


Fig. 1.2.1 Symmetrical and unsymmetrical shear wall

Ductile Design of Shear Walls Just like reinforced concrete (RC) beams and columns, RC shear walls also perform much better if designed to be ductile. Overall geometric proportions of the wall, types and amount of reinforcement, and connection with remaining elements in the building help in improving the ductility of walls. The Indian Standard Ductile Detailing Code for RC members (IS: 13920-1993) provides special design guidelines for ductile detailing of shear walls.

Steel reinforcing bars are to be provided in walls in regularly spaced vertical and horizontal grids as shown in Fig. 1.2.2. The vertical and horizontal reinforcement in the wall can be placed in one or two parallel layers called curtains. Horizontal reinforcement needs to be anchored at the ends of walls. The minimum area of reinforcing steel to be provided is 0.0025 times the cross-sectional area, along each of the horizontal and vertical directions. This vertical reinforcement should be distributed uniformly across the wall cross-section.

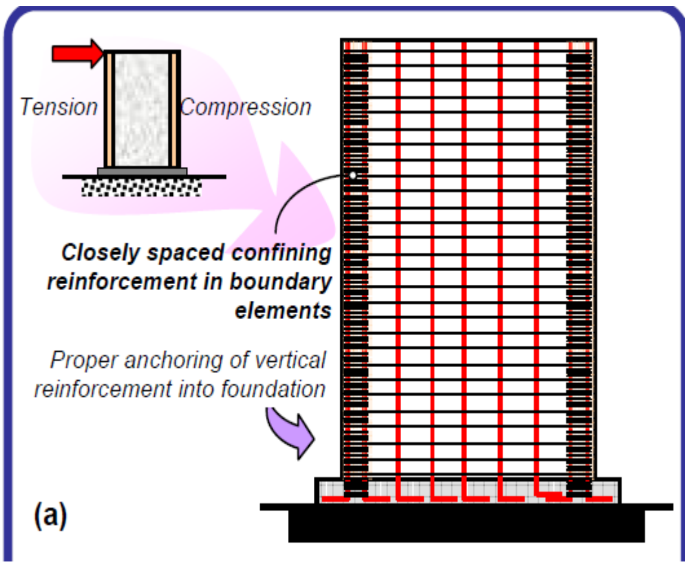


Fig. 1.2.2 Steel distribution in Shear wall

Shear walls are oblong in cross-section, i.e., one dimension of the cross-section is much larger than the other. While rectangular cross-section is common, L- and U-shaped sections are also used. Thin-walled hollow RC shafts around the elevator core of buildings also act as shear walls, and should be taken advantage of to resist earthquake forces.Under the large overturning effects caused by horizontal earthquake forces, edges of shear walls experience high compressive and tensile stresses. To ensure that shear walls behave in a ductile way, concrete in the wall end regions must be reinforced in a special manner to sustain these load reversals without losing strength. End regions of a wall with increased confinement are called boundary elements.

This special confining transverse reinforcement in boundary elements (Fig. 1.2.3) is similar to that provided in columns of RC frames. Sometimes, the thickness of the shear wall in these boundary elements is also increased. RC walls with boundary elements have substantially higher bending strength and horizontal shear force carrying capacity, and are therefore less susceptible to earthquake damage than walls without boundary elements.

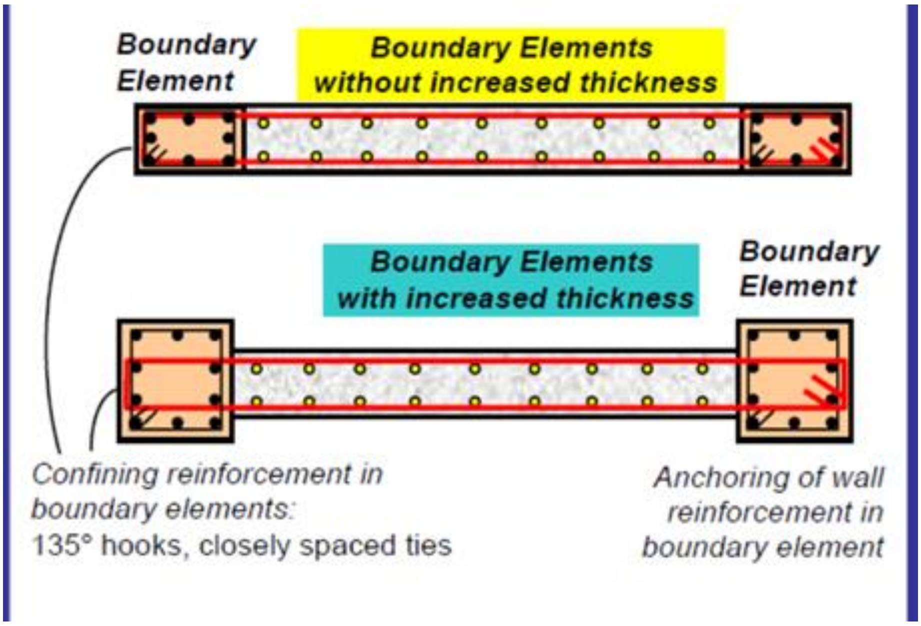


Fig. 1.2.3 Boundary elements in Shear wall

1.3 TYPE OF FORCES SHEAR WALL RESIST

1.3.1 Shear forces:

Connections to the structure above transfer horizontal forces to the shear wall. This transfer creates shear forces throughout the height of the wall between the top and bottom shear wall connections. The strength of the lumber, sheathing and fasteners must resist these shear forces or the wall will tear or “shear” apart.

1.3.2 Uplift forces

Uplift forces exist on shear walls because the horizontal forces are applied to the top of the wall. These uplift forces try to lift up one end of the wall and push the other end down. In some cases, the uplift force is large enough to tip the wall over. Uplift forces are greater on tall short walls and less on low long walls. Bearing walls have less uplift than non-bearing walls because gravity loads on shear walls help them resist uplift. Shear walls need holdown devices at each end when the gravity loads cannot resist all of the uplift. The holdown device then provides the necessary uplift resistance.

1.4 FUNCTIONS OF SHEAR WALL

1.4.1Lateralstrength: Shear walls must provide the necessary lateral strength to resist horizontal earthquake forces. When shear walls are strong enough, they will transfer these horizontal forces to the next element in the load path below them. These other components in the load path may be other shear walls, floors, foundation walls, slabs or footings.

The strength of the shear wall depends on the combined strengths of its three components: lumber, sheathing and fasteners. When all of the components are properly in place, the shear wall can provide its intended strength. For shear wall sheathing, the 1994 Uniform Building Code (UBC) permits the use of gypsum wallboard, cement plaster, fiberboard, wood particleboard, plywood and oriented strand board. Previous editions of the UBC also allowed wood lath and plaster, horizontal and diagonal sheathing for shear walls. All of these sheathing materials provide different strengths. The UBC shows these strengths in pounds per foot of wall length. Fasteners for shear wall construction may be staples, screws or nails. Denser lumber species provide stronger fastener strengths. Values for shear wall strengths assume a dense lumber species like douglas fir-larch or southern pine. Thicker framing members also increase wood structural panel sheathing strengths.

1.4.3 Lateral stiffness: 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 nonstructural damage.

The stiffness of the shear wall, just like its strength, depends on the combined stiffness of its three components: lumber, sheathing and fasteners. The size and grade of end stud, thickness and grade of sheathing, and the sheathing fastener diameter determine how flexible a wood shear wall will be. When present, holdown devices also contribute to the overall stiffness of the shear wall. If holdown devices stretch or slip, the top of the shear wall will move horizontally. This horizontal movement adds to the movement allowed by the lumber, sheathing and fasteners. Any additional movement from the holdown will reduce the effective stiffness of the shear wall.

Shear walls provide stiffness in large part by the ratio of their height to width. Long short walls are stiffer than tall narrow ones. For a wall of constant height, the stiffness will grow exponentially as the wall length increases. To help control stiffness, the UBC requires a minimum wall length for any given wall height. This allowable dimension ratio changes for each type of sheathing material and its construction. Wood structural panels can have smaller shear wall lengths than cement plaster or gypsum wallboard. When this sheathing is fastened at all of its edges, the UBC also permits smaller shear wall lengths.

1.5 <u>CLASSIFICATION</u>

- Shear walls- deflection and strength controlled by shear.

- Ordinary moment shear wall- deflection and strength by flexure

- Ductile moment shear walls- for seismic regions, have good energy dissipation under reversed cyclic loads.

1.6 <u>CENTRE OF RIGIDITY</u>

The point that locates the position of a story shear force which will cause only relative floor translations. It is also referred as centre of stiffness of a system.

- Point on the horizontal plane through which the lateral load should pass in order that there will be no rotation.

- Earthquake induced lateral forces are proportional to the mass, resultant forces due to earthquake passes through the Centre of Mass of floor.

- In building where Centre of Rigidity-Centre of Mass, therefore no torsion. Loads are taken by shear walls in proportion to stiffness, otherwise torsion to be analyzed.

- For wind load Centre of Area = Centre of Rigidity

CHAPTER 2

SEISMO-RESISTANT STRUCTURE

2.1 SEISMO-RESISTANT BUILDING ARCHITECTURE

Objective to prevent stepping of seismo-resistant capacity of building and to optimize seismo resistance in which major aspects involves are:

1. Selection of load resisting system
2. Configuration system.
3. Basic dynamic characteristic.
4. Construction quality

BIS (Bureau of Indian Standards) has approved 3 types of load resisting system in IS 1893 Part

I. Building frame system.
II. Bearing wall system
III. Dual system response

2.2 LATERAL LOAD RESISTING SYSTEMS

Many buildings consist of mixtures of the basic types of the lateral resistive systems. Walls existing with a frame structure, although possibly not used for gravity loads, can still be used to brace the frame for lateral loads. Shear walls may be used to brace a building in one direction whereas a braced frame or rigid frame is used in the perpendicular direction. Multistory buildings occasionally have one type of system, such as rigid frame, for the upper stories and a different system, such as a box system or braced frame, for the lower stories to reduce deformation and take the greater loads in the lower portion of the structure. In many cases it is neither necessary nor desirable to use every wall as a shear wall or to brace every bay of the building frame. This procedure does require that here be some load-distributing elements, such as the roof and floor diaphragms, horizontal struts, and so on, that serve to tie the unsterilized portions of the building to the lateral resistive elements.

There is a possibility that some of the elements of the building construction that are not intended to function as bracing elements may actually end up taking some of the lateral load. In frame construction, surfacing materials, plaster, dry wall, wood paneling, masonry veneer, and so on may take some lateral load even though the frame is braced by other means. This is essentially a matter of relative stiffness, although connection for load transfer is also a consideration. The choice of the type of lateral resistive system must be related to the loading conditions and to the behavior characteristics required. It must also, however, be coordinated with the design for gravity loads and with the architectural planning considerations. Many design situations allow for alternatives, although the choice may be limited by the size of the building, by code restrictions, by the magnitude of lateral loads, by the desire for limited deformation, and so on. Different types of lateral load resisting systems are shown in Fig. 2.1.1

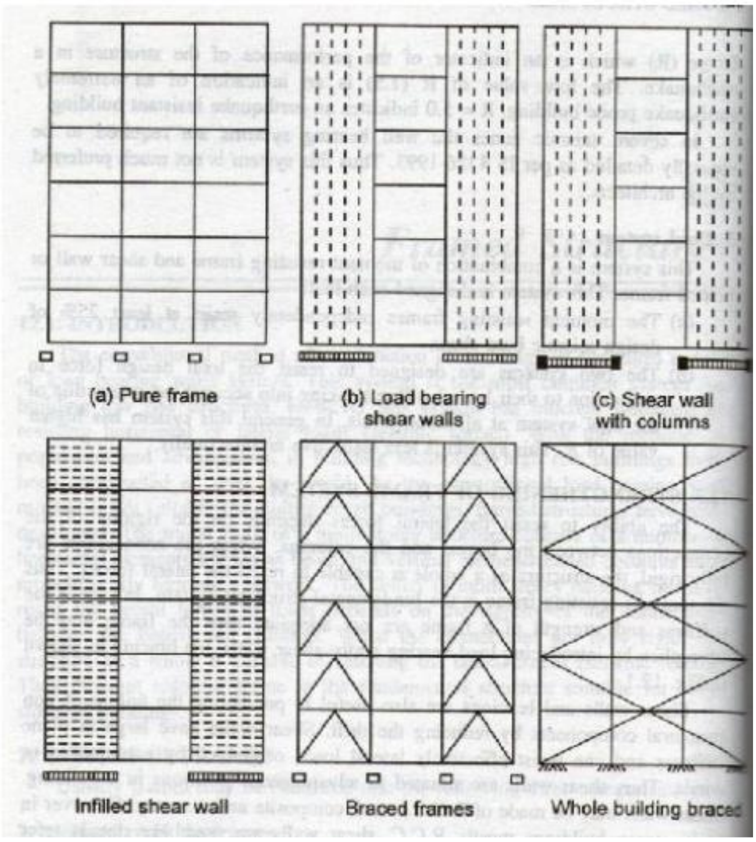


Fig 2.1.1 Load resisting frames

2.3 MOMENT- RESISTANT FRAME

There is some confusion over the name to be used in referring to frames in which interactions between members of the frame include the transfer of moments through the connections. In years past the term most frequently used was rigid frame. This term primarily from the classification of the connections or joints of the frame as fixed (or rigid) versus pined, the later term implying a lack of capability to transfer moment through the joint. As a general descriptive term, however, the name was badly conceived, since the frames of this type were generally the most deformable under lateral loading when compared to trussed frames or those braced by vertical diaphragms.

In rigid frames with moment-resistive connections, both gravity and lateral loads produce interactive moments between the members. In most cases rigid frames are actually the most flexible of the basic types of lateral resistive systems. This deformation character, together with the required ductility, makes the rigid frame a structure that absorbs energy loading through deformation as well as through its shear brute strength. The net effect is that he structure actually works les hard in force resistance because its deformation tends to soften the loading. Most moment-resistive frames consist of either steel or concrete. Steel frames have either welded or bolted connections between the linear members to develop the necessary moment transfers. Frames of concrete achieve moment connections through the monolithic concrete and the continuity and anchorage of the steel reinforcing. Because concrete is basically brittle and not ductile, the ductile character is essentially produced by the density of the reinforcing. The type and amount of reinforcing and the details of its placing become critical to the proper behavior of rigid frames of reinforcing concrete. For lateral loads in general, the rigid frame offers the advantage of a high degree of freedom in architectural terms. Walls and interior spaces are freed of the necessity for solid diaphragms or diagonal members. For building planning as a whole, this is a principle asset. Walls, even where otherwise required to be solid, need not be of a construction qualifying them as shear walls.

2.4 BRACED FRAMES

Bracings are the lateral resistive system used for reduction of responses and earthquake induced torsion in the building. Although there are actually several ways to brace a frame against lateral loads, the term braced frame is used to refer to frames that utilize trussing as the primary bracing technique. In buildings, trussing is mostly used for the vertical bracing system in combination with the usual horizontal diaphragms. It is also possible, however, to use a trussed frame for a horizontal system, or to combine vertical and horizontal trussing in a truly three-dimensional trussed framework. The latter is more common for open tower structures, such as those used for electrical transmission lines and radio and television transmitters.

2.5 SHEAR WALL BASED FRAMES

Shear walls are the lateral resistive system used for reduction of responses and earthquake induced torsion in the building. Shear walls are the walls constructed in structures to resist lateral load or forces developed due to wind or earthquake. Shear walls have a very large in plane stiffness and thus resist lateral loads and control deflection very efficiently. Shear wall being flexible in the perpendicular plane, they can transfer the lateral forces in their own plane by developing movement and shear resistance. Regarding the shapes of shear wall Rectangular-type, C-type and L-type cross section are used, out of which rectangular type is common. In the present study, the main focus is on the reduction of seismic induced torsion by providing shear walls.

2.6 BUILDING CHARACTERISTICS

The seismic forces exerted on a building are not externally developed forces like wind instead they are the response of cyclic motions at the base of a building causing accelerations and hence inertia force.

- The response is therefore essentially dynamic in nature.

- The dynamic properties of the structure such as natural period, damping and mode shape play a crucial role in determining the response of the building.

- Besides other characteristics of the building system also affect the seismic response such as ductility, building foundation, response of non-structural elements etc.

2.6.1 Mode Shapes and Fundamental Period

- The elastic properties and mass of building cause to develop a vibratory motion when they are subjected to dynamic action.

- This vibration is similar to vibration of a violin string, which consists of a fundamental one and the additional contribution of various harmonics.

- The vibration of a building likewise consists of a fundamental mode of vibration and the additional contribution of various modes, which vibrates at higher frequencies.

- Fundamental period of vibration can be determined by the code-based empirical for the fundamental modes of the building may be determined by any one of several methods developed for the dynamic analysis of structures.

- On the basis of time period, building may be classified as rigid ($T< 0.3$ sec), semi – rigid(0.3 sec $< T < 1.0$ sec) and flexible structure ($T > 1.0$ sec).

- Buildings with higher natural frequencies, and a short natural period, tend to suffer higher accelerations but smaller displacement.

- In the case of buildings with lower natural frequencies, and a long natural period, this is reversed: the buildings will experience lower accelerations but larger displacements.

2.6.2 Building Frequency and Ground Period

- Inertial forces generated in the building depend upon the frequencies of the ground on which the building is standing and the building's natural frequency.

- When these are near or equal to one another, the building's response reaches a peak level.

- Past studies show that the predominant period at a firm ground site 0.2 – 0.4 sec rigid structure (0-0.3) will have more unfavorable seismic response than flexible structures, while period on soft ground can reach 2.0 sec or more.

- Seismic response of flexible structures (t>1.0) on soft foundation sites will be less favorable than that of rigid structure.

- Building fundamental periods of approximately 0.1N (where, N is the number of storey).

2.6.3 Damping

- The degree of structural amplification of the ground motion at the base of the building is limited by structural damping.

- Damping is the ability of the structural system to dissipate the energy of the earthquake ground shaking.

- Since the building response in inversely proportional to damping, the more damping in a building possesses, the sooner it will stop vibrating--which of course is highly desirable from the standpoint of earthquake performance.

- In a structure, damping is due to internal friction and the absorption of energy by the building's structural and non-structural elements.

- There is no numerical method available for determining the damping. It is only obtained by experiments.

2.6.4 Ductility

- Ductility is defined as the capacity of the building materials, systems, or structures to absorb energy by deforming in the inelastic range.

- The safety of building from collapse is on the basis of energy, which must be imparted to the structure in order to make it fail. In such instance, consideration must be given to structure's capacity to absorb energy rather than to its resistance.

- Therefore ductility of a structure in fact is one of the most important factors affecting its earthquake performance.

- The primary task of an engineer designing a building to be earthquake resistant is to ensure that the building will possess enough ductility

- The ductility of a structure depends on the type of material used and also the structural characteristics of the assembly.

- It is possible to build ductile structures with reinforced concrete if care is taken in the design to provide the joints with sufficient abutments that can adequately confine the concrete, thus permitting it to deform plastically without breaking.

2.6.5 Seismic Weight

- Seismic forces are proportional to the building weight and increases along the height of building.

- Weight reduction can be obtained by using lighter materials or by reducing the filling and other heavy equipment's not essential for building construction.

- **2.6.6 Hyperstaticity / Redundancy**

- Hyper static (statically indeterminate) structures have advantage because if primary system yields or fails, the lateral force can be redistributed to secondary elements or system to prevent progressive failure (alternate load path).

- Moreover, Hyperstaticity of the structure causes the formation of plastic hinges that can absorb considerable energy without depriving the structure of its stability.

2.6.7 Quality of Construction and Materials

- Grade of concrete not achieved in site – reasons.

- Poor execution of the concrete joint/ discontinuity- quality of concrete

- Reinforcement detailing not taken care of appropriately.

- Accumulation of sawdust, dust and loose materials at the surface of joint.

- Result: A defective concrete joint, which contributed significantly to causing of failure of many building in past earthquakes.

2.7 SEISMIC STRENGTHNING OF RC STRUCTURES WITH EXTERIOR SHEAR WALLS

Many reinforced concrete (RC) buildings have either collapsed or experienced different levels of damage during past earthquakes. Many investigations have been carried out on buildings that were damaged or ruined by earthquakes. Low-quality concrete, poor confinement of the end regions, weak column-strong beam behavior, short column behavior, inadequate splice lengths and improper hooks of the stirrups were some of the important structural deficiencies Most of those buildings were constructed before the introduction of modern building codes. They usually cannot provide the required ductility, lateral stiffness and strength, which are definitely lower than the limits imposed by the modern building codes. Due to low lateral stiffness and strength, vulnerable structures are subjected to large displacement demands, which cannot be met adequately as they have low ductility.

Nowadays, most of the strengthening strategies are based on global strengthening schemes as per which the structure is usually strengthened for limiting lateral displacements in order to compensate the low ductility. In these schemes, global behavior of the system is transformed. Another approach is modification of deficient elements to increase ductility so that the deficient elements will not reach their limit state conditions when subjected to design loads. However, the latter strategy is more expensive and harder to implement in cases of many deficient elements which is the reason that the global strengthening methods have been more popular than element strengthening.

Among the global strengthening methods, addition of RC infill is the most popular one. Many researchers have focused on this subject and found that installation of RC infills greatly improve lateral load capacity and stiffness of the structure. Even in cases of application to damaged buildings, the infill method yields satisfactory results. In some other researches, the use of wing walls, attached to two sides of columns was investigated. The systems strengthened with wing walls exhibited ductile behavior.

Steel bracing for RC frames has also been used to reduce drift demands. Bracing can either be implemented inside the frame or applied from outside the system. Post-tensioned steel bracing is also an efficient alternative for vulnerable framed buildings and it compensates structural irregularities. Experimental results for another alternative, knee bracing with shear links replaced with masonry infills, lead to improvement in energy absorption capacity. Although, each of these methods satisfactorily increased the strength and stiffness, all of them with the exception of external steel bracing require construction work inside the building, which means disturbance ofusers and results in the buildings being out of service. Consequently, research efforts in this field have shifted their focus to new methods that could overcome this difficulty. The precast panel infill method, which causes less disturbance for the building occupants, has been investigated and found to be an efficient solution for strengthening of existing structures. Despite causing some architectural problems, some other researchers perpendicularly installed RC shear walls outside the building. This kind of shear walls was also applied to precast skeletal structures with an external diaphragm at the roof level. This method has increased the lateral load capacity and strength of the structure as well. It should also be noted that the method requires the sides of the buildings to be unobstructed for installation of new shear walls.

The literature review presents numerous strengthening techniques. However, most of them require long-term construction works inside the building, rendering the building out of service for that period of time. On the other hand, external strengthening techniques offer advantages with respect to cost and ease of construction. This study investigates the performance of exterior RC shear walls (ESW) that are placed parallel to the building's sides.

In reality, installing a shear wall to a structural system will surely improve the seismic capacity of the structure. The main concern is whether the design methods for the connection of old and new elements can satisfy codes. To make it clear, an experimental program was carried out on six-storey three-dimensional RC models. The program includes a reference model and a strengthened model. Additionally, numerical solutions are presented and compared with the results of the experiments.

Hence concluding, it is observed and that the newly added external shear wall and the connected end columns and beams behave like a monolithic member. Minor cracks between new and existing elements have been formed after 1% drift. Even after minor cracks developed, the shear walls did not lose their load bearing capacity.

The crack occurs at the bottom of the exterior shear walls due to bending in initial stages of the experiment. Sliding shear capacity of the shear walls droop due to the rupturing of the longitudinal bars and in addition, shear sliding behavior was observed at the bottom of the walls. This had an adverse effect on ductility and energy absorption capacity of the system. To prevent such damage, additional shear reinforcement is required at the web of the wall.

2.8 EFFECT OF INTERNAL AND EXTRNAL SHEAR WALL LOCATIONS

Research has shown that the most effective and economic method of increasing the stiffness and lateral load strength of existing buildings is adding new elements to the current building system. It is known that the bearing system can attain adequate stiffness and strength levels through strengthening by adding a SW to the existing construction system. The selection of especially the location and amount of SWs is of utmost importance in strengthening, which can be accomplished by adding a SW so as not to exert further pressure on the existing RC system that is already weak.

Strengthening SWs may come out in various positions according to their positions in the plan. The method of filling the gaps between columns of the bearing system with complete or partial SWs is defined as the interior SW. However, the method of filling, especially the frame gaps, with SW has some architectural and applicability difficulties. System strengthening performed by filling the frame gaps fully with SW mostly causes an architectural function change and accordingly the alteration of the interior SW gives rise to serious economic loss. On the other hand, in the application of partial interior SW changes occurring in use can be partially lessened considering the gaps that necessitating as per their usage aims.

The most significant disadvantage of the method of strengthening with interior SW is that crucial problems arise in especially widely used public buildings, such as schools and hospitals, since the strengthened building cannot be used for a long time. Being motivated by this problem, research has been carried out with the supposition that shifting the location of the SW from the interior to the exterior of the building is a rapidly and easily applicable strengthening method especially in public buildings. The most important disadvantage of strengthening with exterior SW is the need for a large area for the change in use of the exterior front and the ground of the exterior SW.

The primary problem in strengthening the existing RC buildings with SW, which are defined as deficient against earthquake, is determining the location of the SW. The location of SW is of primary importance in many respects, such as structural performance, applicability, re-assessment in utilization process, economy and environmental conditions.

The issue of the location of the added new RC SW has maximum meaning in spatial asymmetric multi-storey buildings. For this reason, the effect of SW location should be researched for asymmetric multi-storey buildings under real loading conditions, such as a shaking table. The most important parameter that should be considered during the strengthening process of a RC building was the performance of the building against earthquake effects. On the other hand, the architectural function and environmental condition of the building should also be evaluated extensively. Additionally, the cost analysis for the strengthening process of the building should be another significant factor during the analyses. According to the considerations within the light of these criteria, if the buildings are not adjacent to one another and extensively used (public buildings such as schools and hospitals) without any commercial story and constructional overhangs, such as balconies and consoles, the application of exterior SW can be recommended.

Even though it can be applied to a limited number of buildings, the greatest advantage of the application of exterior SW is the fact that there is no need for the user to evacuate the building during the strengthening process and an additional alteration is not required inside the building.

On the other hand, though the interior shear wall can be applied commonly to all kinds of buildings, the user has to evacuate the building during the strengthening process and post-strengthening additional reparation and, therefore, alteration costs arise. In earthquake prone countries and regions, where most of the building stock is composed of RC buildings with poor earthquake strength, the mentioned buildings should be urgently strengthened by considering the conditions, importance and architecture of the building so as to prevent further losses.

2.9 IDENTIFICATION OF DAMAGES IN RC BUILDINGS

- Soft storey failure- Multi-storey buildings, first storey for parking of vehicles/ banking hall etc. due to this first storey has lesser strength and stiffness leads to concentration of forces on second floor, plastic hinge formation.

- Floating column- balconies extended, therefore columns discontinued cantilever beams, leading to overturning forces in the column.

- Plan and mass irregularity- irregularity in plan, mass, stiffness results in significant torsional response, excess mass – large inertia forces, reduced ductility and increased propensity due to p-effect. Irregularity of mass distribution results in irregular response and complex dynamics(Fig. 2.9.1).

- Pounding of buildings: pounding is the result of irregular response of adjacent building of different heights and of different dynamics characteristics damage due to pounding can be minimized by drift control, building separation and aligning floor in adjacent level.

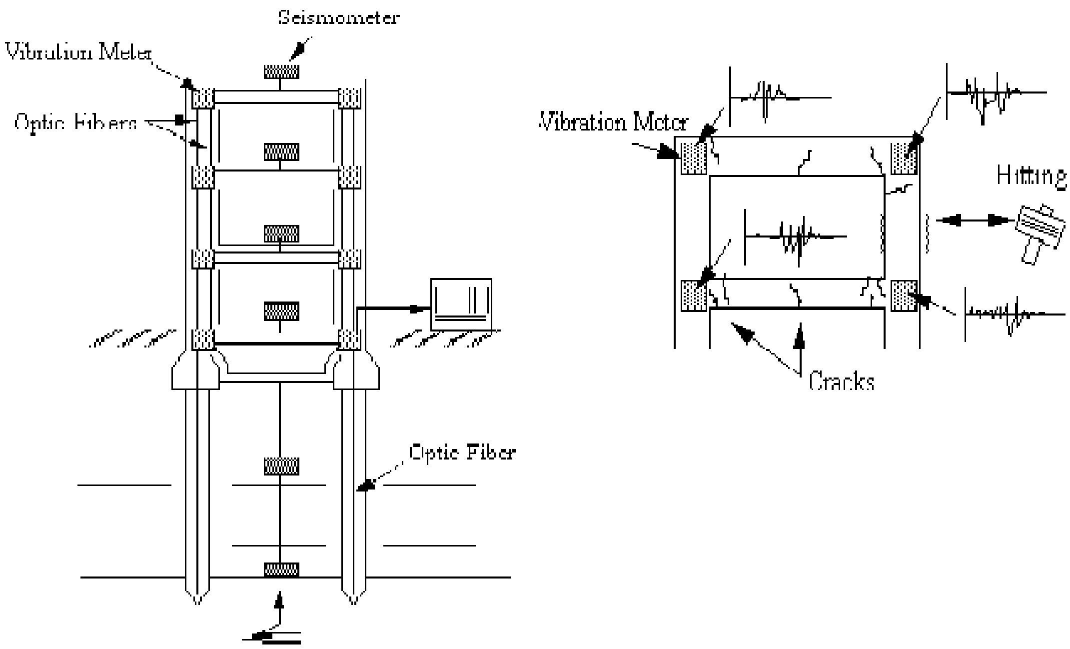


Fig .2.9.1 Identification of damages

2.10 IS 1893-2000 PART 1

2.10.1Cl 3.27 Response Spectrum

The representation of the maximum response of idealized single degree 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.

2.10.2 Cl 3.33Zone Factor (Z)

It is a factor to obtain the design spectrum depending on the perceived maximum seismic risk characterized by Maximum Considered Earthquake (MCE) in the zone in which the structure is located. The basic zone factors included in this standard are reasonable estimate of effective peak ground acceleration.

Zone Factor, *Z*

(*Clause* 6.4.2)

Seismic Zone	II	III	IV	V
Seismic Intensity	Low	Moderate	Severe	Very Severe
Z	0.10	0.16	0.24	0.36

Table 2.10.1 Zone factor Cl 6.4.2

2.10.3 Cl 3.14 Importance Factor

It is a factor used to obtain the design seismic force depending on the functional use of the structure, characterized by hazardous consequences of its failure, its post-earthquake functional need, historic value, or economic importance.

Importance Factors, *I*

(*Clause* 6.4.2)

Sl No.	Structure	Importance Factor
(1)	(2)	(3)
i)	Important service and community buildings, such as hospitals; schools; monumental structures; emergency buildings like telephone exchange, television stations, radio stations, railway stations, fire station buildings; large community halls like cinemas, assembly halls and subway stations, power stations	1.5
ii)	All other buildings	1.0

NOTES

1 The design engineer may choose values of importance factor *I* greater than those mentioned above.

2 Buildings not covered in Sl No. (i) and (ii) above may be designed for higher value of *I*, depending on economy, strategy considerations like multi-storey buildings having several residential units.

3 This does not apply to temporary structures like excavations, scaffolding etc of short duration.

Table 2.10.2 Cl 3.14 Importance factor

CHAPTER 3

LOAD CALCULATIONS

3.1 PROBLEM DEFINITION

In the present work, a 5-storey apartment building has been considered for various locations of shear walls. The frame is shown in Fig. 3.1 following are the details of the frame

Number of storeys = 5,6,7&8

Storey height = 3.5m

Width of bays in X direction = 4

Width of bays in Y direction = 4

Size of primary beam = 300*400mm

Size of secondary beam = 200*300mm

Size of columns = 400*600mm

Slab thickness = 150 mm

Concrete grade = M30

Steel reinforcement grade = Fe 415

Seismic zone of site = Zone II & Zone III

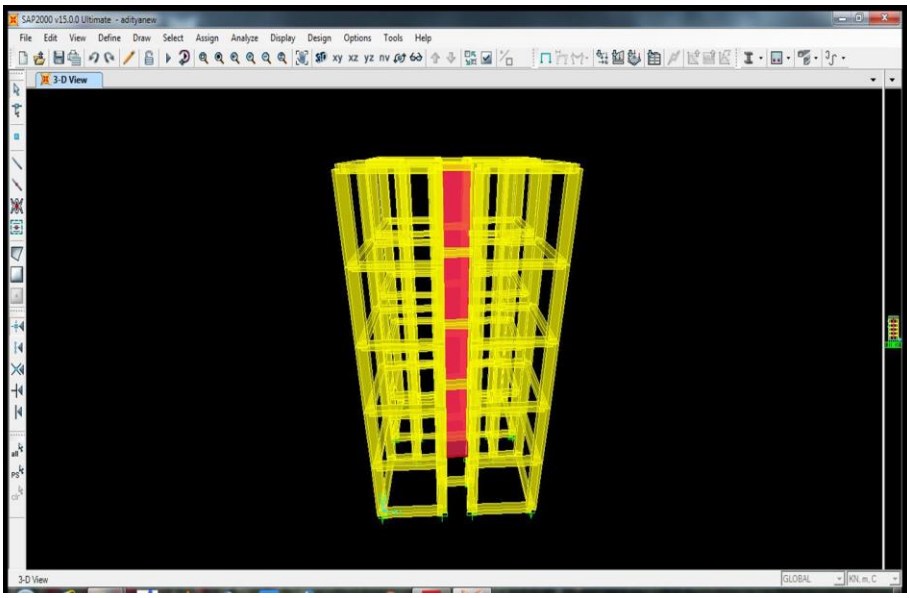


Fig. 3.1 Model 1

3.2 DEAD LOAD (IS 875 Part 1 -1987)

1. Thickness of slab = 150mm
2. Thickness of wall = 100mm
3. Height of wall on roof = 1m
4. Height of floor = 3.5m

A. Roof Slab

1. DL due to wall = 1.91kN/m
2. DL due to slab = 2.5kN/m^2
3. DL due to slab on beam = 10kN/m

   TOTAL DL = 11.91kN/m

B. Floor slab

1. DL due to wall = 6.685 kN/m
2. DL due to slab on beam = 10 kN/m

   TOTAL DL = 16.685kN/m

C. Corridor

1. DL due to wall = 6.685 kN/m
2. DL due to slab on beam = 5 kN/m

TOTAL DL = 11.68 kN/m

3.3 LIVE LOAD (IS 875 Part 2- 1987)

A. Roof

1. LL on beam = 3kN/m

On Corridor

1. LL on beam = 1.5kN/m

B. Floor

1. LL on beam = 16kN/m

On Corridor

1. LL on beam = 8kN/ m

CHAPTER 4

MODELING

4.1 MODEL 1:5 STOREY TOWER

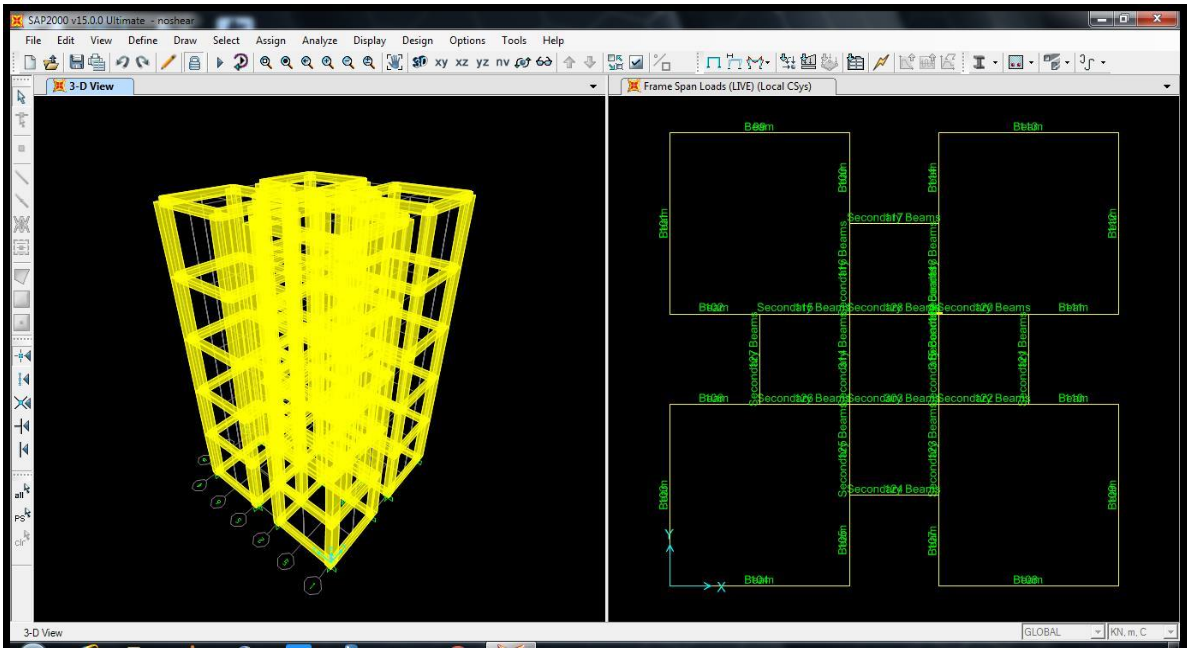


Fig. 4.1.1 No Shear Wall M-1

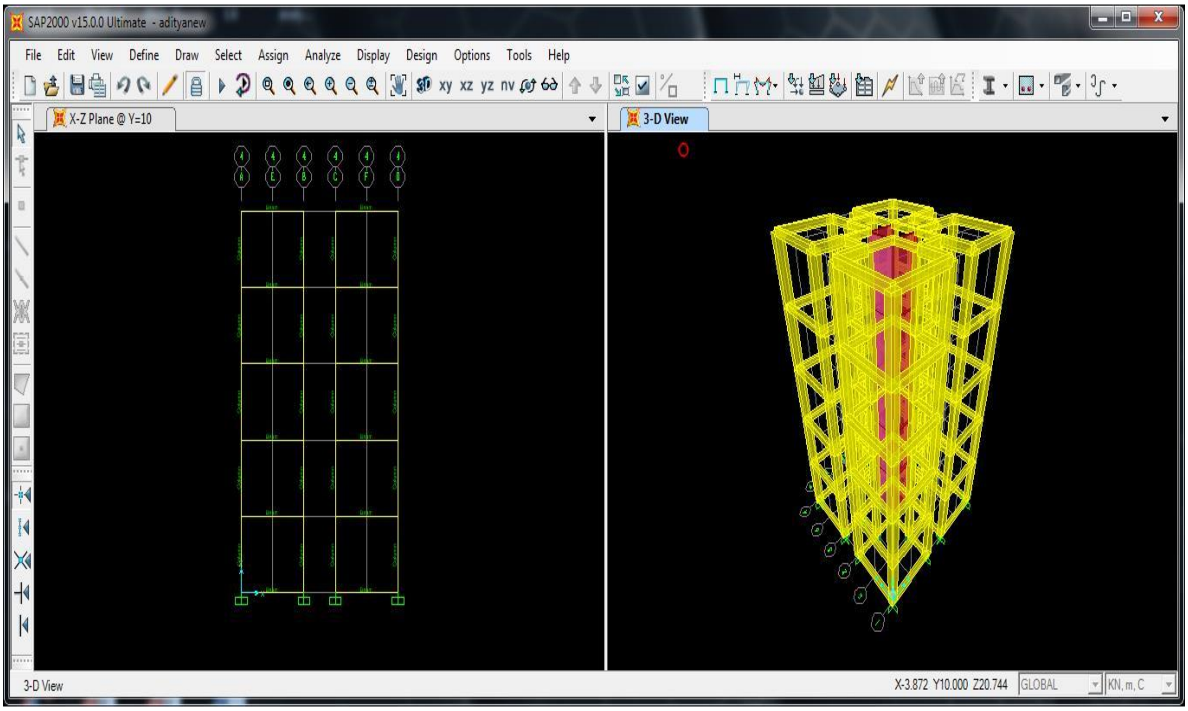


Fig. 4.1.2 Interior Shear Wall M-1

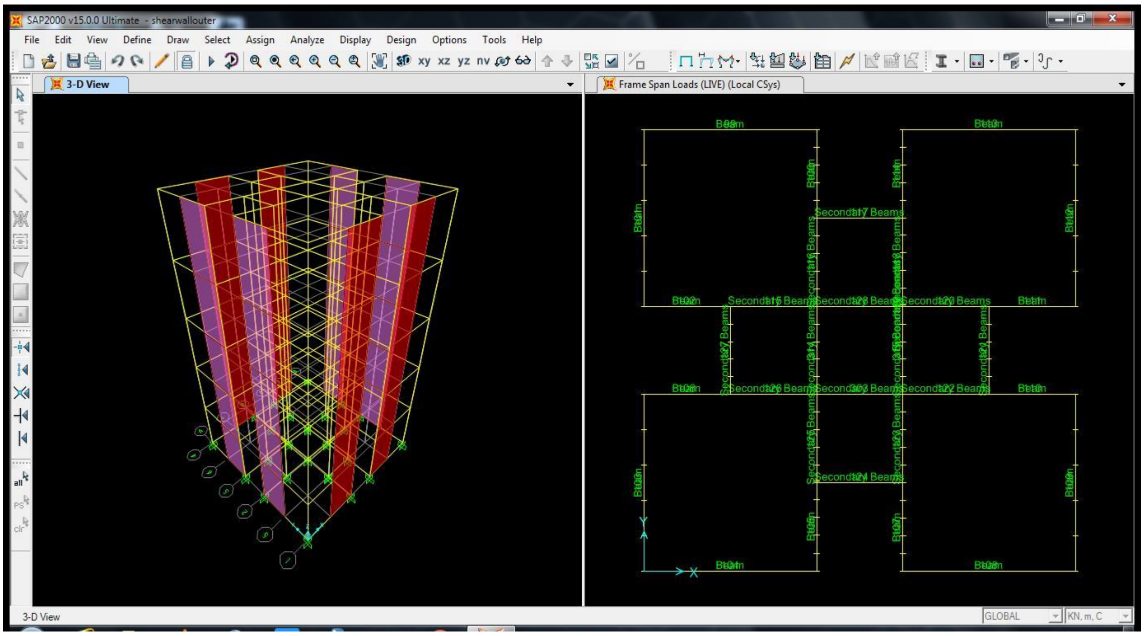


Fig. 4.1.3 Exterior Shear wall

	Without Shear Wall	Interior Shear Wall	Exterior Shear Wall
Max Axial Force	43.636KN/m	44.559KN/m	41.605KN/m
Max Shear Force	103.519KN/m	85.626KN/m	83.339KN/m
Max Moment	107.274KN-m/m	86.277KN-m/m	82.673KN-m/m

Table 4.1 Max forces in members M-1

4.2 MODEL 2: 6 STOREY TOWER

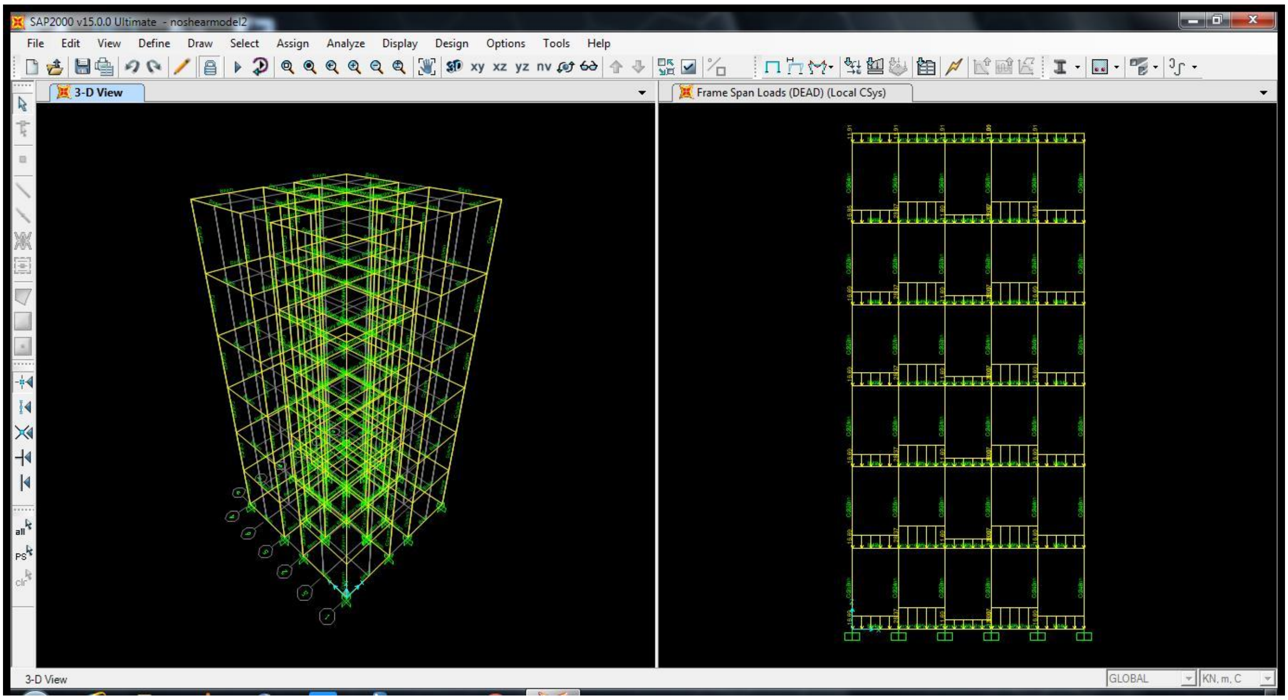


Fig. 4.2.1Without Shear wall M-2

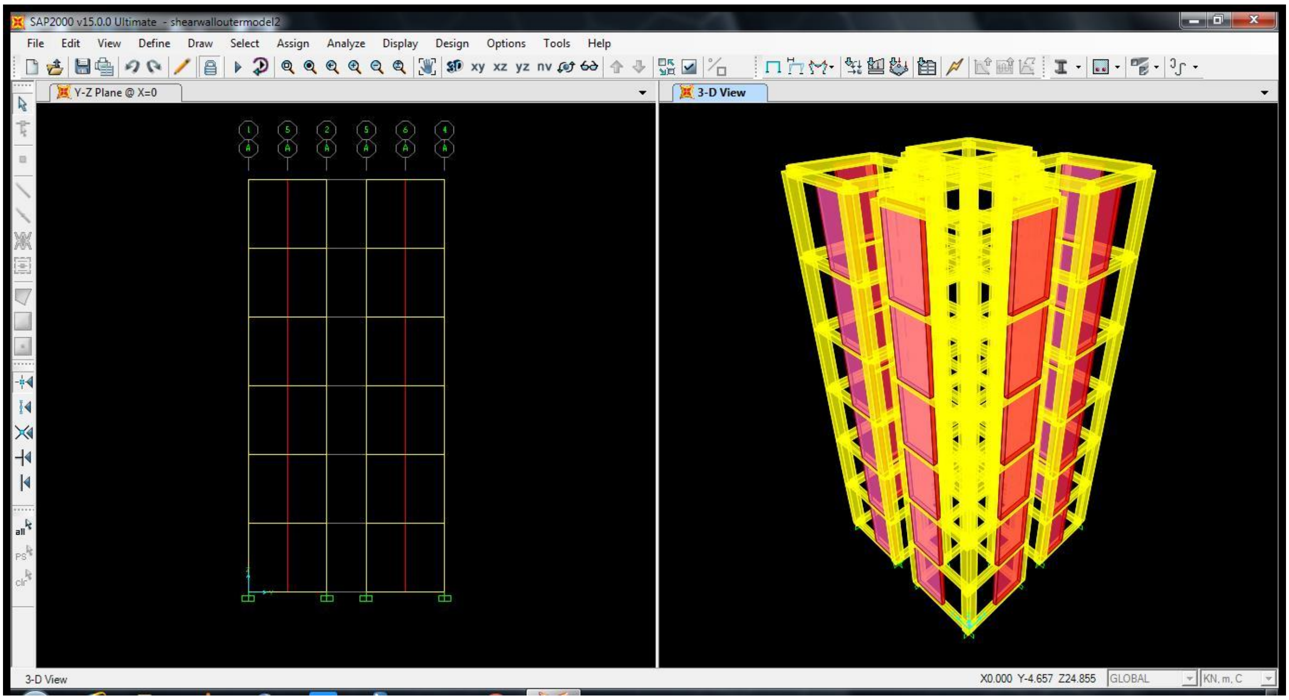


Fig. 4.2.2Exterior Shear Wall

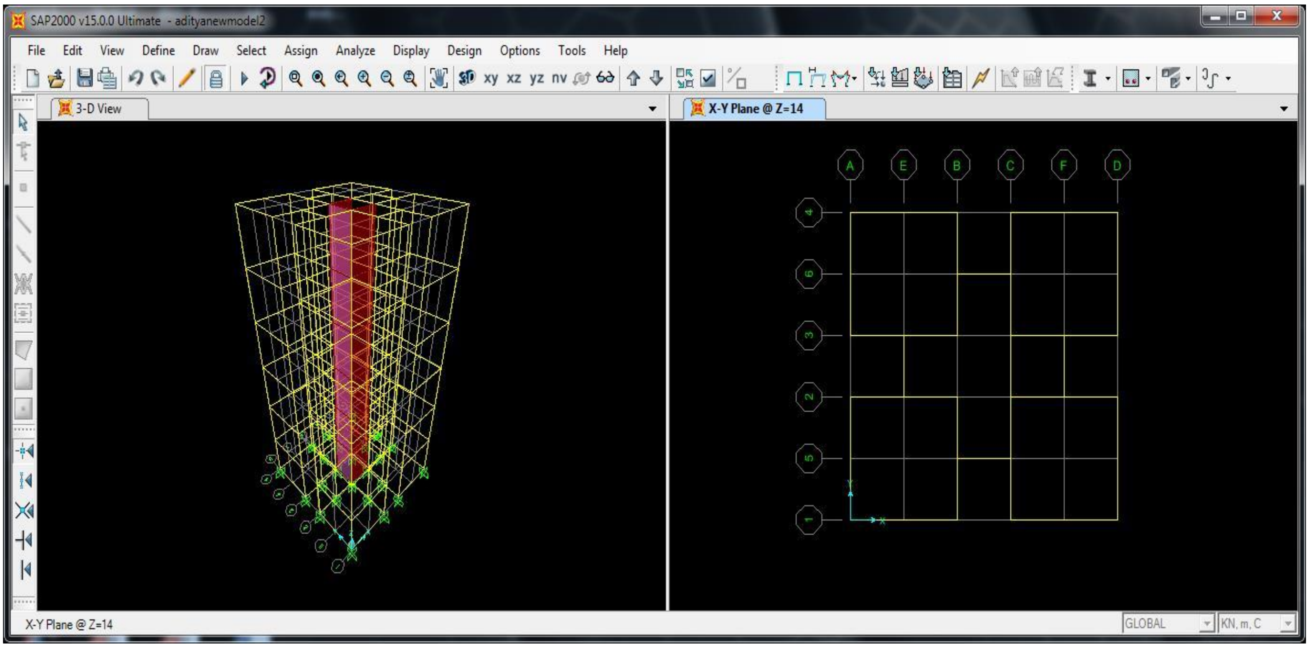
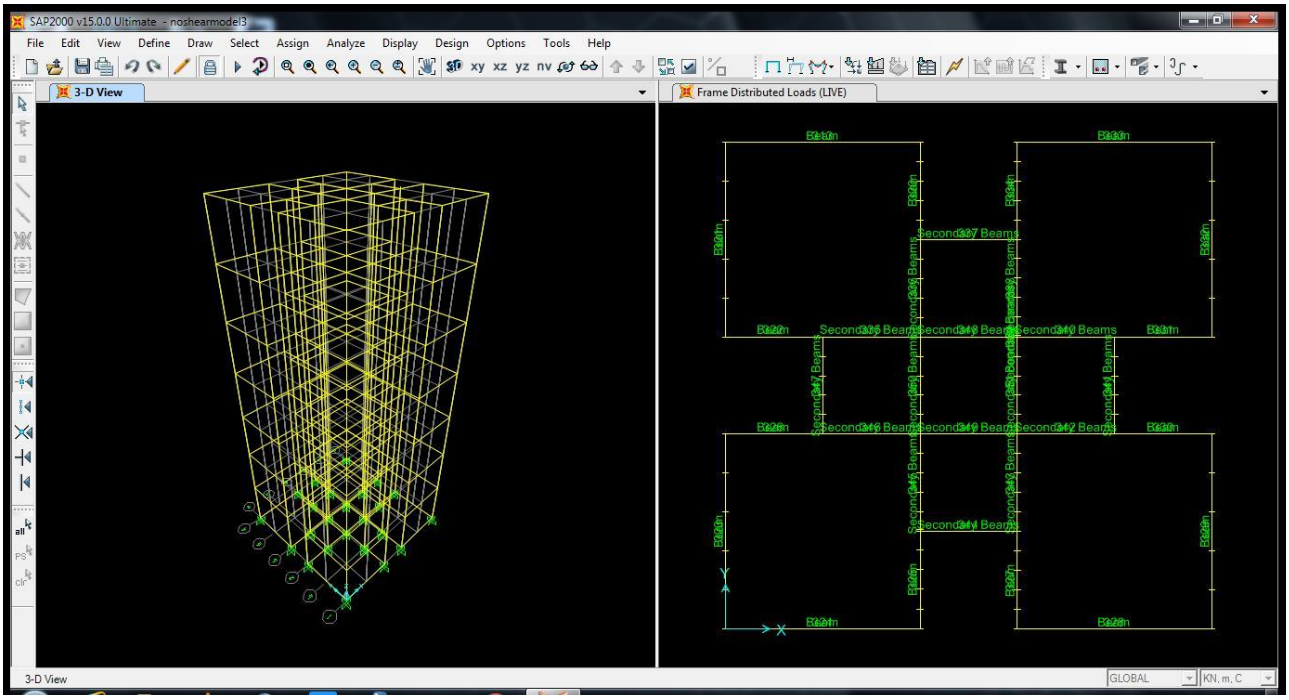


Fig. 4.2.3 Interior Shear Wall

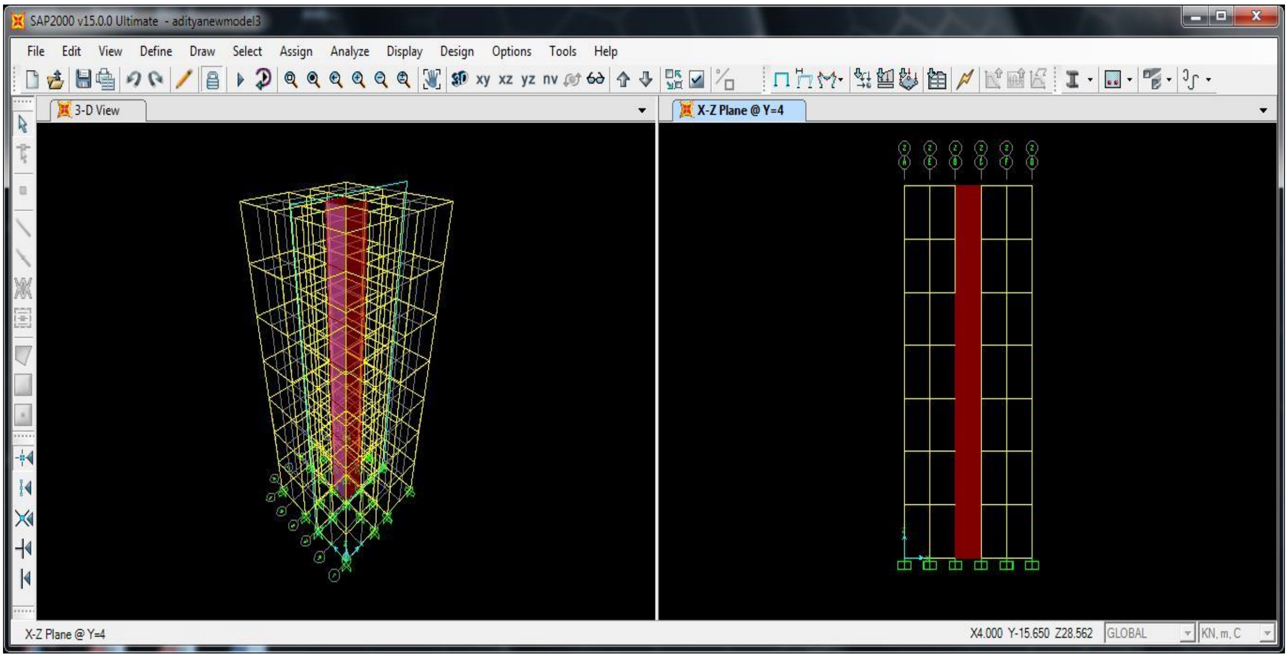
	Without Shear Wall	Interior Shear Wall	Exterior Shear Wall
Max Axial Force	57.129KN/m	41.624KN/m	42.638KN/m
Max Shear Force	111.768KN/m	107.108KN/m	94.745KN/m
Max Moment	117.649KN-m/m	94.297KN-m/m	96.057KN-m/m

Table 4.2 Max forces in members M-2

4.3<u>MODEL 3: 7 STOREY TOWER</u>



<u>Fig 4.3.1Without Shear Wall M-3</u>



<u>Fig. 4.3.2 Interior shear wall</u>

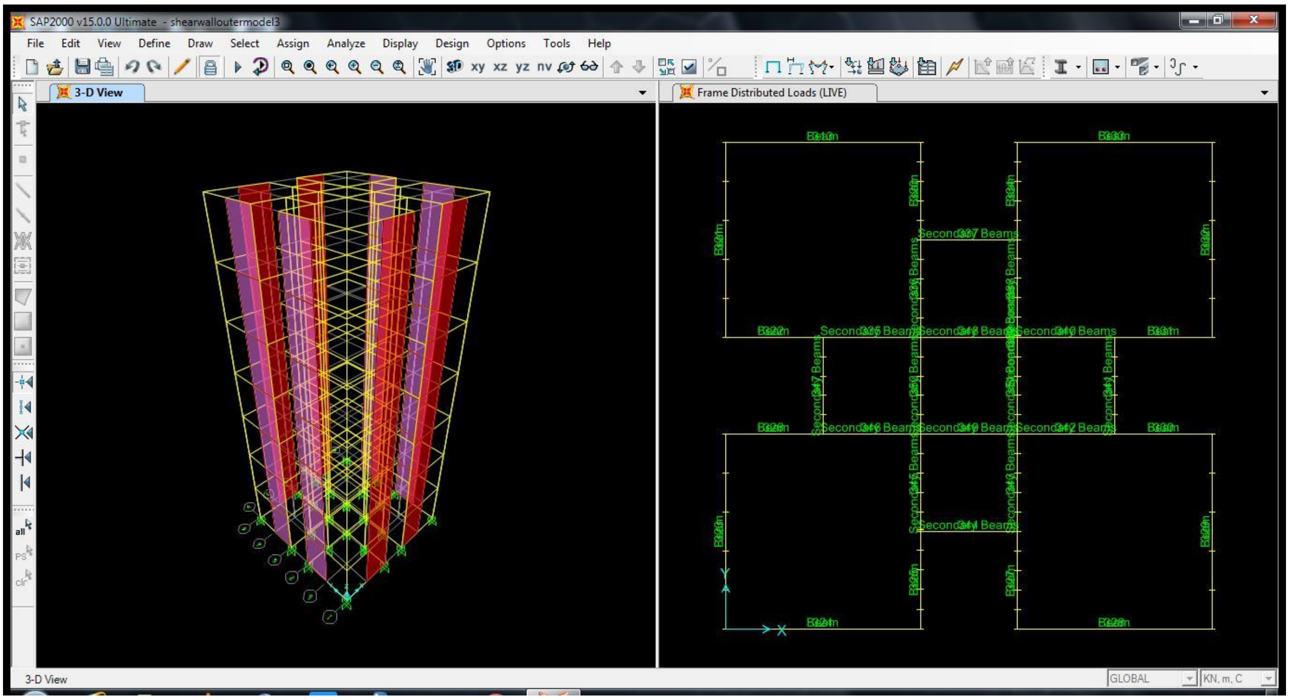


Fig. 4.3.3 Exterior shear wall

	Without Shear Wall	Interior Shear Wall	Exterior Shear Wall
Max Axial Force	81.681KN/m	44.394KN/m	59.295KN/m
Max Shear Force	162.743KN/m	118.145KN/m	122.736KN/m
Max Moment	147.629KN-m/m	108.392KN-m/m	110.724KN-m/m

Table 4.3 Max forces in member M-3

4.4 MODEL 4: 8 STOREY TOWER

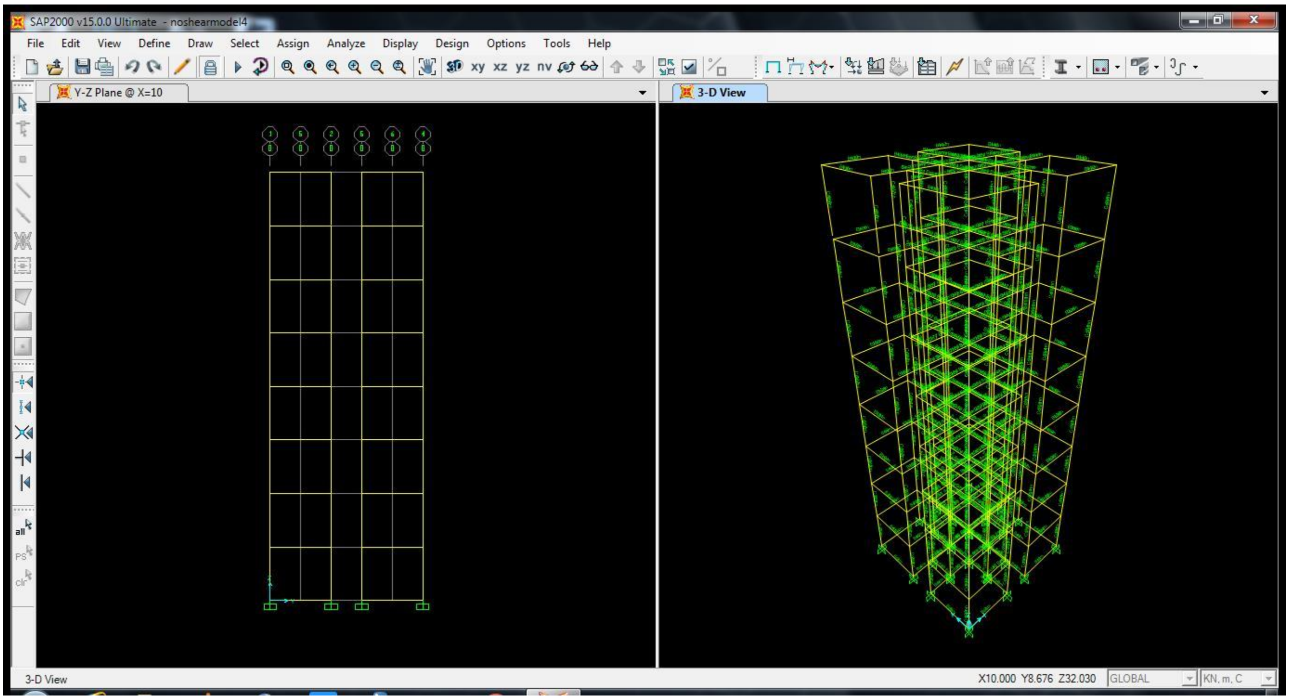


Fig. 4.4.1 Without Shear wall M-3

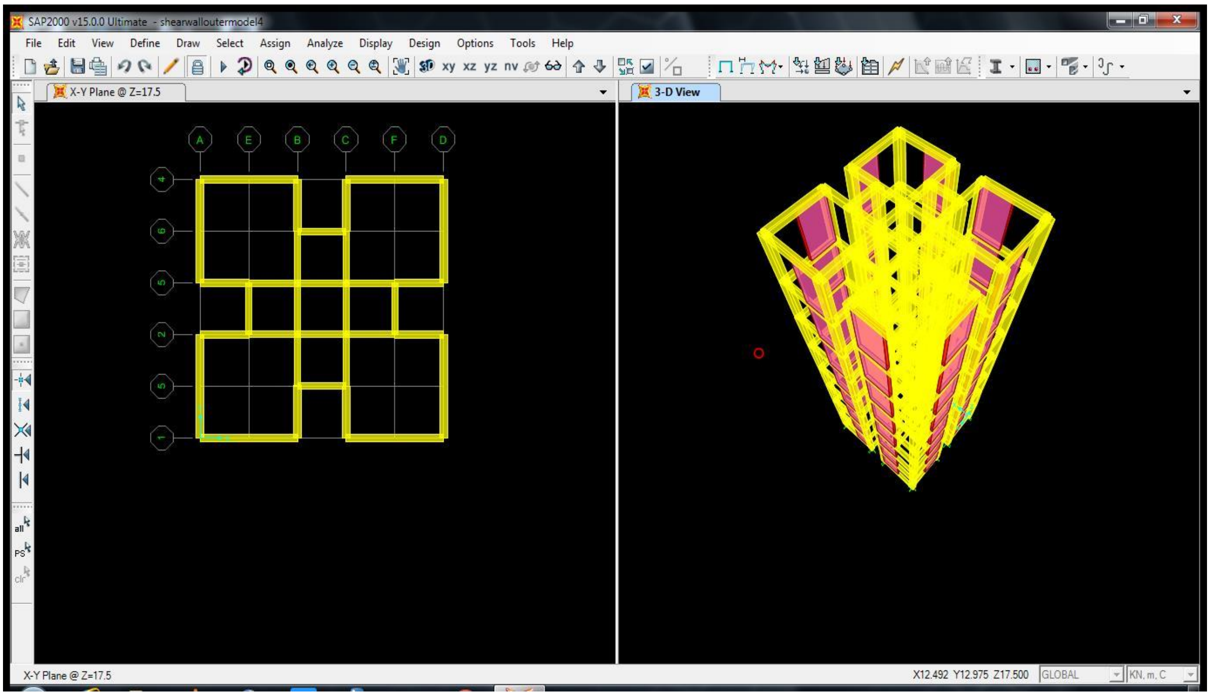


Fig. 4.4.2 Exterior Shear Wall M-4

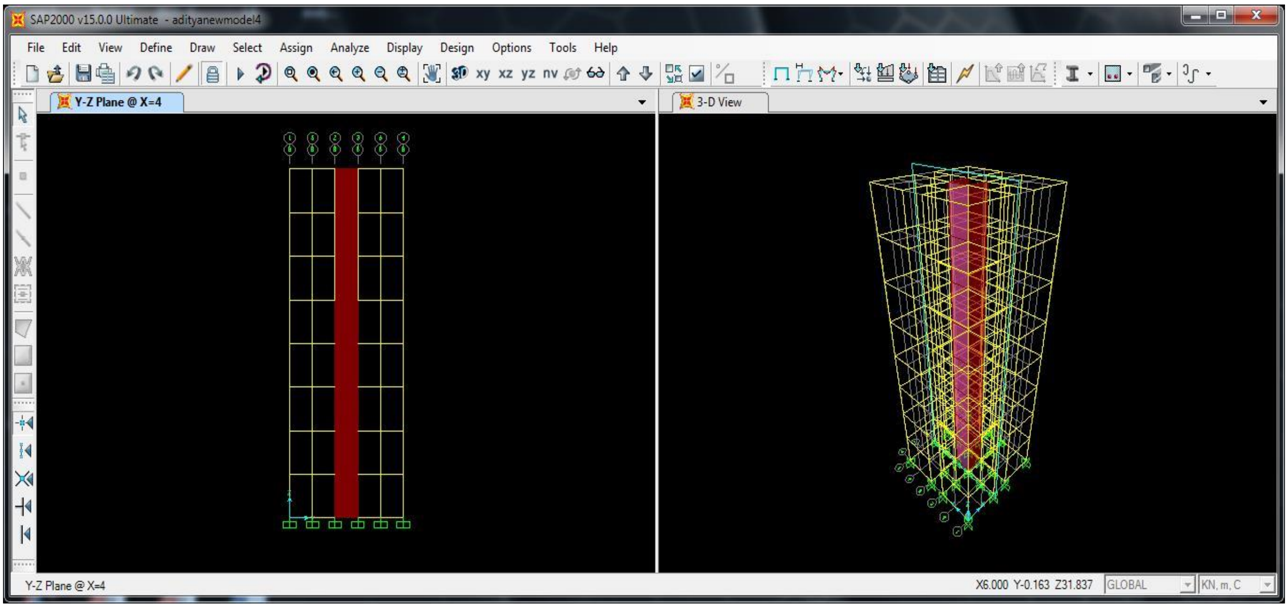


Fig.4.4.3 Interior Shear Wall M-4

	Without Shear Wall	Interior Shear Wall	Exterior Shear Wall
Max Axial Force	91.044KN/m	44.289KN/m	61.87KN/m
Max Shear Force	193.767KN/m	124.916KN/m	133.36KN/m
Max Moment	162.3788KN-m/m	88.1359KN-m/m	126.0837KN-m/m

Table 4.4 Max forces in members M-4

CHAPTER 5

DEISGN OF SHEAR WALL

5.1 FOLLOWING ARE THE DESIGN GUIDELINES FOLLOWED HERE (as per IS 13920:1993):-

5.1.1 The thickness of any part of the wall shall preferably, not be less than 150 mm.

5.1.2 The effective flange width, to be used in the design of flanged wall sections, shall be assumed to extend beyond the face of the web for a distance which shall be the smaller of (a) half the distance to an adjacent shear wall web, and (b) $1/10^{th}$ of the total wall height.

5.1.3 Shear walls shall be provided with reinforcement in the longitudinal and transverse directions in the plane of the wall. The minimum reinforcement ratio shall be 0.0025 of the gross area in each direction. This reinforcement shall be distributed uniformly across the cross section of the wall.

5.1.4 If the factored shear stress in the wall exceeds $0.25\sqrt{(f_{ck})}$ or if the wall thickness exceeds 200 mm, reinforcement shall be provided in two curtains, each having bars running in the longitudinal and transverse directions in the plane of the wall.

5.1.5 The diameter of the bars to be used in any part of the wall shall not exceed 1/10th of the thickness of that part.

5.1.6 The maximum spacing of reinforcement in either direction shall not exceed the smaller of $l_w/5$, $3xt_w$, and 450 mm; where l_w is the horizontal length of the wall, and t_w is the thickness of the wall web.

5.1.7 The nominal shear stress, τ_v, shall be calculated as:

$$\tau_v = \frac{V_u}{t_w\, d_w}$$

where:

V_u = factored shear force,
t_w = thickness of the web, and
d_w = effective depth of wall section. This may bytaken as 0.8 l_w for rectangular sections.

5.1.8 The design shear strength of concrete, τ_c, shall be calculated as per Table 13 of IS 456 :1978.

5.1.9 The nominal shear stress in the wall, τ_v, shall not exceed τ_c, max, as per Table 14 of IS 456 : 1978.

5.1.10 When τ_v is less than τ_c shear reinforcement shall be provided in accordance with **6.1.3**, **5.1.4** and **5.1.6**.

5.1.11 Whenτ_v is greater than τ_c, the area of horizontal shear reinforcement, A_h, to be provided within a vertical spacing, S_v, is given by

$$V_{us} = \frac{0.87\, f_y\, A_h\, d_w}{S_v}$$

Where V_{us} = ($V_u - \tau_c x t_w x d_w$), is the shear force to be resisted by the horizontal reinforcement. However, the amount of horizontal reinforcement provided shall not be less than the minimum, as per **5.1.3**.

5.1.12 The vertical reinforcement that is uniformly distributed in the wall shall not be less than the horizontal reinforcement calculated as per **5.1.10**.

5.1.13 Flexural Strength

5.1.13.1 The moment of resistance, M_{uv}, of the wall section may be calculated as for columns subjected to combined bending and axial load as per IS 456 : 1978. The moment of resistance of slender rectangular shear wall section with uniformly distributed vertical reinforcement is given in Annex A.

5.1.13.2 The cracked flexural strength of the wall section should be greater than its uncracked flexural strength.

5.1.13.3 In walls that do not have boundary elements, vertical reinforcement shall be concentrated at the ends of the wall. Each concentration shall consist of a minimum of 4 bars of 12 mm diameter arranged in at least 2 layers.

5.1.14 Boundary Elements

Boundary elements are portions along the wall edges that are strengthened by longitudinal and transverse reinforcement. Though they may have the same thickness as that of the wall web it is advantageous to provide them
with greater thickness.

5.1.14.1 Where the extreme fibre compressivestress in the wall due to factored gravity loadsplus factored earthquake force exceeds $0.2f_{ck}$,boundary elements shall be provided along thevertical boundaries of walls. The boundary elements may be discontinued where the calculated compressive stress becomes less than $0.15f_{ck}$. The compressive stress shall be calculated using a linearly elastic model and gross section properties.

5.1.14.2 A boundary element shall have adequate axial load carrying capacity, assuming short column action, so as to enable it to carry an axial compression equal to the sum of factored gravity load on it and the additional compressive load induced by the seismic force. The latter may be calculated as:

$$\frac{M_u - M_{uv}}{C_w}$$

where:

M_u = factored design moment on the entire wall section,

M_{uv} = moment of resistance provided by distributed vertical reinforcement across the wall section

c_w = center to center distance between the boundary elements along the two vertical edges of the wall.

5.1.14.3 If the gravity load adds to the strength of the wall, its load factor shall be taken as 0.8.

5.1.14.4 The percentage of vertical reinforcement in the boundary elements shall not be less than 0.8 percent, nor greater than 6 percent. In order to avoid congestion, the practical upper limit would be 4 percent.

5.1.14.5 Boundary elements, where required, as per **5.1.14.1**, shall be provided throughout their height, with special confining reinforcement.

5.1.14.6 Boundary elements need not be provided, if the entire wall section is provided with special confining reinforcement.

5.1.15.1 Coupled shear walls shall be connected by ductile coupling beams. If the earthquake induced shear stress in the coupling beam exceeds

$$\frac{0 \cdot 1\, l_s \sqrt{f_{ck}}}{D}$$

Where, l_s is the clear span of the coupling beam and D is its overall depth, the entire earthquake induced shear and flexure shall, preferably, be resisted by diagonal reinforcement.

5.1.15.2 The area of reinforcement to be provided along each diagonal in a diagonally reinforced coupling beam shall be:

$$A_{sd} = \frac{V_u}{1 \cdot 74\, f_y \sin \alpha}$$

Where V_u is the factored shear force, and α is the angle made by the diagonal reinforcement with the horizontal. At least 4 bars of 8 mm diameter shall be provided along each diagonal. The reinforcement along each diagonal shall be enclosed by special confining reinforcement, as per **5.4**. The pitch of spiral or spacing of ties shall not exceed 100 mm.

5.1.15.3 The diagonal or horizontal bars of a coupling beam shall be anchored in the adjacent walls with an anchorage length of 1.5 times the development length in tension.

5.1.16 Openings in Walls

5.1.16.1 The shear strength of a wall with openings should be checked along critical planes that pass through openings.

5.1.16.2 Reinforcement shall be provided along the edges of openings in walls. The area of the vertical and horizontal bars should be such as to equal that of the respective interrupted bars. The vertical bars should extend for the full storey height. The horizontal bars should be provided with development length in tension beyond the sides of the opening.

5.1.17 Discontinuous Walls

Columns supporting discontinuous walls shall be provided with special confining reinforcement.

5.1.18 Construction Joints

The vertical reinforcement ratio across a horizontal construction joint shall not be less than:

$$\frac{0{\cdot}92}{f_y}\left(\tau_v - \frac{P_u}{A_g}\right)$$

where τ_v is the factored shear stress at the joint, P_u is the factored axial force (positive for compression), and A_g is the gross cross sectional area of the joint.

5.1.19 Development, Splice and Anchorage Requirement

5.1.19.1 Horizontal reinforcement shall be anchored near the edges of the wall or in the confined core of the boundary elements.

5.1.19.2 Splicing of vertical flexural reinforcement should be avoided, as far as possible, in regions where yielding may take place. This zone of flexural yielding may be considered to extend for a distance of l_w above the base of the wall or one sixth of the wall height, whichever is more. However, this distance need not be greater than 2 l_w. Not more than one third of this vertical reinforcement shall be spliced at such a section. Splices in adjacent bars should be staggered by a minimum of 600 mm.

5.1.19.3 Lateral ties shall be provided around lapped spliced bars that are larger than 16 mm in diameter. The diameter of the tie shall not be less than one fourth that of the spliced bar nor less than 6 mm. The spacing of ties shall not exceed 150 mm center to center.

5.1.19.4 Welded splices and mechanical connections shall confirm to **25.2.5.2** of IS 456 : 1978. However, not more than half the reinforcement shall be spliced at a section, where flexural yielding may take place.

5.2 MOMENT OF RESISTANCE OF RECTANGULAR SHEAR WALL SECTION

5.2.1 The moment of resistance of a slender rectangular shear wall section with uniformly distributed vertical reinforcement may be estimated as follows:

(a) For $x_u/l_w < x_u^* /l_w$,

$$\frac{M_{uv}}{f_{ck}\, t_w\, l^2_w} = \phi \left[\left(1 + \frac{\lambda}{\phi} \right) \left(\frac{1}{2} - 0{\cdot}416\, \frac{x_u}{l_w} \right) - \left(\frac{x_u}{l_w} \right)^2 \left(0{\cdot}168 + \frac{\beta^2}{3} \right) \right]$$

where

$$\frac{x_u}{l_w} = \left(\frac{\phi + \lambda}{2\,\phi + 0{\cdot}36} \right); \quad \frac{x_u^*}{l_w} = \left(\frac{0{\cdot}003\ 5}{0{\cdot}003\ 5 + 0{\cdot}87\, f_y/E_s} \right);$$

$$\phi = \left(\frac{0{\cdot}87\, f_y\, \rho}{f_{ck}} \right); \quad \lambda = \left(\frac{P_u}{f_{ck}\, t_w\, l_w} \right);$$

ρ = vertical reinforcement ratio = $A_{st}/(\, t_w\, l_w\,)$,
A_{st} = area of uniformly distributed vertical reinforcement,
β = $0.87\, f_y/(0.003\,5\, E_s)$,
E_s = elastic modulus of steel, and
P_u = axial compression on wall.

Table 5.2.1 Moment for tension

5.2.2 For flexural compression failure

(b) For $x_u^* /l_w < x_u/l_w < 1.0$,

$$\frac{M_{uv}}{f_{ck}\, t_w\, l^2_w} = \alpha_1 \left(\frac{x_u}{l_w} \right) - \alpha_2 \left(\frac{x_u}{l_w} \right)^2 - \alpha_3 - \frac{\lambda}{2}$$

where

$$\alpha_1 = \left[0{\cdot}36 + \phi \left(1 - \frac{\beta}{2} - \frac{1}{2\,\beta} \right) \right]$$

$$\alpha_2 = \left[0{\cdot}15 + \frac{\phi}{2} \left(1 - \beta - \frac{\beta^2}{2} - \frac{1}{3\,\beta} \right) \right]; \text{ and } \alpha_3 = \frac{\phi}{6\,\beta} \left(\frac{1}{(\, x_u/l_w\,)} - 3 \right)$$

The value of x_u/l_w to be used in this equation, should be calculated from the quadratic equation

$$\alpha_1 \left(\frac{x_u}{l_w} \right)^2 + \alpha_4 \left(\frac{x_u}{l_w} \right) - \alpha_5 = 0,$$

where

$$\alpha_4 = \left(\frac{\phi}{\beta} - \lambda \right); \text{ and } \alpha_5 = \left(\frac{\phi}{2\,\beta} \right).$$

Table 5.2.2 Moment for compression

5.3 DESIGN OF EXTERIOR SHEAR WALL

5.3.1 Model 1

t= 200mm

b= 4400mm

τ_v = v/bd

= 166.15 x10^3/200x4400

= 0.1888 N/mm^2

τ_v = 0.25$\sqrt{f_{ck}}$

= 0.25$\sqrt{30}$

= 1.369 N/mm^2

Since τ_v<1.369 N/mm^2 therefore, no need for double reinforcement

τ_c = 0.37 N/mm^2 (IS 456-2000)

$\tau_{c\ max}$ = 3.5 N/mm^2 (IS 456-2000)

Since $\tau_v<\tau_c<\tau_{c\ max}$ therefore providing minimum steel

$A_{st\ min}$ = 0.0025x200x1000

= 500 mm^2

φ_{max} = (1/10)xt_w

= 20 mm

Providing φ=12 mm

no. of bars = $A_{st}/(\pi/4)\varphi^2$

= $500/(\pi/4)x12^2$

= 5 bars

Spacing= min of($l_w/5, 3xt_w$,450mm)

= min (4400/5,3x200,450)

= 450mm

Spacing=1000/5 (to be provided)

=200mm

Checking for moment

ρ = $A_{st}/t_w x l_w$

= 500/200x1000

= 0.0025

Φ = $(0.87 x f_y x \rho)/f_{ck}$

= 0.030

λ = $(P_u/f_{ck} x t_w x l_w)$

= $(81.83x10^3)/(30x200x4400)$

= 0.00324

x_u/l_w = $(\Phi + \lambda)/(2x\ \Phi + 0.36)$

= (0.030+0.00324)/(2*0.030+0.36)

= 0.07873

x_u^*/l_w = $(0.0035)/(0.0035+0.87(f_y/E_s))$

= 0.659

Therefore

$x_u/l_w < x_u^*/l_w$

$\beta \quad = 0.87xf_y/0.0035xE_s$

$= 0.87x415/0.0035x2x10^5$

$= 0.516$

$M_{uv}/(f_{ck}xt_wxl_w^2) = \Phi\ [(1+\lambda/\ \Phi)(0.5-0.416x(x_u/l_w))-(x_u/l_w)^2(0.168+\beta^2/3)]$

$M_{uv}/(30x200x1000^2) = 0.030[(1+0.00324/0.030)(0.5-0.416x0.07873)-$

$(0.07873)^2(0.168+0.516^2/3)]$

$= 92.4kN\text{-}m$

M_u (acting due to load) =7.101*4.4

= 31.244 kN-m

therefore wall is safe against flexure.

Therefore providing $A_{st,min}$ both vertically and horizontally

i.e. 5bars,φ=12mm@200mmc/c

5.3.2 Model 2

t= 200mm

b= 4400mm

τ_v = v/bd

= $163.84 \times 10^3/200 \times 4400$

= 0.1861 N/mm^2

τ_v = $0.25\sqrt{f_{ck}}$

= $0.25\sqrt{30}$

= 1.369 N/mm^2

Since $\tau_v < 1.369$ N/mm^2 therefore, no need for double reinforcement

τ_c = 0.37 N/mm^2 (IS 456-2000)

$\tau_{c\ max}$ = 3.5 N/mm^2 (IS 456-2000)

Since $\tau_v < \tau_c < \tau_{c\ max}$ therefore providing minimum steel

$A_{st\ min}$ = 0.0025x200x1000

= 500 mm^2

φ_{max} = $(1/10) x t_w$

= 20 mm

Providing φ=12 mm

no. of bars = $A_{st}/(\pi/4)\varphi^2$

= $500/(\pi/4) x 12^2$

= 5 bars

Spacing= min of(l_w/5,3xt_w,450mm)

= min (4400/5,3x200,450)

=450mm

Spacing=1000/5 (to be provided)

=200mm

Checking for moment

ρ = $A_{st}/t_w x l_w$

= 500/200x1000

= 0.0025

Φ = $(0.87 x f_y x \rho)/f_{ck}$

= 0.030

λ = $(P_u/f_{ck} x t_w x l_w)$

= $(92.52x10^3)/(30x200x4400)$

= 0.003504

x_u/l_w = $(\Phi + \lambda)/(2x\Phi + 0.36)$

= (0.030+0.003504)/(2x0.030+0.36)

= 0.07977

x_u^*/l_w = $(0.0035)/(0.0035+0.87(f_y/E_s))$

= 0.659

therefore

$$x_u/l_w < x_u^*/l_w$$

β $= 0.87*f_y/0.0035*E_s$

$= 0.87*415/0.0035*2*10^5$

$= 0.516$

$M_{uv}/(f_{ck} x t_w x l_w^2) = \Phi\ [(1+\lambda/\ \Phi)(0.5-0.416x(x_u/l_w))-(x_u/l_w)^2(0.168+\beta^2/3)]$

$M_{uv}/(30x200x1000^2) = 0.030[(1+0.003504/0.030)(0.5-0.416x0.07977)-$

$(0.07977)^2(0.168+0.516^2/3)]$

$= 93.444$ kN-m

M_u (acting due to load)=7.525x4.4

= 33.11 kN-m

therefore shear wall is safe against flexure.

Therefore providing $A_{st,min}$ both vertically and horizontally

i.e. 5bars,φ=12mm@200mmc/c

5.3.3 Model 3

t= 200mm

b= 4400mm

τ_v = v/bd

= $166.55\text{x}10^3/200\text{x}4400$

= 0.188 N/mm^2

τ_v = $0.25\sqrt{f_{ck}}$

= $0.25\sqrt{30}$

= 1.369 N/mm^2

Since τ_v<1.369 N/mm^2 therefore, no need for double reinforcement

τ_c = 0.37 N/mm^2 (IS 456-2000)

$\tau_{c\ max}$ = 3.5 N/mm^2 (IS 456-2000)

Since $\tau_v<\tau_c<\tau_{c\ max}$ therefore providing minimum steel

$A_{st\ min}$ = 0.0025x200x1000

= 500 mm^2

φ_{max} = (1/10)xt_w

= 20 mm

Providing φ=12 mm

no. of bars = $A_{st}/(\pi/4)\varphi^2$

= $500/(\pi/4)\text{x}12^2$

= 5 bars

Spacing= min of(l_w/5,3xt_w,450mm)

= min (4400/5,3x200,450)

=450mm

Spacing=1000/5 (to be provided)

=200mm

Checking for moment

ρ = $A_{st}/t_w x l_w$

= 500/200x1000

= 0.0025

Φ = $(0.87 x f_y x \rho)/f_{ck}$

= 0.030

λ = $(P_u/f_{ck} x t_w x l_w)$

= $(117.08x10^3)/(30x200x4400)$

= 0.00443

x_u/l_w = $(\Phi + \lambda)/(2x\ \Phi + 0.36)$

= (0.030+0.00443)/(2x0.030+0.36)

= 0.08197

x_u*/l_w = $(0.0035)/(0.0035+0.87(f_y/E_s))$

= 0.659

therefore

$$x_u/l_w < x_u^*/l_w$$

$\beta \quad = 0.87xf_y/0.0035xE_s$

$= 0.87x415/0.0035x2x10^5$

$= 0.516$

$M_{uv}/(f_{ck}xt_wxl_w^2) = \Phi\ [(1+\lambda/\ \Phi)(0.5-0.416x(x_u/l_w))-(x_u/l_w)^2(0.168+\beta^2/3)]$

$M_{uv}/(30x200x1000^2) = 0.030[(1+0.00443/0.030)(0.5-0.416x0.08197)-$

$(0.08197)^2(0.168+0.516^2/3)]$

$= 95.734$ kN-m

M_u (acting due to load)=8.0541*4.4

= 35.438 kN-m

therefore shear wall is safe against flexure.

Therefore providing $A_{st,min}$ both vertically and horizontally

i.e. 5bars,φ=12mm@200mmc/c

5.3.4 Model 4

t= 200mm

b= 4400mm

τ_v = v/bd

= $173.94 \times 10^3/200 \times 4400$

= 0.1976 N/mm^2

τ_v = $0.25\sqrt{f_{ck}}$

= $0.25\sqrt{30}$

= 1.369 N/mm^2

Since $\tau_v < 1.369$ N/mm^2 therefore, no need for double reinforcement

τ_c = 0.37 N/mm^2 (IS 456-2000)

$\tau_{c\ max}$ = 3.5 N/mm^2 (IS 456-2000)

Since $\tau_v < \tau_c < \tau_{c\ max}$ therefore providing minimum steel

$A_{st\ min}$ =0.0025x200x1000

= 500 mm^2

φ_{max} = $(1/10) \times t_w$

= 20 mm

Providing φ=12 mm

no. of bars = $A_{st}/(\pi/4)\varphi^2$

= $500/(\pi/4) \times 12^2$

= 5 bars

Spacing= min of($l_w/5, 3xt_w$,450mm)

= min (4400/5,3x200,450)

=450mm

Spacing=1000/5 (to be provided)

=200mm

Checking for moment

ρ = $A_{st}/t_w x l_w$

= 500/200x1000

= 0.0025

Φ = $(0.87 x f_y x \rho)/f_{ck}$

= 0.030

λ = $(P_u/f_{ck} x t_w x l_w)$

= $(136.51 x 10^3)/(30x200x4400)$

= 0.00517

x_u/l_w = $(\Phi + \lambda)/(2x\Phi + 0.36)$

= (0.030+0.00517)/(2x0.030+0.36)

= 0.0837

x_u^*/l_w =$(0.0035)/(0.0035+0.87(f_y/E_s))$

= 0.659

therefore

$x_u/l_w < x_u^*/l_w$

$\beta \quad = 0.87xf_y/0.0035xE_s$

$= 0.87x415/0.0035x2x10^5$

$= 0.516$

$M_{uv}/(f_{ck}xt_wxl_w^2) = \Phi\ [(1+\lambda/\ \Phi)(0.5-0.416xx_u/l_w)-(x_u/l_w)^2(0.168+\beta^2/3)]$

$M_{uv}/(30x200x1000^2) = 0.030[(1+0.00517/0.030)(0.5-0.416x0.0837)-$

$(0.0837)^2(0.168+0.516^2/3)]$

$= 97.62$ kN-m

M_u (acting due to loading)= 8.666x4.4

= 38.132kN-m

therefore shear wall is safe against flexure.

Therefore providing $A_{st,min}$ both vertically and horizontally

i.e. 5bars,φ=12mm@200mmc/c

5.4 DESIGN OF INTERIOR SHEAR WALL

5.4.1Model 1

t= 200mm

b= 4400mm

τ_v = v/bd

= $124.39 \times 10^3 / 200 \times 4400$

= 0.141 N/mm^2

τ_v = $0.25\sqrt{f_{ck}}$

= $0.25\sqrt{30}$

= 1.369 N/mm^2

Since $\tau_v < 1.369$ N/mm^2 therefore, no need for double reinforcement

τ_c = 0.37 N/mm^2 (IS 456-2000)

$\tau_{c\ max}$ = 3.5 N/mm^2 (IS 456-2000)

Since $\tau_v < \tau_c < \tau_{c\ max}$ therefore providing minimum steel

$A_{st\ min}$ = 0.0025x200x1000

= 500 mm^2

φ_{max} = (1/10)xt_w

= 20 mm

Providing φ=12 mm

no. of bars = $A_{st}/(\pi/4)\varphi^2$

= $500/(\pi/4)x12^2$

= 5 bars

Spacing= min of($l_w/5, 3xt_w$, 450mm)

= min (4400/5,3x200,450)

=450mm

Spacing=1000/5 (to be provided)

=200mm

Checking for moment

ρ = $A_{st}/t_w x l_w$

= 500/200x1000

= 0.0025

Φ = $(0.87 x f_y x \rho)/f_{ck}$

= 0.030

λ = $(P_u/f_{ck} x t_w x l_w)$

= $(53.75x10^3)/(30x200x4400)$

= 0.002035

x_u/l_w = $(\Phi + \lambda)/(2x\ \Phi + 0.36)$

= (0.030+0.002035)/(2x0.030+0.36)

= 0.07627

$x_u{*}/l_w$ = $(0.0035)/(0.0035+0.87(f_y/E_s))$

= 0.659

therefore

$x_u/l_w < x_u{*}/l_w$

β = $0.87xf_y/0.0035xE_s$

= $0.87x415/0.0035x2x10^5$

= 0.516

$M_{uv}/(f_{ck}xt_wxl_w^2) = \Phi\ [(1+\lambda/\ \Phi)(0.5-0.416x(x_u/l_w))-(x_u/l_w)^2(0.168+\beta^2/3)]$

$M_{uv}/(30x200x1000^2) = 0.030[(1+0.002035/0.030)(0.5-0.416x0.07627)-$

$(0.07627)^2(0.168+0.516^2/3)]$

= 89.731kN-m

M_u (acting due to loads)=8.544x4.4

= 37.594 kN-m

therefore shear wall is safe against flexure.

Therefore providing $A_{st,min}$ both vertically and horizontally

i.e. 5bars,φ=12mm@200mmc/c

5.4.2 Model 2

t= 200mm

b= 4400mm

τ_v = v/bd

= $127.03 \times 10^3 / 200 \times 4400$

= 0.144 N/mm^2

τ_v = $0.25\sqrt{f_{ck}}$

= $0.25\sqrt{30}$

= 1.369 N/mm^2

Since $\tau_v < 1.369\ N/mm^2$ therefore, no need for double reinforcement

τ_c = 0.37 N/mm^2 (IS 456-2000)

$\tau_{c\ max}$ = 3.5 N/mm^2 (IS 456-2000)

Since $\tau_v < \tau_c < \tau_{c\ max}$ therefore providing minimum steel

$A_{st\ min}$ = 0.0025x200x1000

= 500 mm^2

φ_{max} = (1/10)xt_w

= 20 mm

Providing φ=12 mm

no. of bars = $A_{st}/(\pi/4)\varphi^2$

= $500/(\pi/4) \times 12^2$

= 5 bars

Spacing= min of(l_w/5,3xt_w,450mm)

= min (4400/5,3x200,450)

=450mm

Spacing=1000/5 (to be provided)

=200mm

Checking for moment

$\rho = A_{st}/t_w x l_w$

= 500/200x1000

= 0.0025

$\Phi = (0.87 x f_y x \rho)/f_{ck}$

= 0.030

$\lambda = (P_u/f_{ck} x t_w x l_w)$

= $(59.26 x 10^3)/(30x200x4400)$

= 0.00224

$x_u/l_w = (\Phi + \lambda)/(2x\Phi + 0.36)$

= (0.030+0.00224)/(2x0.030+0.36)

= 0.07676

$x_u^*/l_w = (0.0035)/(0.0035+0.87(f_y/E_s))$

= 0.659

therefore

$$x_u/l_w < x_u^*/l_w$$

β $= 0.87 x f_y / 0.0035 x E_s$

$= 0.87x415/0.0035x2x10^5$

$= 0.516$

$M_{uv}/(f_{ck} x t_w x l_w^2) = \Phi\ [(1+\lambda/\ \Phi)(0.5-0.416 x x_u/l_w)-(x_u/l_w)^2(0.168+\beta^2/3)]$

$M_{uv}/(30x200x1000^2) = 0.030[(1+0.00224/0.030)(0.5-0.416x0.07676)-$

$(0.07676)^2(0.168+0.516^2/3)]$

$= 90.268$kN-m

M_u (acting due to loads)= 10.712x4.4

= 47.1322 kN-m

therefore shear wall is safe against flexure.

Therefore providing $A_{st,min}$ both vertically and horizontally

i.e. 5bars,φ=12mm@200mmc/c

5.4.3 Model 3

t= 200mm

b= 4400mm

τ_v = v/bd

= $126.01 \times 10^3 / 200 \times 4400$

= $0.143\ N/mm^2$

τ_v = $0.25\sqrt{f_{ck}}$

= $0.25\sqrt{30}$

= $1.369\ N/mm^2$

Since $\tau_v < 1.369\ N/mm^2$ therefore, no need for double reinforcement

τ_c = $0.37\ N/mm^2$ (IS 456-2000)

$\tau_{c\ max}$ = $3.5\ N/mm^2$ (IS 456-2000)

Since $\tau_v < \tau_c < \tau_{c\ max}$ therefore providing minimum steel

$A_{st\ min}$ = $0.0025 \times 200 \times 1000$

= $500\ mm^2$

φ_{max} = $(1/10) \times t_w$

= 20 mm

Providing φ=12 mm

no. of bars = $A_{st} / (\pi/4)\varphi^2$

= $500 / (\pi/4) \times 12^2$

= 5 bars

Spacing= min of($l_w/5, 3xt_w$, 450mm)

= min (4400/5,3x200,450)

=450mm

Spacing=1000/5 (to be provided)

=200mm

Checking for moment

ρ = $A_{st}/t_w x l_w$

= 500/200x1000

= 0.0025

Φ = $(0.87 x f_y x \rho)/f_{ck}$

= 0.030

λ = $(P_u/f_{ck} x t_w x l_w)$

= $(66.33x10^3)/(30x200x4400)$

= 0.002512

x_u/l_w = $(\Phi + \lambda)/(2x\ \Phi + 0.36)$

= (0.030+0.002512)/(2x0.030+0.36)

= 0.0774

x_u*/l_w = $(0.0035)/(0.0035+0.87(f_y/E_s))$

= 0.659

therefore

$x_u/l_w < x_u^*/l_w$

$\beta = 0.87xf_y/0.0035xE_s$

$= 0.87x415/0.0035x2x10^5$

$= 0.516$

$M_{uv}/(f_{ck}xt_wxl_w^2) = \Phi\ [(1+\lambda/\ \Phi)(0.5-0.416x(x_u/l_w))-(x_u/l_w)^2(0.168+\beta^2/3)]$

$M_{uv}/(30x200x1000^2) = 0.030[(1+0.002512/0.030)(0.5-0.416x0.0774)-$

$(0.0774)^2(0.168+0.516^2/3)]$

$= 90.979$ kN-m

M_u (acting due to loads)= 12.804x4.4

= 56.338 kN-m

therefore shear wall is safe against flexure.

Therefore providing $A_{st,min}$ both vertically and horizontally

i.e. 5bars,φ=12mm@200mmc/c

5.4.4 Model 4

t= 200mm

b= 4400mm

τ_v = v/bd

= $128.08 \times 10^3 / 200 \times 4400$

= 0.1455 N/mm^2

τ_v = $0.25\sqrt{f_{ck}}$

= $0.25\sqrt{30}$

= 1.369 N/mm^2

Since $\tau_v < 1.369\ N/mm^2$ therefore, no need for double reinforcement

τ_c = 0.37 N/mm^2 (IS 456-2000)

$\tau_{c\ max}$ = 3.5 N/mm^2 (IS 456-2000)

Since $\tau_v < \tau_c < \tau_{c\ max}$ therefore providing minimum steel

$A_{st\ min}$ = 0.0025x200x1000

= 500 mm^2

φ_{max} = $(1/10) x t_w$

= 20 mm

Providing φ=12 mm

no. of bars = $A_{st}/(\pi/4)\varphi^2$

= $500/(\pi/4) x 12^2$

= 5 bars

Spacing= min of(l_w/5,3xt_w,450mm)

= min (4400/5,3x200,450)

=450mm

Spacing=1000/5 (to be provided)

=200mm

Checking for moment

ρ = $A_{st}/t_w x l_w$

= 500/200x1000

= 0.0025

Φ = $(0.87 x f_y x \rho)/f_{ck}$

= 0.030

λ = $(P_u/f_{ck} x t_w x l_w)$

= $(73.2x10^3)/(30x200x4400)$

= 0.002772

x_u/l_w = $(\Phi + \lambda)/(2x\ \Phi +0.36)$

= (0.030+0.002772)/(2*0.030+0.36)

= 0.07977

x_u^*/l_w = $(0.0035)/(0.0035+0.87(f_y/E_s))$

= 0.659

therefore

$x_u/l_w < x_u^*/l_w$

β $= 0.87xf_y/0.0035xE_s$

$= 0.87x415/0.0035x2x10^5$

$= 0.516$

$M_{uv}/(f_{ck}xt_wxl_w^2) = \Phi\ [(1+\lambda/\ \Phi)(0.5\text{-}0.416xx_u/l_w)\text{-}(x_u/l_w)^2(0.168+\beta^2/3)]$

$M_{uv}/(30x200x1000^2) = 0.030[(1+0.002772/0.030)(0.5\text{-}0.416x0.07977)\text{-}$

$(0.07977)^2(0.168+0.516^2/3)]$

$= 91.488$ kN-m

M_u (acting due to loads)= 12.162x4.4

= 53.513 kN-m

therefore shear wall is safe against flexure.

Therefore providing $A_{st,min}$ both vertically and horizontally

i.e. 5bars,φ=12mm@200mmc/c

CHAPTER 6

CONCLUSION

From the work done in previous chapters, following conclusions have been made

1. The shear wall is effective in reducing both lateral forces and moments.

2. The shear wall also limits displacements

3. The Interior shear wall(29%) reduces forces in the member to a greater extent than the Exterior shear wall(30.5%).

4. Interior shear wall reduces axial force(68%) developed and lateral force(25%) to a larger extent than Exterior shear wall.

5. Exterior shear wall has benefit that it develops lesser moment(41%) than Interior shear wall.

6.

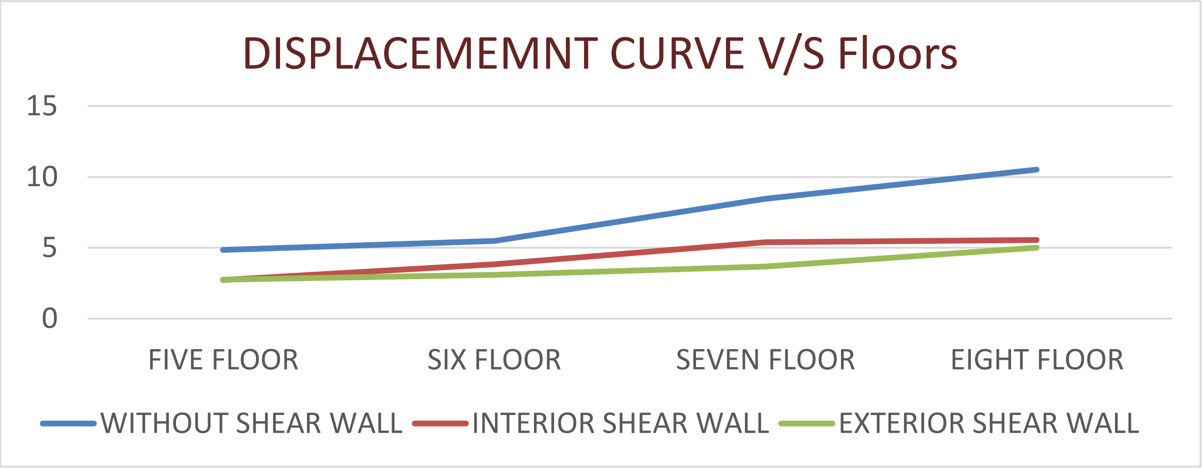


Fig. 6.1 Displacement curve VS Floors

7. From the floor displacement curve of structurtes , it is clear that shear wall reduces the displacements to a greater extent.

8. RCC buildings with Exterior shear wall has less displacements compared to the RCC buildings with Interior shear wall.

		Structural Performance	Practicability	Post Occupancy Evaluation	Environmental Effect	Cost
ESW	Advantages	Total displacement of the system lessens pretty much; the horizontal load on the existing building decreases. Stiffness and strength increase. It has been observed in the experiments that no anchorage problem arises.	Since the shear wall will be made exterior of the building. The manufacturing can be easily carried out by means of a cage established outside.	There is no need for evacuating the building. There is not any alteration or reparation in the building, the inner use of the building does not change.	———	There is no additional inner alteration cost besides the strengthening cost.
ESW	Disadvantages	There is a need for an additional exterior ground, since the new shear wall will be located at the exterior front of and vertically to the building. Since normal strength level is too low and the moment is too high on this ground, rotation of the ground may occur.	It cannot be applied to all buildings. It is necessary that the building is on separate statute. The shear wall is located between the existing column-beam system; the connection is established only through columns.	Since the shear walls are established exterior of the building, the illumination is influenced. Use of balcony and commercial story in the building becomes difficult.	The exterior front of the building changes. The pedestrian traffic on pavement is influenced; it is troublesome in respect of reconstruction.	———
PISW	Advantages	Total displacement of the system lessens pretty much; the horizontal load on the existing building decreases. Stiffness and strength increase. Rotation at the ground is partially prevented, since the additional or strengthened ground system is located between the column axes of the existing building.	This method can be applied to all reinforced concrete buildings. Since the shear wall is between the existing column-beam system, full connection can be established. If the existing ground system is continuous, by means of a simple strengthening operation on the ground there may be no need for changing the ground system.	———	It does not affect the exterior view of the building.	———
PISW	Disadvantages	Continuity of the shear wall needs to be secured so as to make the shear wall fully functional. The beam at the axe need also be drilled for the joints of the shear wall, which will be built evenly to the axe at top and bottom stories. Also, the column that the shear wall joins needs to be drilled longitudinally for monolithic operation.	Producing of shear wall in the building is difficult. The brick walls that are not bearing and that contributes to the stiffness of the building may need to be removed even if partially.	The building should be completely or partially evacuated during strengthening. Aftermaths of strengthening mostly complicate the use of inner axes. The satisfaction of user falls off.	———	Besides the cost of shear wall, the function of the building may partially change due to the collapse of the walls that cannot be bearing on the inner front, and inner alteration may be required.

Table 6.1 Comparison between exterior and interior shear wall

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