EVALUATION OF VARIOUS RETROFITTING TECHNIQUES FOR RC FRAMED BUILDING

A Dissertation Report submitted in Partial Fulfillment of the Requirement for the Award of the Degree of

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IN

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DECLARATION

I hereby declare that this is a bonafide work which is presented in this dissertation report entitled **"Evaluation of Various Retrofitting Techniques for RC Framed Building"** as per the requirements for the award of **Master of Engineering** in **Structural Engineering**, submitted in the Civil Engineering Department, Thapar Institute of Engineering and Technology (TIET), Patiala. This work is carried out under the guidance and supervision of **Dr. Shruti Sharma**, Professor and **Dr. Naveen Kwatra**, Professor, Department of Civil Engineering, Thapar Institute of Engineering and Technology (TIET), Patiala. It is declared that this work is original and has not been submitted anywhere else for award of any other degree or certificate.

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This is to certify that the above declaration made by the student concerned is correct according to the best of my knowledge and belief.

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ABSTRACT

In recent times due to the frequent earthquakes in the Indian subcontinent region, structures need to be designed seismic resistant as per the latest IS codal provisions. The IS codal guidelines for seismic resistant design of buildings have undergone significant change in the Sixth revision of IS1893 (Part1) "Criteria for Earthquake Resistant Design of Structures, Part 1: General Provisions and Buildings" in 2016. Thus, the old buildings which were constructed prior to year 2016 designed as per the fifth revision are more prone to seismic failures. In the present study an existing building has been analysed and checked as per the revised codal guidelines and has been retrofitted by various techniques.

The building taken in this study is an existing residential S+6 building built in the district of Rohtak in Haryana. The existing building was constructed in the year 2013, hence the provisions followed in design were in accordance with IS1893:2002. In this study, this existing structure has been modelled in STAAD-Pro and a dynamic analysis is carried out following the latest revision of the code in 2016. It was observed after analysis that the model did not comply with some of the clauses of the latest code. There were also some structural members failures observed which required retrofitting of members for safe functioning of the building. Different retrofitting techniques such as Jacketing of distressed members, addition of Shear Walls and addition of Steel Bracings were studied. Jacketing was done by increasing the size of columns and beams experiencing failure. Shear walls were added in both x and z-directions in the building and analysed for various parameters. Similarly, a model with steel bracings in both the dimensions of building i.e. x and z direction of the building between columns with convenient and preferable location was analysed. All these models were analysed and studied for different parameters like Base Shear, Time Period, Lateral Displacement, Storey Drift, Bending Moment, Shear

Force etc. It was seen that by applying these retrofitting techniques, the dynamic behaviour of the structure improved resulting in safe design as compared to existing building. A complete analysis as to which retrofitting procedure is preferable and at which location, also is performed in this research.

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1. INTRODUCTION

1.1 GENERAL

1.1.1 Framed Structure

A RC framed building consists of framework of horizontal and vertical members joined together by rigid connections. The entire structure is a monolithic construction, ensuring rigidity to the connections. The building frame is subjected to gravity loads e.g. dead load, live load and lateral loads due to seismic and wind action. The lateral loads are primarily resisted by the vertical members of the frame like columns, shear walls etc. These lateral loads resisting elements are the most critical members in the frame. The complete framework of beam and slab is integrated with these vertical members of the frame and thus make it behave as a single unit. (Fig. 1.1)

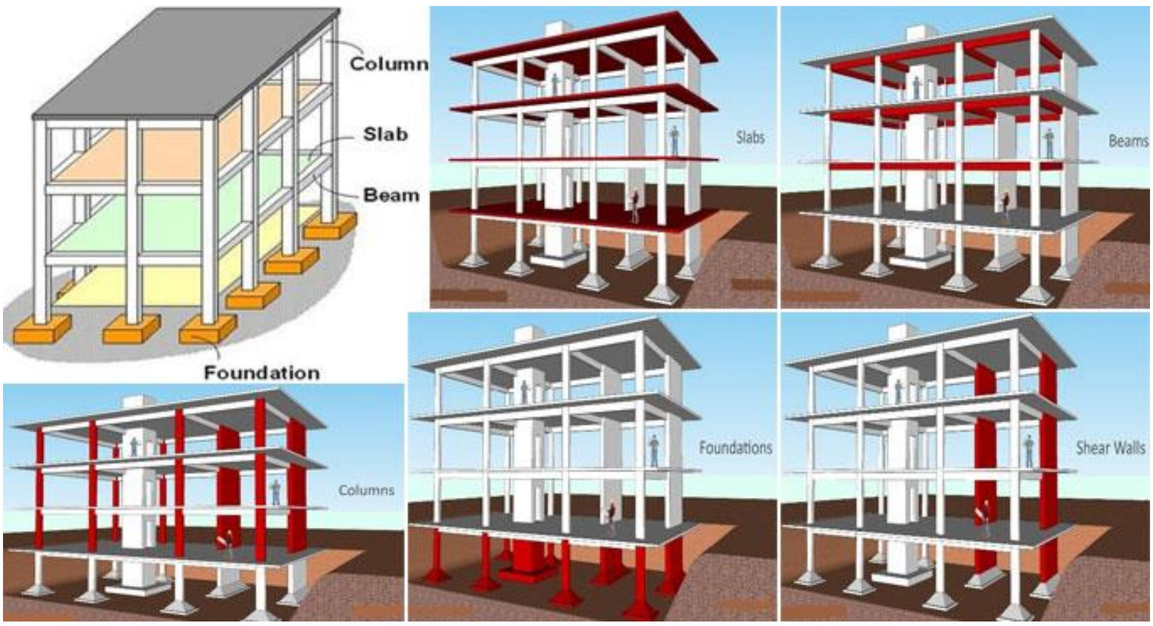


Fig. 1.1 Typical arrangement of members in RC framed building (Ref: www.civildigital.com)

When the number of storeys in a building increases the size requirement for the column increases and heavy reinforcement is required in them. As a result, there is a lot of congestion of steel bars at the joints, making it difficult to place concrete in a homogenous manner. So, in tall buildings, shear walls are used to resist the lateral loads along with the columns. The shear walls form an effective lateral force resisting system in the building when they are placed in the right locations. They prove to be efficient and cost effective.

1.1.2 Seismic Analysis

Any structure that is hit by a seismic wave, its base or foundation will respond to the ground motion. Seismic analysis is performed to evaluate the behaviour of a structure under the action of seismic excitation at its base.
Seismic ground vibrations' characteristics (intensity, duration, frequency) depend on earthquake magnitude, focal depth, epicentral distance, path characteristics, and soil strata beneath structures. Ground motion is analyzed in three mutually perpendicular directions. Understanding these factors aids in designing earthquake-resistant structures and mitigating seismic hazards. The predominant direction of the ground vibration is usually horizontal. The effect of vertical component of ground motion is significant for structural with large spans or for structures in which the overall stability is a criteria in for design.

IS: 1893-2016, the latest Seismic Indian Code, provides amendments pertaining to the design of earthquake-resistant buildings. These amendments encompass a range of guidelines, with a significant focus on dynamic seismic analysis. According to the code, dynamic seismic analysis is required for all buildings, except for regular structures below 15 m in height located in seismic zone II.

Dynamic seismic analysis was typically used for buildings taller than 40 m, while static analysis was suitable for shorter ones. Designing earthquake-resistant buildings involves considering seismic parameters like structure type, construction materials, and foundation soil type. These factors ensure appropriate measures are taken to withstand seismic forces. With advancements in technology and increased awareness, dynamic analysis may now be considered for shorter structures to enhance overall safety and resilience against earthquakes.

Following methods are used for determining design seismic forces and their effect on the behaviour of the structure:

- Equivalent static method
- Dynamic Analysis Method
    i. Response spectrum method
    ii. Time history method

- **Equivalent static method**

Equivalent static method is generally used for analysis of regular structure with natural time period less than 0.4 sec. In this method firstly the design base shear is computed for the whole building, then this base shear is distributed to the various floor levels at the corresponding centers of mass and finally this design seismic force at each floor level is distributed to the columns and shear walls through structural analysis considering the in plane rigidity of the floor slab with floor acting as the diaphragm. The design storey shear for a storey level is calculated by summing the design lateral forces at all floors above that storey. It is applicable for regular buildings with height less than 15m in seismic zone II.

The design base shear is obtained as under: $\mathbf{V_b = A_h * W}$

Where, V_b = Design Base Shear

W = Seismic Weight

Ah is design horizontal acceleration coefficient equal to $\frac{(\frac{Z}{2})*(\frac{S_a}{g})}{(\frac{R}{I})}$,

Where, Z = seismic zone factor

I = Importance factor

R = Response reduction factor

Sa/g = Design acceleration coefficient

- **Dynamic analysis method**

Linear dynamic analysis is required to be performed to obtain the design lateral seismic force and its distribution to different levels along the height of the building for all buildings in Seismic Zone III, IV and V and for buildings higher than 15 m in Seismic

Zone II. It can be performed by either response spectrum method or time history method. Dynamic analysis demonstrates the structure's actual behaviour.

- **Response Spectrum Method**

The response spectrum is a linear-dynamic analysis used to study the behavior of a building or structure subjected to lateral or seismic forces. It involves a system-based analysis with one degree of freedom. The response spectrum curve of a building structure subjected to lateral or earthquake forces is plotted with time period on the horizontal axis and acceleration, velocity, or displacement on the vertical axis to determine the peak response of the structure to past earthquake forces (Fig.1.2).

This approach considers the dynamic response of the structure by analyzing its response to various modes of vibration separately and then combining them to examine their combined effects. By applying ground acceleration with a certain amount of damping in the structure, the peak responses such as displacement, acceleration, etc., of an elastic structure with a single degree of freedom can be plotted. The response spectrum method is valuable in assessing the potential seismic performance of structures and guiding engineers in designing earthquake-resistant buildings.

In the analysis, an adequate number of mode shapes should be chosen to ensure the combined modal masses represent at least 90% of the total modal mass. This ensures a comprehensive understanding of the building's dynamic behavior. Additionally, provisions must be made to account for increased shear forces caused by eccentricity between the center of mass and center of resistance.

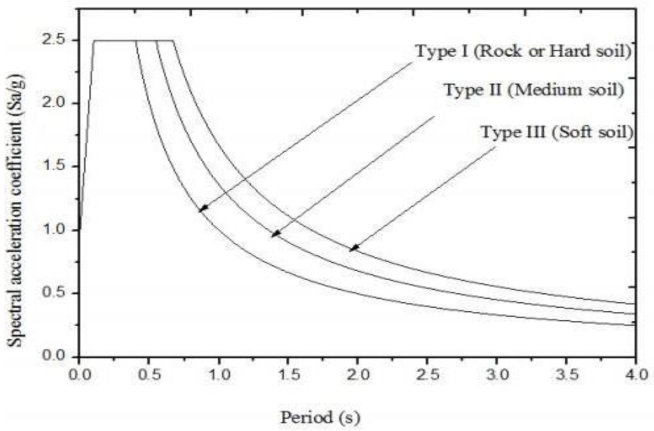


Fig. 1.2 Response spectra for 5% damping (Ref.: IS 1893-2016)

- **Time history analysis**

In time history analysis the mass, stiffness and damping matrices of the elements are used to solve for the forced vibration on the structure due to the seismic load which varies with time. The dynamic equilibrium equation is a system of linear differential equation of second order and its solution is obtained by using modal or direct integration method. For Time History Analysis either a linear or a non-linear approach can be used. Time history analysis provides the actual linear/non-linear response or behaviour of the structure under historical earthquake data, and it produces superior findings than other forms of analysis. The PEER ground motion database website provides access to historical earthquake data worldwide. This method allows to analyze seismic effects more accurately, ensuring safer and more resilient structures against earthquake forces.

- **Pushover analysis**

Pushover analysis has proven to be a useful tool for identifying the inelastic strength and displacement capacity of a structure. Under constant gravity loads and escalating lateral loads, it is a static non-linear analysis. The SAP2000 program is used to obtain the finite element modal of a frame when it experiences lateral and gravitational stresses. Incrementally applying lateral loads allows studying various structural failures like cracks, plastic hinge formation, and yielding. These occurrences are documented and their performance is assessed, providing a reliable method to pinpoint weak areas within the structure. This approach helps to identify vulnerabilities and improve the overall seismic performance and safety of the building design.

Progressive plastic hinge formation redistributes internal forces, but in some hinge locations, local collapse may occur due to plastic deformation exceeding its limit. The structure is pushed to the point of total collapse, the development of a collapse mechanism by a suitable number of plastic hinges, or the system's inability to endure the load pattern to maintain force equilibrium.

According to the performance of the structure for which it is intended, the hinge properties of the structure are used to calculate the performance point (from the capacity spectrum curve) and performance level of the structure. Although there is minimal damage, the hinges are produced more in the beam than in the column. Additionally, a structure with a lot of plastic hinges in the columns is not desired because it could result in structural damage.

1.1.3 Comparison of IS 1893:2002 and IS 1893:2016

The seismic codes are developed, considering the seismology of the country, accepted seismic risk levels, properties of construction materials, construction methods, and various structure typologies. In India, the IS 1893 (Part1) code is the designated standard for earthquake-resistant design and analysis of structures. Over the last decade, the Earthquake Engineering Sectional Committee of the Bureau of Indian Standards has conducted detailed and advanced research, along with damage surveys.

Through these efforts, a vast amount of data on the behavior of different types of structures during earthquakes has been collected, contributing to a better understanding of seismic performance. This continuous research and data collection led to the revision of the IS 1893 (Part 1): 2002 standard, resulting in the sixth revision, which was published in 2016. The updated code incorporates the latest findings and knowledge to further enhance the seismic resilience of buildings and structures in India. These revisions ensure that the code remains relevant and effective in guiding engineers in designing earthquake-resistant structures and safeguarding the built environment against seismic hazards.

The revisions in major clauses has been presented below:

- As per the clause 1.2 & 1.3, the parking structures, security cabins, ancillary structures, scaffolding, temporary excavations need to be designed for seismic forces
- Clause 6.1.3 expects to design the structures for at least the minimum design lateral force specified in Table 7 of standard, which is newly added in latest version of code.
- The clause 6.3.1.1 from latest code expects to adopt provisions for earthquake resistant design, ductile detailing & construction related to seismic conditions as per the standard even when load combinations that do not contain seismic effects but indicate larger demand than combinations including the seismic effects.
- As per the clause 6.3.3.1, the structures located in seismic zone IV or V, structures which has plan or vertical irregularity, structures founded on soft soils, bridges, structures with long spans or with large lateral overhangs of structural members are required to consider the effects due to vertical earthquake shaking in load

combinations. The load combinations for three directional earthquake ground shaking are mentioned in clause 6.3.4.

- In IS 1893 (Part1): 2016, separate design spectra are defined for natural period up to 6 seconds for equivalent static method and response spectrum method. Fig. 2 in the standard shows these graphs of design acceleration coefficient corresponding to 5% damping. Hence, clause 6.4.2 mentions the expressions for determination of design acceleration coefficient (Sa /g) for use in equivalent static method as well as use in response spectrum method. The table 4 in new standard deals with the classification of type of soil on which structure can be founded. It is used to be in the determination of correct spectrum, to calculate the Sa /g.
- When seismic forces are considered, net bearing pressure in soils can be increased, depending upon type of foundation & type of soil. To determine the type of soil for this purpose, soils are divided into four types which are mentioned in Table 2 of the new standard.
- As per the clause 6.4.3.1, for structural analysis, to take into consideration the cracked section, the moment of inertia shall be taken as 70% of gross moment of inertia of columns & 35% of gross moment of inertia of beams in case for RC & masonry structures. The gross moment of inertia can be considered for columns & beams in case of steel structures.
- In IS 1893 (Part1): 2016, Table 8 enlists the values of Importance factor depending upon the use, occupancy & service provided by the structures. The important factor value "1.2" is introduced for residential or commercial buildings with occupancy of more than 200 people.
- The code expect to ensure that the first 3 modes together contribute at least 65% mass participation factor in each principal plan direction & the fundamental natural periods of the building in the two principal plan directions are away from each other by at least 10% of the larger value, to avoid the irregular modes of oscillation in two principal plan directions.
- The Table 9 in code deals with Response Reduction factor R for various lateral load resisting systems. Five types of lateral load resisting system & their respective R values are mentioned in the table which are, Moment Frame systems, Braced Frame Systems, Structural Wall systems, Dual systems, and Flat slab – structural wall

systems. According to the code, followings are the revised & newly added types of load resisting systems & their respective R values.

- Steel Buildings with OMRF – 3.0
- Steel Buildings with SMRF – 5.0
- Buildings with ordinary braced frame having concentric braces – 4.0
- Buildings with special braced frame having concentric braces – 4.5
- Buildings with special braced frame having eccentric braces – 5.0
- Unreinforced masonry with horizontal RC seismic bands – 2.0
- Unreinforced masonry with horizontal RC seismic bands & vertical reinforcing bars at corners of rooms & jambs of opening (with reinforcement as per IS 4326) – 2.5
- Confined masonry – 3.0
- Buildings with ductile RC structural walls with RC OMRFs – 4.0
- Flat Slab- Structural Wall - 3.0

- The clause 7.3.5 & 7.3.6 states that, in regions of severe snow loads & sand storms exceeding intensity of 1.5 kN /m2, 20% of uniform design snow load or sand load shall be included in the estimation of seismic weight. In buildings with interior partitions, the weight of these partitions on floors shall be included in the estimation of seismic weight & this value shall not be less than 0.5 kN /m^2. In case the minimum values of seismic weights corresponding to snow loads or sand storms or partitions given in IS 875 are higher, the higher values shall be used.
- The clause 7.6.2 gives newly added equations for calculation of approximate fundamental natural period,
  - For Bare steel MRF building, Ta = 0.085 h $^{0.75}$
  - For Building with RC Structural Walls,

$$T_a = \frac{0.075h^{0.75}}{\sqrt{A_w}}$$

Where h is the height of building as defined in clause 7.6.2, in meters, d is base dimension of building at plinth along considered direction of seismic, in meters. Aw is total effective area in m^2 of walls in first storey of building which is given by,

$$A_w = \sum_{i=1}^{N_w} \left[A_{wi} \left\{ 0.2 + \left(\frac{L_{wi}}{h} \right)^2 \right\} \right]$$

Where, Awi is effective cross sectional area of wall i in first storey of building in m^2, Lwi is length of structural wall i in the first story in the considered direction of seismic force in meters, Nw is number of walls in the considered direction of seismic force. The value of Lw / h to be used in the equation shall not exceed 0.9.

- In the IS 1893: 2016, Fig. 5 explains the definition of height & base width of buildings, which is newly introduced.
- As per the clause 7.6.4, a floor diaphragm shall be flexible, if it deforms such that the maximum lateral displacement measured from the chord of the deformed shape at any point of diaphragm is more than 1.2 times average displacement of the entire diaphragm.
- Clause 7.7.1 expects to perform linear dynamic analysis to obtain design seismic base shear & its distribution at different levels along height of building, for all buildings other than regular buildings lower than 15 m in seismic zone II.
- The newly added recommendations regarding RC frame buildings with unreinforced masonry infill walls are given as clause 7.9. These provisions are made to estimate the in-plane stiffness & strength of URM infill walls in the structures. Also the design equations are provided along with the clauses.
- The clause 7.10.3 states that RC structural walls must be designed so as the lateral stiffness in open storey is more than 80% of that in the storey above & lateral strength in open storey is more than 90% of that in the storey above & RC structural wall must not increase torsional irregularity in plan than that already present in the building.
- As per clause 7.12.3, the compound walls shall be designed for design horizontal coefficient Ah of 1.25Z, that is, with I =1, R =1, & Sa /g = 2.5. The Annex F in IS 1893:2016 deals with simplified procedure for evaluation of liquefaction potential which is newly added.

1.1.4 Retrofitting Techniques

The seismic protection of buildings is aimed at enhancing the performance of structures during earthquakes. India has witnessed earthquakes of varying magnitudes in recent times, resulting in significant damage to life and property. Therefore, advancements in materials and techniques have been developed to effectively repair, strengthen, and retrofit existing buildings, whether they are damaged or undamaged. Some of the techniques are shown in Fig.1.3 The primary objective of structural engineers is to restore structures promptly and efficiently. However, selecting the appropriate materials, techniques, and procedures for repairing a specific structure has posed significant challenges. Innovative repair techniques offer several advantages over conventional methods. Key factors for successful repair, strengthening, and restoration of damaged structures include the use of standard and innovative repair materials, employing appropriate technology, ensuring skilled workmanship, and implementing quality control measures during the repair process.

Fig.1.3 Different Retrofitting Techniques (Ref: www.tutorialtipscivil.com)

Retrofitting is the process of modifying existing structures, including residential buildings, bridges, and historical buildings, to enhance their resistance against seismic activities such as earthquakes, volcanic eruptions, and other natural disasters. In the case of RCC (Reinforced Concrete) structural members, retrofitting aims to restore the strength of deteriorated concrete elements and prevent further damage. The concrete element may have lost its strength due to design errors, poor workmanship, or the aggressive actions of harmful agents. Retrofitting addresses these deficiencies and helps preserve the integrity of the structure. Few types of Retrofitting techniques are discussed below:

1.1.4.1 Jacketing

Jacketing is a structural retrofitting and strengthening method involving encasing an existing member with new materials. It increases or restores the size of the section using concrete, carbon/glass fibers, or plates. The process enhances the structural integrity and load-carrying capacity of the element, making it more resilient against external forces and potential damage.

Jacket, a thin additional layer of material, increases the load-bearing capacity of structural members. It can be used in repairing foundations, columns, beams, and walls, enhancing their strength and resilience against damaging forces (Fig.1.4 & Fig.1.5). The process involves wrapping additional reinforcement around the section and placing shotcrete or in situ concrete on it. This combination of reinforcement and concrete enhances the strength of the section and protects it from further deterioration, making it a durable and effective structural strengthening technique.

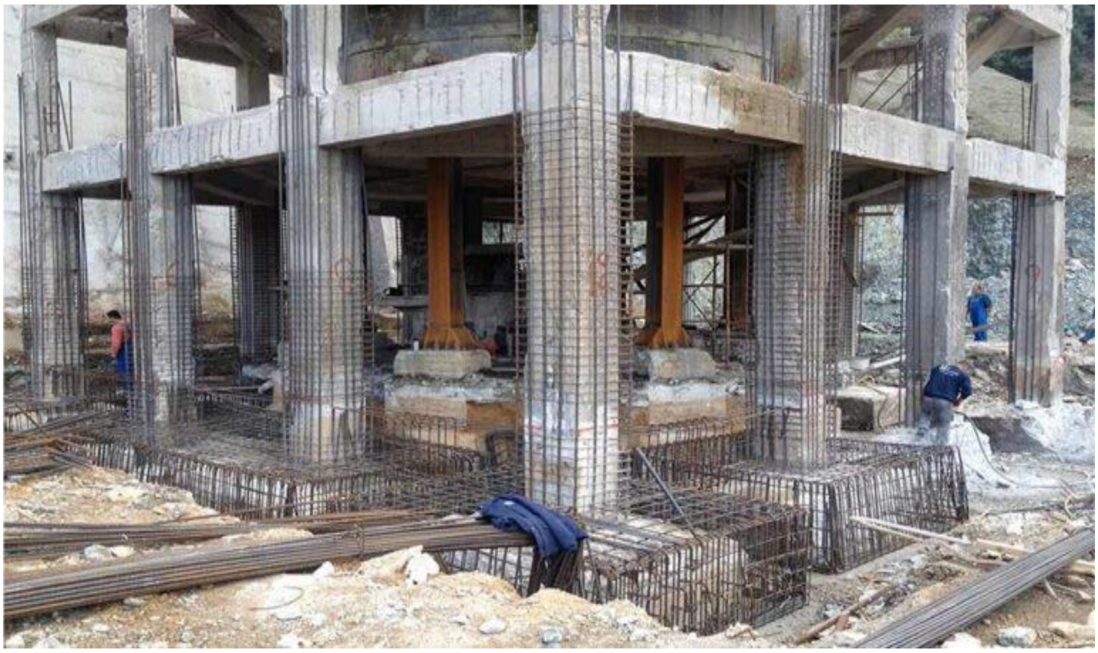


Fig. 1.4 Jacketing of column and foundation in a building (Ref: www.tutorialtipscivil.com)

Fig. 1.5 Jacketing of Column and Beam (Ref: www.tutorialtipscivil.com)

Reinforced Concrete Jacketing involves securing additional longitudinal and lateral steel reinforcement around an existing structure, followed by the placement of formwork. The void between the original member and the newly placed formwork is subsequently filled with either regular or modified concrete. The utilization of regular concrete is discouraged due to its propensity to shrink, potentially resulting in inadequate bonding with the old concrete. Various different method of column jacketing are shown in Fig.1.6.

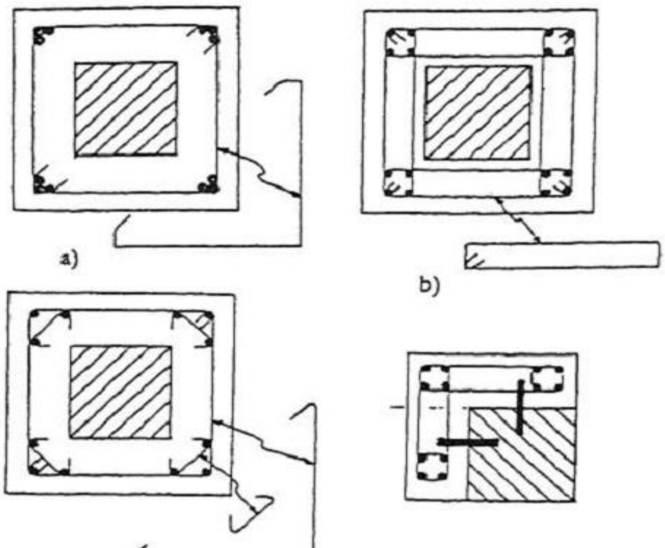


Fig. 1.6 Various methods of Reinforced Column Jacketing (Ref: www.sciencedirect.com)

Therefore, micro concrete is used as it does not undergo shrinkage. The inclusion of reinforced concrete as a jacket enhances the flexural strength, ductility, and shear strength of the column. While achieving effective jacketing is relatively straightforward for circular columns, it poses challenges for square columns. The vertical bars located in the mid-region of each face of the column are vulnerable to buckling, whereas the corners are adequately confined. To address this issue, a hole is drilled through the column, and cross-links are installed, as illustrated in Fig.1.7.

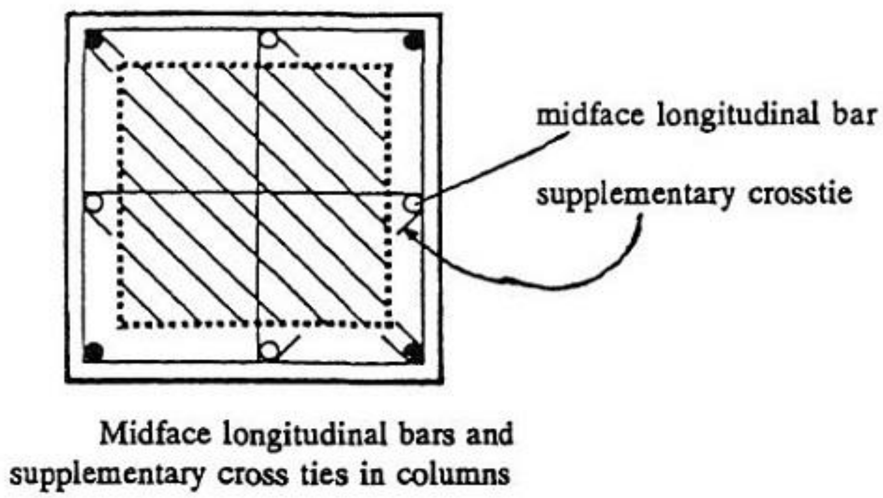


Fig.1.7 Reinforcement detail of Column Jacketing (Ref: www.sciencedirect.com)

1.1.4.2 Bracing

Bracing systems are important in structures as they primarily endure lateral loads caused by earthquakes or wind. The purpose of bracing is to reduce the lateral deflection of the structure. One of the main reasons for incorporating it is to control the buckling of the primary beams. Steel bracings, despite their relatively small tonnage, offer significant resistance against beam bending.

Steel bracings are commonly utilized in the rehabilitation of nominally ductile moment-resisting frame structures. They possess the necessary strength, stiffness, and ductility required for the structure. However, careful attention must be given to their connections with the existing structure. Steel bracings can also enhance the seismic performance of reinforced concrete shear structures. In such cases, the steel bracing can be anchored to the reinforced concrete structure at short intervals, minimizing the buckling length. This increases the capacity of the bracing member compared to retrofitting moment-resisting frames, where the capacity is primarily governed by the buckling of the compressed bracing member. There are many ways to provide steel bracing (Fig.1.8)

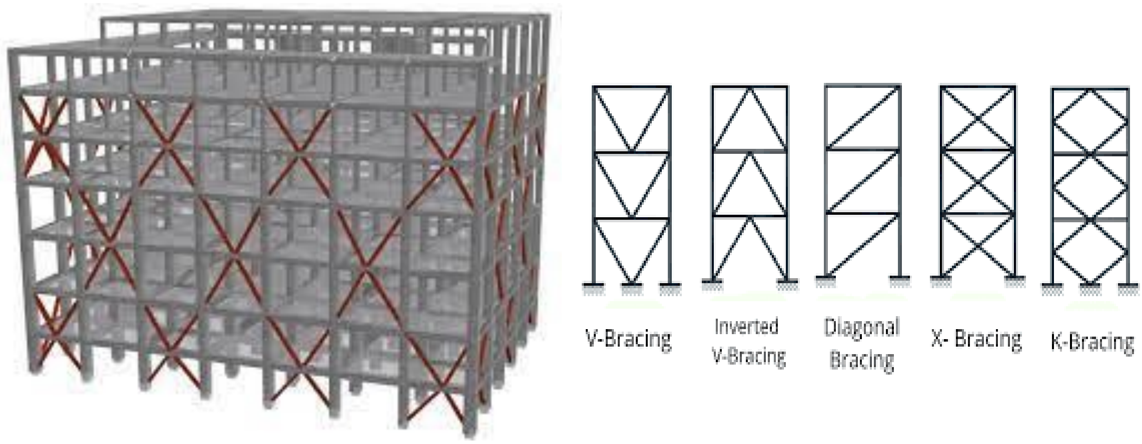


Fig. 1.8 Different ways in which steel racing can be provided (Ref: www.seismosoft.com)

When using diagonal bracings, it is advisable to add vertical steel strips at the edges of the structure. This is because the diagonal forces in the bracing members have a vertical component (compression/tension) that exerts higher forces on the structure. By providing vertical strips at the ends of the structure, a portion of these forces can be resisted by the concrete.

1.1.4.3 Beam Addition

The transfer of load from secondary beams to main beams, and subsequently from the main beams to the columns, is a vital aspect of structural design. In cases where there is asymmetric loading between the two beams, one main beam may experience greater stress than the other. To achieve a more balanced distribution of the load to both beams, the introduction of a secondary beam between the two main beams can be implemented. This helps ensure an even distribution of the load across the structural members.

1.1.4.4 Shear Wall

Shear walls are essential vertical components of a structural system designed to resist horizontal forces. These walls are typically straight and extend over the entire height of the building (Fig.1.8). Commonly, they are positioned at the lift and staircase core regions. Shear walls possess significant depth in the direction of lateral loads. They function by bending like a vertical cantilever fixed at the base, experiencing bending moments and shear forces primarily due to lateral loads, along with axial compression from gravity loads.

Their design focuses on resisting lateral forces parallel to their plane and providing ductility to the structure. By offering structural stability and lateral load resistance, shear walls play a crucial role in enhancing the overall seismic performance of buildings and ensuring their safety during earthquakes.

Fig. 1.9 Depiction of Shear Wall in a live project (Ref: www.tutorialtipscivil.com)

In high-rise buildings, which are vulnerable to lateral wind and seismic stresses, shear walls are extremely vital. Shear walls often have a plane or flanged element at its ends called the boundary element. They offer adequate stiffness and strength. These walls are positioned in such a way in building plans to minimize lateral displacements caused by lateral loads. They transmit the lateral loads to the components in the load path below them. These components in the load path could be additional floors, slabs, foundation walls, or shear walls. Shear walls also provide lateral stiffness to prevent excessive side-sway of the roof or floor above it. Shear walls that are sufficiently rigid will keep floor and roof framing members against sliding from their supports. Therefore, structures with shear walls sustain less structural damage.

Shear wall should extend into every floor of the building. Equal-length shear walls should be symmetrically installed on the outer periphery of the building to create a box like structure. Shear walls perform best when they are vertically aligned and supported by footings or foundation walls.

➢ Requirements of RC Shear Wall as per Indian Standards

Shear walls play a crucial role as a part of the lateral force resisting system in earthquake-resistant RC buildings. They are designed with uniformly spaced reinforcement in their cross-section along both vertical and horizontal directions. The vertical steel bars are enclosed within the horizontal steel bars, creating a closed core concrete area with closed loops and cross ties. This configuration ensures enhanced strength and ductility, contributing to the overall seismic performance and safety of the building. Following are the minimum requirements for a RC Shear wall:

- The minimum thickness of shear wall is to be 150 mm for single shear wall system and 300 mm for coupled shear wall system.
- The minimum ratio of length of wall to its thickness is to be 4.
- The maximum diameter of longitudinal steel bars in the shear wall should not exceed one tenth of the thickness of shear wall.
- The maximum spacing of vertical or horizontal reinforcement shall not exceed:
    1. One fifth of the length of the wall.
    2. Three time the thickness of the wall
    3. 450 mm
- The shear walls should have a properly designed foundation resting on soil and should not be discontinued to rest on beams, columns or inclined members.

- The minimum percentage of steel bars to be provided along vertical and horizontal directions range between 0.25 % to 0.8 % depending upon the type of wall.

Shear wall are categorized as follows:

1. Squat Wall: The ratio of height of wall to length of wall is less than 1.
2. Intermediate Walls: The ratio of height of wall to length of wall is between 1 and 2
3. Slender Walls: The ratio of height of wall to length of wall is greater than 2

Boundary Elements of a Shear Wall

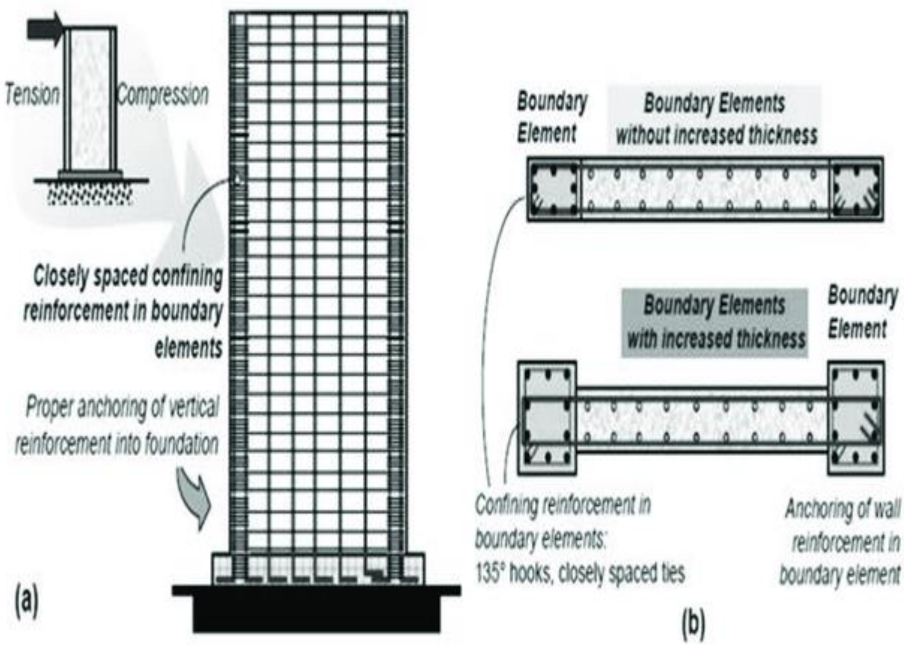


Fig. 1.10 Boundary Element in Shear Wall (Ref: www.tutorialtipscivil.com)

Boundary Elements are portions along the wall edges that are strengthened by the longitudinal and transverse reinforcement (Fig. 1.9). The thickness of boundary element can be kept same or greater than the thickness of the inner core. These elements are required when extreme fibre compressive stress in the wall exceeds 0.2 f_{ck} due to factored gravity loads plus factored earthquake force. They can be discontinued at levels when extreme fibre compressive stress becomes less than 0.15 f_{ck}. The vertical reinforcement in the boundary element shall not be less than 0.8 % and greater than 6 %.

- Coupled Shear Wall

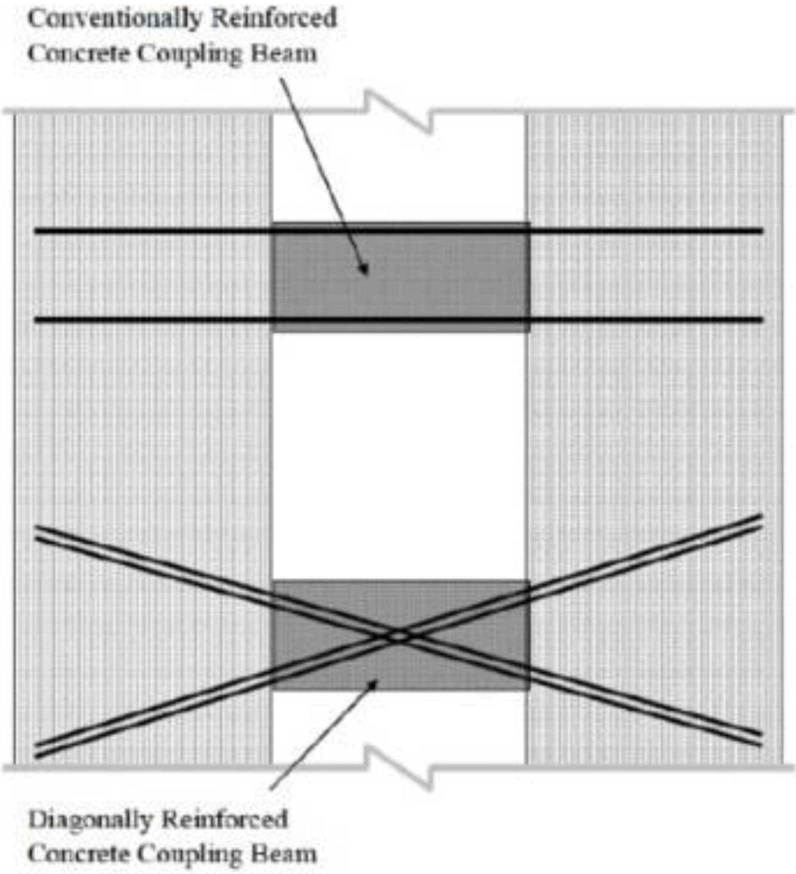


Fig. 1.11 Coupled Shear Wall (Ref: www.seismosoft.com)

Coupled Shear wall is the shear wall arrangement in which the two isolated shear walls are connected by means of coupling beams thereby increasing the total stiffness of the walls and also the cantilever action of the walls is restrained (Fig. 1.4). It also provides large ductility to the structure and sustain much less damage. It has been seen that in this type of structure the hysteresis curve of load vs. displacement behaviour is very stable. Thus this type of structure meets the requirements of seismic resistant design most satisfactorily. In this, the diagonal reinforcement of coupling beam is anchored into the adjacent shear walls by adequate anchorage length. The diagonal reinforcement of the coupling beams and the ties are provided to form the compression strut to sustain yield load without buckling.

2. LITERATURE REVIEW

Burak & Comlekoglu (2013) investigated in this research the influence of shear wall to floor area ratio on the seismic response of midrise RC structures. The study aimed to examine the structural behavior of midrise buildings with different shear wall area to floor area ratios under seismic loading, observing how the overall behavior improved with higher shear wall ratios. To assess the impact of shear wall area to floor area ratio on the seismic performance of RC structures without torsional anomalies, nonlinear time history analyses were conducted. The analyses included roof drift, inter-storey drift, and base shear responses.

Twenty Four midrise building models with no. of floors between five and eight, and shear wall ratios between 0.51 and 2.17 % in both directions were designed for this use. Then, using nonlinear time history analysis, the behaviour of these models under earthquake loads was investigated. Seven separate ground motion records were applied to the building models during the analyses, and the average of the data collected was utilised to assess the seismic performance. The roof and inter storey drifts as well as the base shear responses were the primary factors taken into account in this research that impact the overall seismic performance of the buildings. According to the analytical findings, midrise structures should be designed with a shear wall ratio of at least 1.0 % in order to control drift. Furthermore, it was shown that the seismic performance did not significantly improve as the shear wall ratio rose above 1.5 %.

As the shear wall ratio increases, the observed drift in midrise RC structures reduces. However, beyond a shear wall ratio of 1.5%, the reduction in maximum inter-storey drift and roof drift values becomes less significant compared to the reduction observed for changes in shear wall ratio below 0.5%. For five-storey structures, it was found that including at least 1.0% of the shear wall ratio in the design is essential for managing the drift in a shear wall-frame structure. The analysis of five-storey building models with increasing shear wall ratios showed no apparent relationship between the time of first yield of load-bearing elements and the shear wall ratio, as depicted in Fig. 2.1 & 2.2.

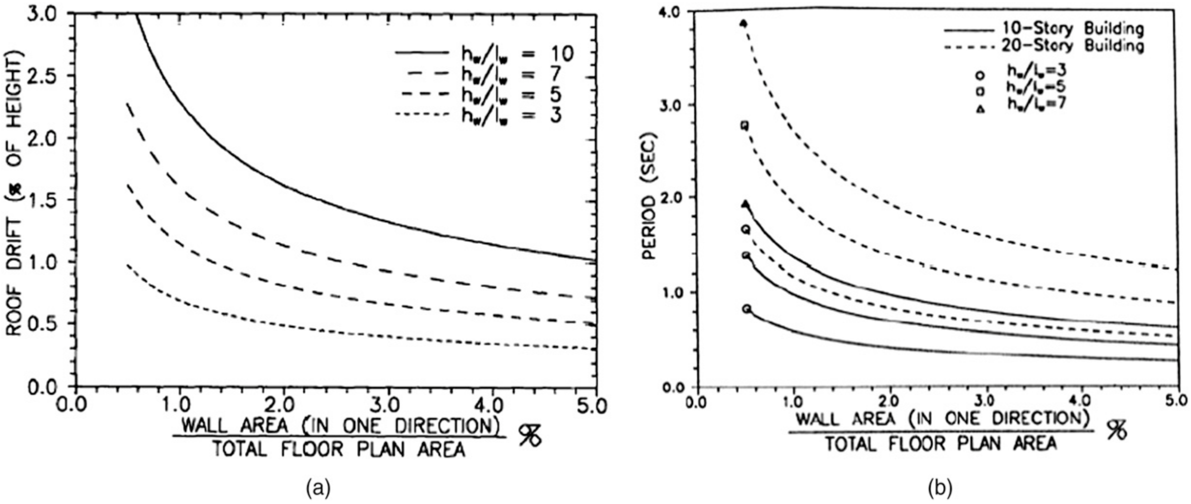


Fig. 2.1. (a) Roof drift versus shear wall ratio (Wallace 1994; ASCE 2006); (b) estimated fundamental periods for shear wall buildings (Wallace and Moehle 1992; ASCE 2006)

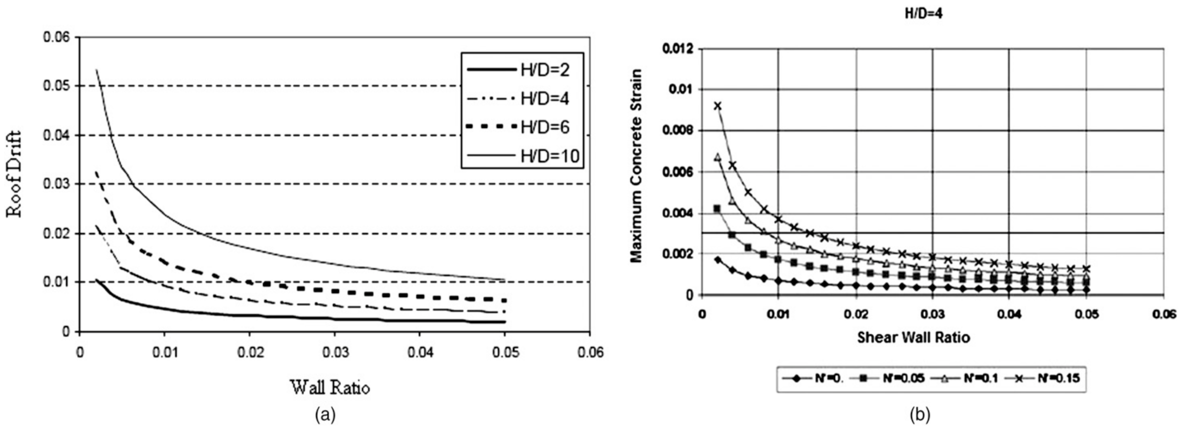


Fig. 2.2 (a) Estimated roof drift; (b) estimation of shear wall ratio for constant aspect ratio and variable axial load (data from Gulkan and Utkutug 2003; ASCE 2006)

Therefore, the total number of members that yield varies depending on the shear wall ratio in various building models. Even in the presence of significant ground vibrations, the percentage of yielded members decreases as the shear wall ratio rises up to 1.5 %. After 1.5 % shear wall area to floor area ratio, this effect seems to decrease. Roof drift and inter-storey drift often reduce as the ratio of shear wall area to floor area rises. While the magnitudes of the drift values for two orthogonal directions are comparable for 8-storey buildings, the difference is more prominent for 5-storey buildings, especially for shear wall ratios of 1.0 % and higher. This was suggested to be related to drift amplifications in the direction of the strong component, caused by the fundamental periods of the five-storey structures being close to the dominant periods of the chosen earthquake records.

Although maximum inter-storey drifts were often larger in magnitude than roof drifts, their general trends were relatively comparable. The highest inter-storey drifts were seen between the top two stories of both 5- and 8-storey buildings when the shear wall ratio was higher than 0.5 %. Until the shear wall ratio reaches 1.5 %, the proportion of the base shear force experienced by the ground floor shear walls rises significantly. Between 0.5 and 1.0 % shear wall ratios, however, the rate of increase was significantly larger than between 1.0 and 1.5 %. Shear walls on the ground floor of structures with a shear wall ratio of 1.0 % support around 80 % of the base shear. For shear wall ratios of 1.5 % and 2.0 %, it is noted that the base shear force carried by the walls is greater than 90 %; however, the impact of this increase on the drift performance was not significantly different.

Heerema et al (2015): In this study, a two-storey reinforced concrete block scaled building underwent fully reversed quasi-static displacement-controlled stress testing until failure occurred.

The building's seismic force-resisting system (SFRS) comprised eight structural walls, with four walls asymmetrically placed along the loading direction to establish a centre of rigidity eccentricity roughly 20% of the building width away from the floor center of mass, as shown in Figure 2.3. To prevent torsion, the other four orthogonal walls were symmetrically positioned around the floor's center of mass. All walls, except one flanged wall, had rectangular cross-sections and were connected by two RC slabs representing floor diaphragms.

To facilitate building twist through diaphragm action and reduce slab-wall moment transmission to prevent coupling, hinge lines were incorporated during the slab construction process. The structure underwent fully reversed displacement-controlled loading cycles until it experienced approximately 50% of its original strength.

The primary aim of this study was to evaluate the impact of twist on the ductility capacity of the structure and the ductility and strength requirements of its wall components. Throughout the structure's loading history, each wall experienced distinct displacement and strength demands. The interaction between the system-level twist reaction and the resulting displacement demands on each wall component was crucial. The analysis emphasized the significance of considering the variation in inelastic response characteristics of the individual walls that constitute the SFRS. Additionally, it underscored the contributions of wall strength to the overall building capacity and the subsequent mobilization of ductility levels when assessing the overall building SFRS performance.

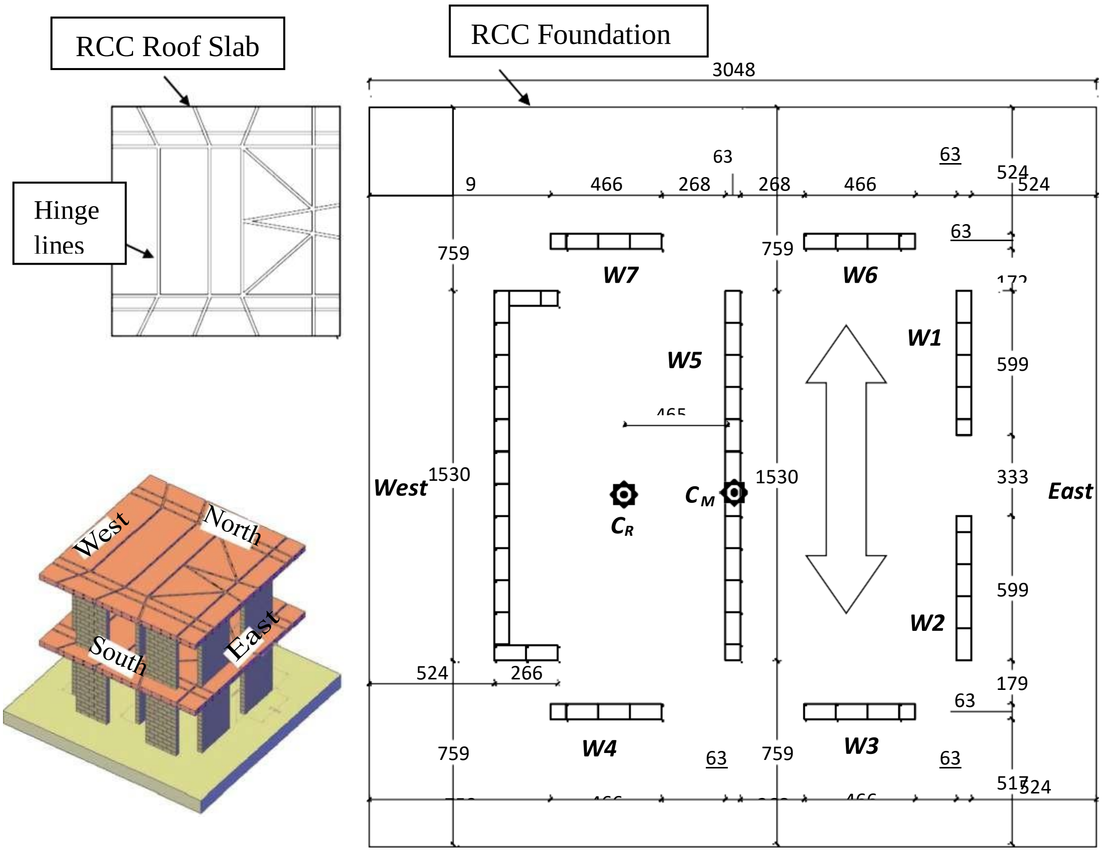


Fig 2.3 Arrangements of Walls (Heerema et al (2015))

The deflection profile of the building walls resembled that of a cantilever throughout their heights. Hysteretic loops indicated symmetrical responses for both loading directions. The slopes of the loading portions of the loops gradually decreased with increased lateral top displacement, similar to previous experiments on individual walls. Up to 80% of its maximum capacity, or approximately 0.4% top drift, the building exhibited an almost linear elastic response. At low displacement levels, the load-displacement response showed narrow, symmetrical hysteresis loops. However, at higher displacement levels, the loops widened, resulting in decreased capacity at the same displacement level due to stiffness and strength degradation caused by higher inelastic deformations.

The variation in the inelastic response properties of the walls resulted in component-system interaction, impacting both the load redistribution upon damage within the wall components and the overall building's ductility capacity of the Special Moment Frame System (SFRS). The building's twist influenced displacement and, consequently, the strength demands of the individual walls that composed the SFRS. To quantify the building's ductility, a bilinear relationship was established, describing the nonlinear load-displacement relation.

At maximum load, which corresponds to 1 % drift, and at 20 % strength degradation, which corresponds to 2 % drift, the load-displacement relationship was modeled. In general, 2.15 and 4.2 were the computed ductility values for the maximum load and 20% strength degradation, respectively. The experimental data can be used for future benchmarking of analytical modelling methodologies.

Ugalde & Lopez-Garcia (2017) In this study, the response history analysis was used to assess three structures that were unaffected by the 2010 Chile earthquake while taking into account the ground vibrations recorded at the nearby stations. Linear response history analysis was done because the buildings were intact. Effective strength and stiffness parameters are taken into account by the building models. As is customary in Chile during the design phase, each leg of each non-rectangular wall was assessed separately in the first analysis.

It was discovered that in many walls' demand-to-capacity (D/C) ratios are higher than unity, which conflicts with the lack of damage that has been seen. In the second analysis, the exact cross-section of non-rectangular walls was taken into consideration when calculating the demand and capacity (the latter in complete fibre models), although no appreciable changes in the D/C ratio values were found. Finally, foundation uplift was taken into consideration in a third analysis (nonlinear analysis), which produced much lower D/C ratio estimates that are more (but not entirely) consistent with the lack of damage that was actually seen.

The prospect of foundation uplift is then put out as a way to prevent harm. However, the D/C ratio is still higher than unity in many walls, indicating that little intrusions into the nonlinear range may have taken place without exhibiting any damage.

Deepna U et al (2018) This study focused on the effects of lateral loads, including earthquake, wind, and blast forces, which are major concerns in structural design. It aimed to ensure sufficient strength and stability to withstand lateral stresses in tall structures. The research compared the seismic performance of (G+20) story buildings and explored the optimization of RCC, Steel Plate, and Composite Shear Wall thicknesses. Using ETABS software, the design and analysis of the buildings were conducted. The study provided a detailed examination of different. The following broad conclusions were drawn from the work completed and analytical research conducted for the project.

- When the thickness of the RCC shear panel and steel plate is decreased, the base shear decreases.

- The RCC shear walled construction has the lowest base shear, whereas the Steel plate shear walled structure experiences the highest base shear.

- The basement storey drift of the structures is unaffected by the thickness of the RCC panel and steel plates.

- The structural system's storey drift is influenced by the thickness of the RCC panel and the steel infill plate.

-The storey drift is observed to be increasing as the RCC panel's thickness is reduced.

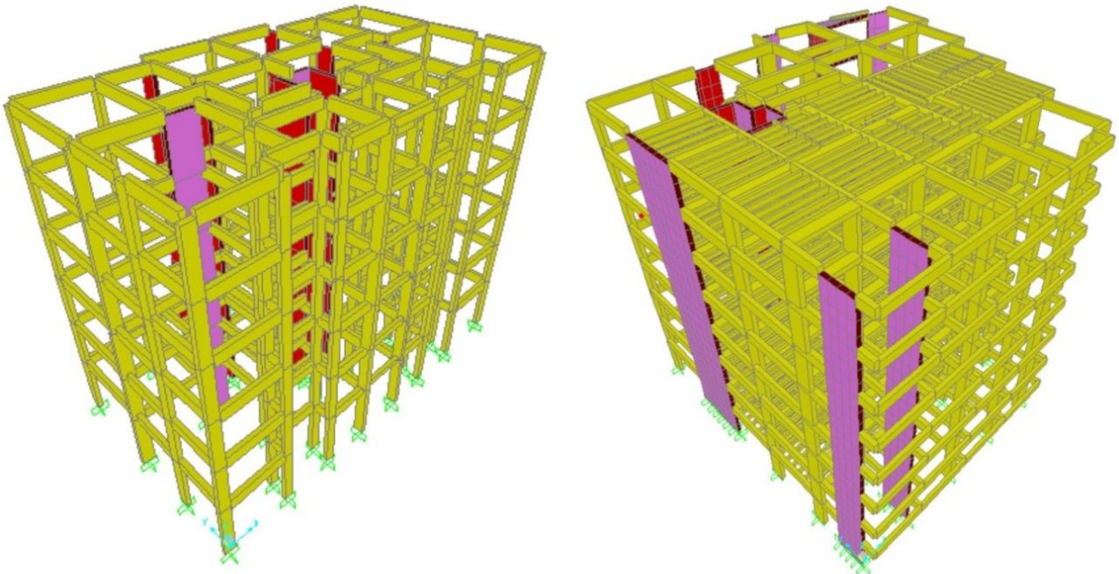
- For steel plate shear walls and composite shear walls, it is discovered that storey drift is increasing while steel plate thickness is reduced.

Banerjee & Srivastava (2019) considered a G+15 storey structure. The structure is irregular in nature (T shaped). To determine the shear wall's ideal location within the building, a comparative analysis was conducted. The shear wall's overall length in the structure was kept constant for optimization purposes. ETABS v. 2016 was used for all of the modelling and analysis. The base shear, storey displacement, and storey drift were used as the basis for the comparison analysis. Ten test models were created and examined for the purpose of conducting a comparison analysis to identify the ideal location of the shear wall.

One bare frame model was among the ten test models (with no shear wall). In each of the other nine models' shear walls were set up differently. The overall length of the shear wall was 80 m in all nine models. The comparative investigation led to the conclusion that the shear wall's position is crucial in improving the resistance to lateral stresses. The building should maintain its centre of gravity as best it can by having the gravity loads and lateral loads distributed appropriately by the site.

The configuration of model with shear walls at and around the building's core was chosen so that it distributes lateral stresses as evenly as feasible. Spectral displacement, storey-drift, and storey displacement values are thus decreased as a result of the stresses caused by earthquakes. Additionally, seismic forces increase in the buildings in terms of base shear. This suggests that structures with shear walls can withstand greater seismic loads.

Ozkul et al (2019) This study looked into how a shear wall affected an RC building's seismic performance. Two different RC frame structures namely A and B with shear walls that were located in Van, Turkey and suffered structural damage as a result of the Van earthquake in 2011 were taken into consideration (Fig.2.4). To determine the damage states of building structural elements, buildings were modeled in SAP 2000 and nonlinear time history analysis was carried out utilising Van 2011 Earthquake acceleration records. Based on the strain limitations listed in TSC 2007, damage states were established.



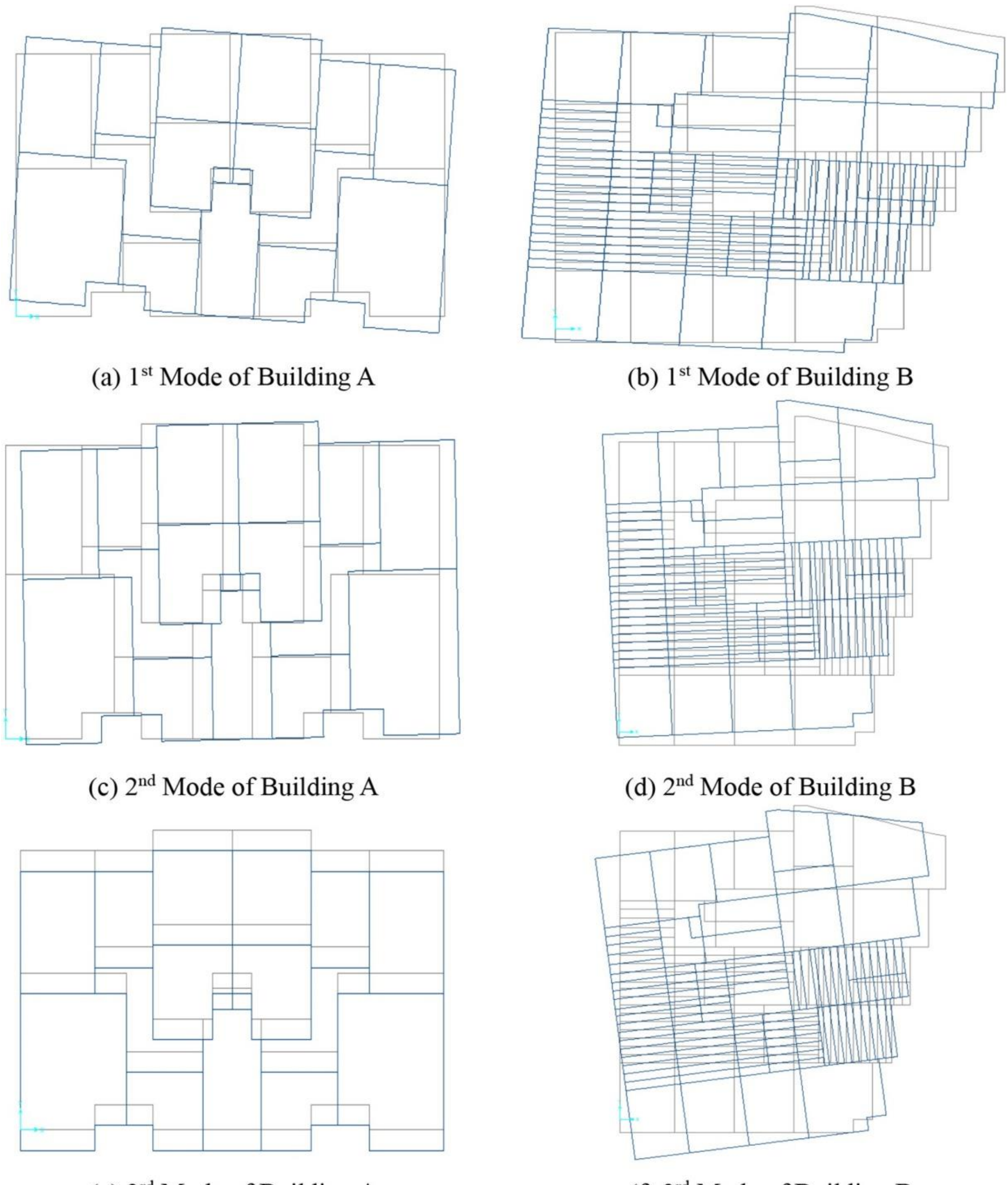
(a) Building A (b) Building B

Fig.2.4 Analytical models of Building A and Building B (Ozkul et al (2019))

In the Van 2011 earthquake, one of the buildings, Building A, sustained major damage, whilst Building B sustained far less damage. Building shear wall reinforcements were later designed in accordance with TSC 2007 specifications, and damage states of structural components were evaluated using nonlinear time history analysis (Fig.2.5). So, even if the reinforcement of shear walls was planned in accordance with a contemporary seismic code, the seismic performances of structures were studied. The damage conditions of existent buildings and those that had been analysed were also compared. Through nonlinear time history analysis, it was attempted to identify the actual damage states. The findings of this study allow for the following conclusions:

Shear walls have a significant impact on the seismic performance of RC buildings, and they improve building performance. Even if columns have poor ductility, it can be claimed that well constructed shear walls reduce the damage levels of structural elements like columns and beams. In other words, shear walls can significantly help buildings resist earthquake loads if their hysteretic energy damping ability and corresponding ductility capacity are high.

Even if their dimensions are built within acceptable ranges from an engineering perspective, shear walls with insufficient reinforcement do not improve the seismic performance of buildings. The expected impact of shear walls on seismic performance does not happen if the strength and ductility of the shear walls do not satisfy codal standards.



(a) 1st Mode of Building A (b) 1st Mode of Building B

(c) 2nd Mode of Building A (d) 2nd Mode of Building B

(e) 3rd Mode of Building A (f) 3rd Mode of Building B

Fig. 2.5 Mode shapes of Building A and Building B. (Ozkul et al (2019))

Existing buildings exhibit poor lateral and vertical reinforcements, along with low compressive strength of concrete, resulting in low ductility and strength. These limitations underscore the need for structural improvement and seismic retrofitting measures to enhance the buildings' overall safety and resilience against potential seismic events. To get improved seismic performance, frame buildings with shear walls were designed according to modern seismic code requirements, named Building A2 and Building B2.

30 MPa Concrete Strength and 420 MPa Yield strength of reinforcement bars were used for both new buildings. The study employed nonlinear time history analysis to determine the damage states of structural elements. It investigated the seismic performance of buildings, even when shear wall reinforcements were designed according to a modern seismic code. Additionally, a comparison was made between the damage states of the analytically modeled building and existing buildings. The objective was to ascertain the actual damage states through this comprehensive time history analysis approach. Following conclusions can be drawn from the results of this study:

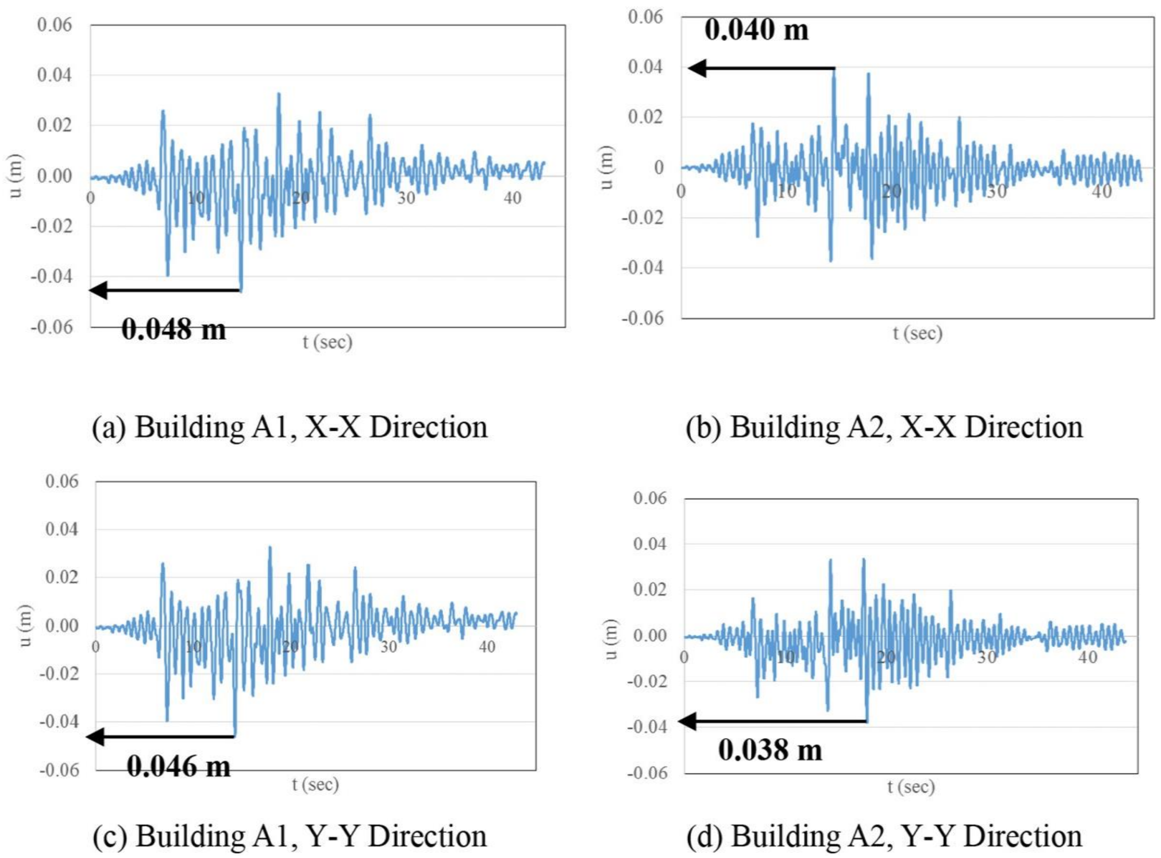


Fig.2.6 (a) Displacement time histories of Building A1 and Building A2 (Ozkul et al (2019))

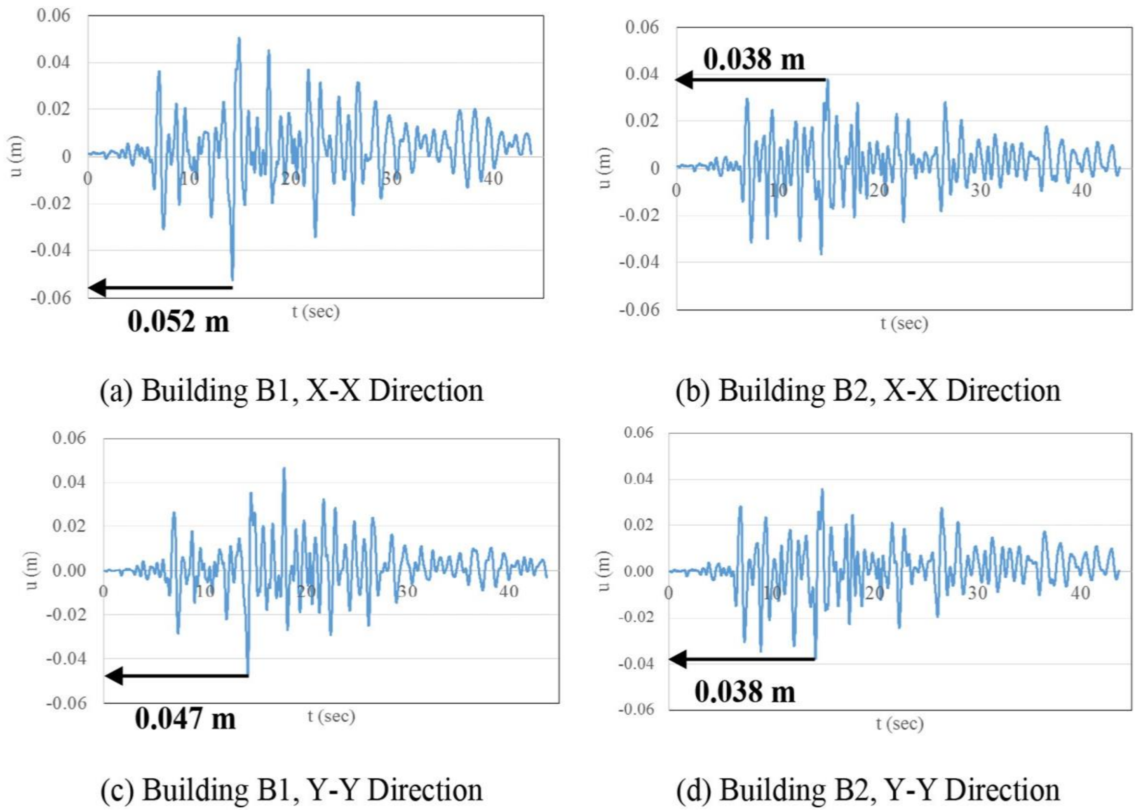


Fig.2.6 (b) Displacement time histories of Building B1 and Building B2

(Ozkul et al (2019))

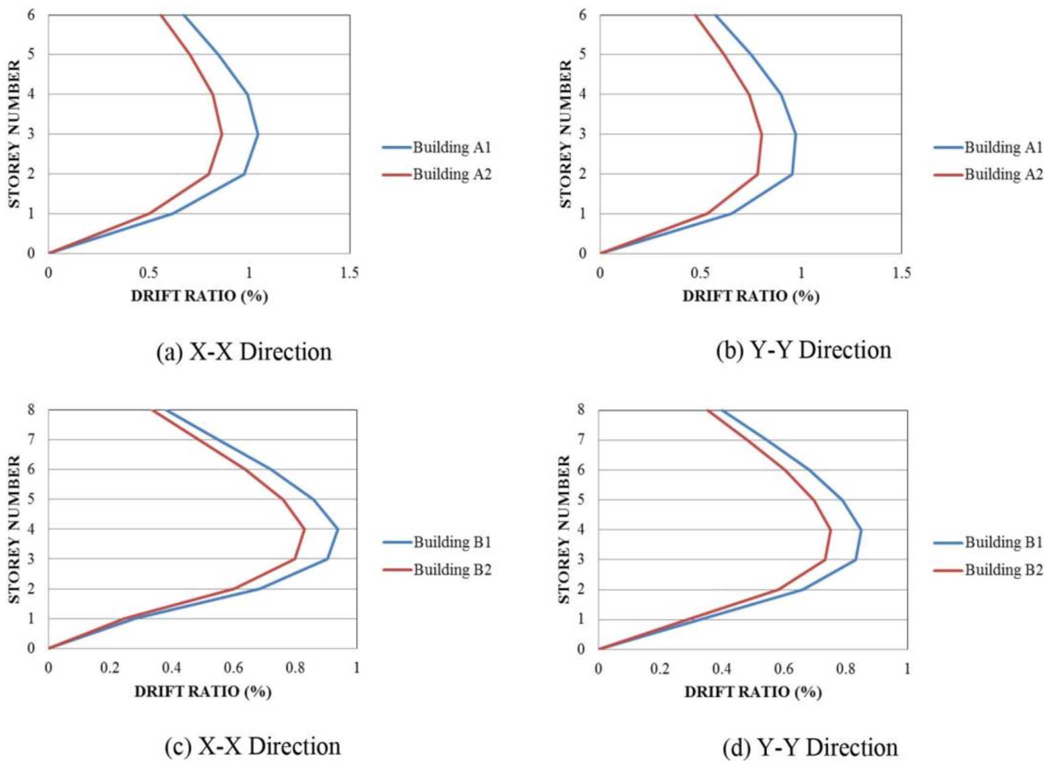


Fig. 2.7 Interstorey drift ratio of Building A and B in each orthogonal direction

(Ozkul et al (2019))

- First mode periods of Building A1 and Building B1 were estimated as 0.72 sec and 1.67 sec, respectively. First mode periods decreased to 0.60 sec for Building A2 and 1.51 sec for Building B2 with the increasing compressive strength of concrete.

- In X-X direction: Displacement demands of Building A1 and Building A2 are 0.048 m and 0.040 m, respectively. Displacement demands of Building B1 and Building B2 are 0.052 m and 0.038 m, respectively. In Y-Y direction: Displacement demands of Building A1 and Building A2 are 0.046 m and 0.038 m, respectively. Displacement demands of Building B1 and Building B2 are 0.047 m and 0.038 m, respectively. (Fig. 2.6 (a) & (b))

- Between Building A1 and Building A2, in X-X direction, maximum drift ratio decreased with the ratio of 20 % and in Y-Y direction this decreasing is 25 %. (Fig.2.7)

- Between Building B1 and Building B2, in X-X direction, maximum drift ratio decreased with the ratio of 5.5 % and in Y-Y direction this decreasing is 3.75 %. (Fig.2.7)

- Since the column dimensions and stiffness are relatively small, shear wall has considerable effect on displacement and drift ratio.

- The number of columns in each damage state indicates that well-designed shear walls can reduce damage levels in structural elements like columns and beams, even when the columns have low ductility. This suggests that shear walls with high hysteretic energy damping capacity and ductility can significantly support buildings in resisting earthquake loads.

- Shear walls with insufficient reinforcement fail to enhance the seismic performance of buildings, even if their dimensions are within acceptable engineering ranges. When the strength and ductility of shear walls do not meet code requirements, the expected positive impact on seismic performance does not materialize.

- Performance-based evaluation of an RC building yields accurate results when nonlinear time history analysis is employed. The outcomes closely align with the actual building performance, as the analysis uses the same earthquake record to subject the existing building.

When nonlinear time history analysis is performed, performance-based evaluation of an RC structure produces results that are fairly close to actual performance. As the structure that was already there was subjected to the identical seismic records that were used to determine this finding. It can be stated that when a structure's performance is evaluated using a nonlinear time history analysis, ground motion data should be chosen so that they closely resemble earthquakes whose future occurrence in the region where the building was built is likely.

Anya & Ghosh (2021) focuses on two distinct models, one utilising shear walls at various places and the other without shear wall. The paper presents the parametric analysis in terms of base shear, storey drift, and displacement comparison for various shear wall locations in the building. The study used SAP 2000 software to analyse the RC frame structure (residential building), changing the location of shear walls throughout the structure to determine where they should be placed for maximum resistance to earthquake force, storey drift, and actual building behavior.

Eight models in total were created, two of which had no shear walls and the other six had different shear wall placements for residential buildings. It was a basic structure with identical-sized columns throughout the height and identical-sized beams in both directions. Models 1 and 2 share the same material characteristics but differ in size.

Models G+4 and G+6 of RC frames were first created using SAP 2000, and each model had a different shear wall position. The foundation of the structure is fixed, and the capacity ratio was also examined (i.e. as per IS code 456 the capacity ratio of column beam should be more then 1.2). The most effective model was with shear wall at core as well as corner. (Fig. 2.8 & Fig. 2.9) and its results are shown below.

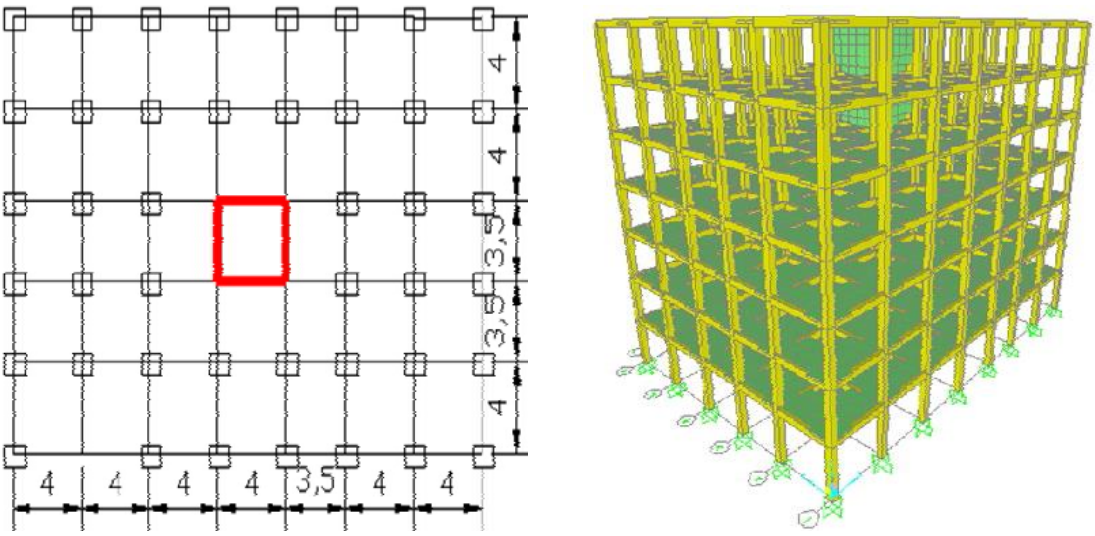


Fig.2.8 PLAN AND 3D VIEW WITH SHEAR WALL AT CORE

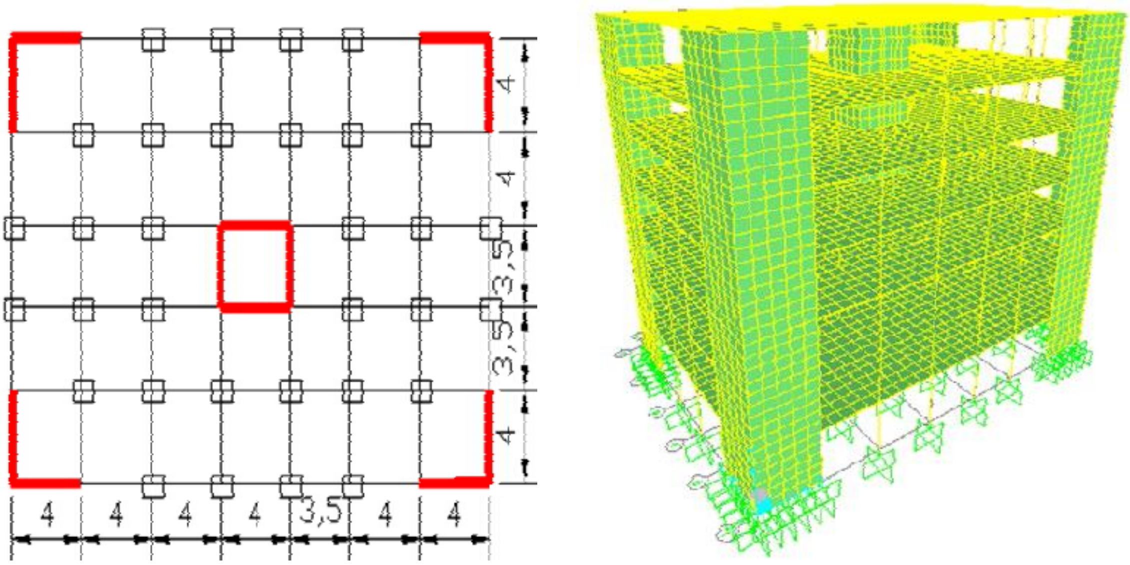


Fig.2.9 PLAN AND 3D VIEW WITH SHEAR WALL AT CORE AS WELL AS CORNER

When comparing the types of shear wall placement, it has been found that the location of shear walls in the core and corners exhibits the least amount of storey drift. However, in this situation, more base shear was seen in both the x and y directions. A similar pattern of displacement was seen in the case of the bare frames. According to IS code 1893:2002, the storey drift cannot exceed 0.004 times the storey height. Thus, storey drift does not surpass its limit, as per the findings. (Fig. 2.10, 2.11, 2.12, 2.13)

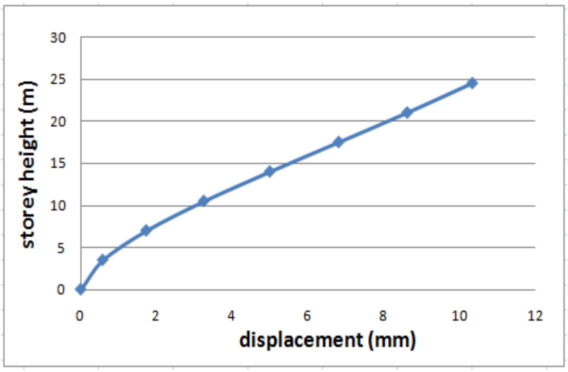


Fig.2.10 (a) height vs. displacement curve with shear wall placed at the centre as well as at corner of the structure along x-direction

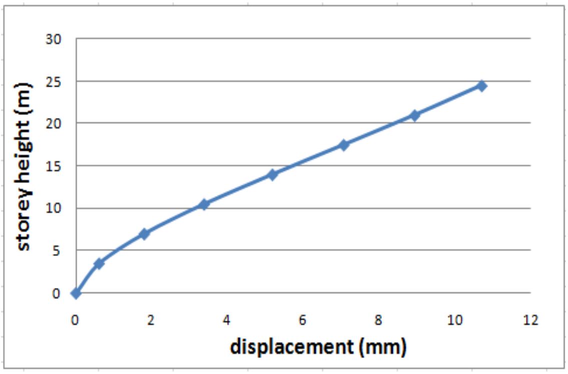


Fig.2.10 b) height vs. displacement curve with shear wall placed at the centre as well as at corner of the structure along y-direction

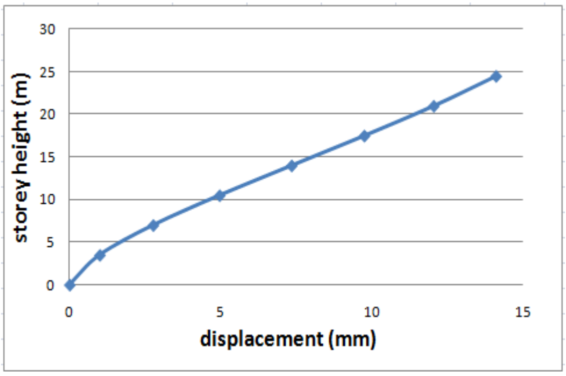


Fig. 2.11 (a) height vs. displacement curve with shear wall placed in core of the structure along x-direction

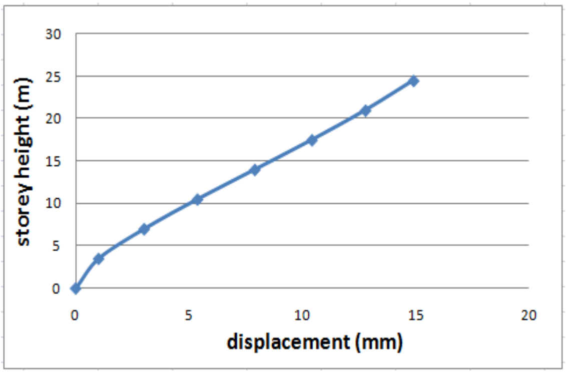


Fig. 2.11 (b) height vs. displacement curve with shear wall placed in core of the structure along y-direction

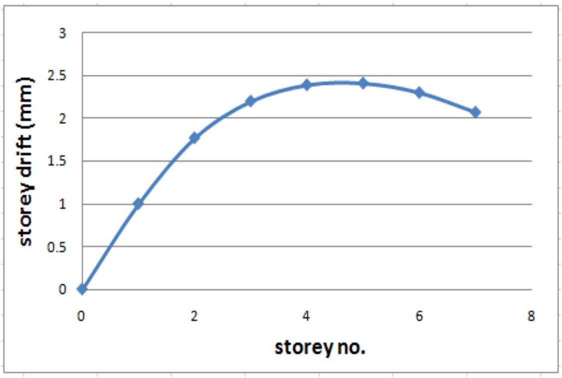


Fig.2.12 (a) storey no. vs. storey drift curve with shear wall is placed in core of the structure along x- direction

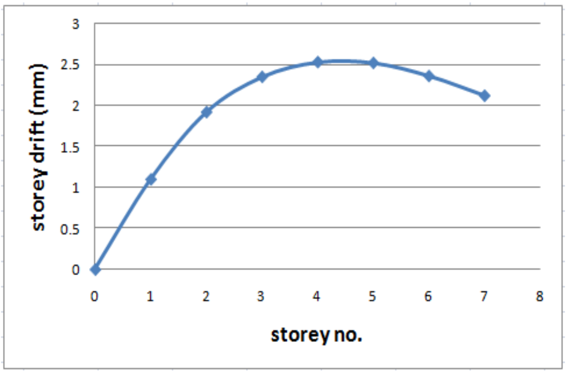


Fig.2.12 (b) storey no. vs. storey drift curve with shear wall is placed in core of the structure along y-direction

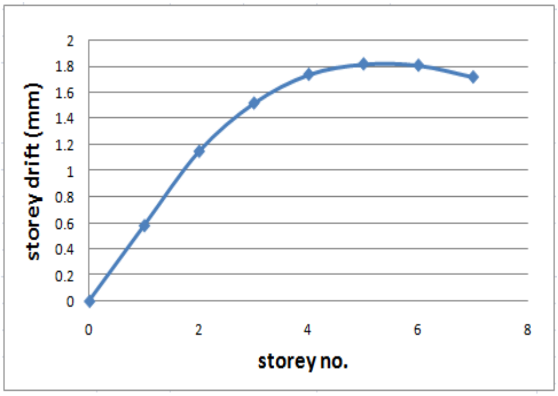


Fig.2.13 (a) storey no. vs. storey drift curve with shear wall placed at the centre as well at corner of the structure along x-direction

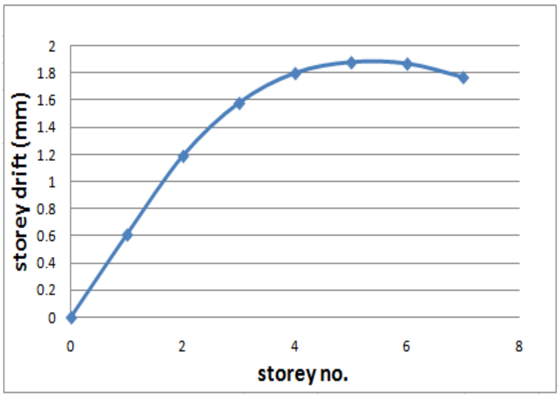


Fig.2.13 (b) storey no. vs. storey drift curve with shear wall placed at the centre as well as at corner of the structure along y-direction

Hosseini & Farookh (2020) In their research, they modeled a G+39 RC framed multistorey building with RC shear walls and fixed support conditions and used Etabs software to analyse the dynamic (Response Spectrum) data. Five distinct models, each forty stories tall and equipped with a shear wall at the building's core, were used for the analysis. Three models without openings and two models with openings conditions have each been examined. Each building model's seismic performance was assessed, and the outcomes were compared. We have discovered that placing shear walls at the centre of each side of the external perimeter and in the outermost perimeter significantly lessens the effects of displacements and drifts.

In terms of maximum storey displacements and storey drifts, Shear Walls performed better. Additionally, the base shear for this case was discovered to be highest. In comparison to previous models, it was also discovered that this model displayed great rigidity. When openings are present, the concentration of stresses in shear walls rises. It was discovered that openings caused the maximum stress to be created to increase threefold. When compared to shear walls without openings, there was a deviation of about 5% due to the presence of openings. In this work, only centre window openings are studied. Both the base shear values and the displacement drifts fell within the 5% range. In order to achieve economy, shear walls with openings are provided.

Shinde B.H. (2020) In this study, performance of RCC buildings with G+12 and G+16 storey was analysed by both equivalent static analysis and response spectrum analysis method (Fig. 2.14). Design results of these building models were studied for zone II, II, IV and V and compared using the finite element software ETABS.

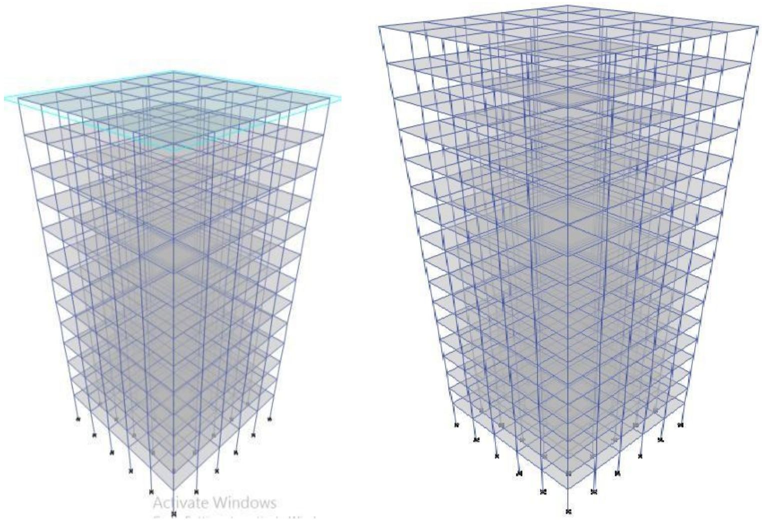


Fig. 2.14: 3D Model of building generated in ETABS (G+12) and (G+16) (Shinde B.H. (2020)

Upon generating the building models using the finite element software ETABS 17, they were subjected to analysis using both the equivalent static method and response spectrum method. Based on the obtained results, the following conclusions were made:

Overall, there was a significant increase in the building's response when analyzed using the new seismic code IS 1893:2016 compared to the previous code IS 1893:2002, regardless of whether the equivalent static or response spectrum analysis was applied. Specifically, the storey shear and base shear experienced a 20% increase after being analyzed with the new code IS 1893:2016, in comparison to IS 1893:2002, for both equivalent static and response spectrum analysis.

This increase in response can be attributed to the change in the importance factor for multi-storey residential buildings, which was increased from 1.0 to 1.2 in the new code. With an increased importance factor, the design acceleration coefficient (Ah) also increases, leading to a subsequent increase in the base shear (VB). Consequently, there may be a need to enlarge the lateral load resisting members and reinforcement, potentially resulting in higher construction costs for the structure.

The response spectra for the Equivalent Static Method and Response Spectrum method were provided separately, and as a result, the Sa/g values were altered. These changes influenced the values of Ah (design acceleration coefficient) and VB (base shear).

In the previous seismic code, IS 1893-2002, the analysis considered the full moment of inertia (M.I.) of columns and beams, assuming an intact section. However, the new code, IS 1893-2016, considers a cracked section with 70% M.I. for columns and 35% M.I. for beams. This consideration is justified by the potential development of cracks in the structure over time. By accounting for cracked section properties in the analysis, it enhances safety. Due to the modification in M.I., a reduction in displacement was observed. The reduction in displacement was up to 134% for a G+12 Building and up to 170% for a G+16 Building.

According to the new seismic code IS 1893:2016, the equivalent static analysis is applicable only to regular buildings with a height of less than 15m in seismic Zone II. As a result, dynamic analysis becomes mandatory for almost all buildings in all seismic zones. It was observed that dynamic analysis led to a significant increase in lateral drift and displacement demand, consequently raising member forces and design requirements. Additionally, the time period of the building increased by approximately 40% for a G+12 building and 51% for a G+16 building. This increase in time period could be attributed to the reduction in moment of inertias of structural elements under the IS 1893:2016 code compared to the IS 1893:2002 code. For a G+12 storey building, the use of IS 1893:2016 in response spectrum analysis resulted in an approximately 156% increment in storey drift compared to IS 1893:2002. Similarly, for a G+16 storey building, there was an approximately 188% increment in storey drift when using IS 1893:2016 compared to IS 1893:2002 in response spectrum analysis.

Mohite et al (2021) Due to the fact that tall buildings are becoming slender, they may be able to sway more than prior high rise structures. In Indian cities, the Floor Spacing Index (FSI) is significantly rising as transportation and safety measures are implemented. Shear walls that are parallel to the direction of the lateral load are typically used to resist the lateral forces of wind and earthquake. By resisting shearing and overturning, these shear walls shift the lateral loads to the foundation.

The primary objective of this paper is to review G+15 storey high-rise buildings with and without shear walls that are situated in seismic zone IV. Shear walls are designed and optimized using the ETAB software, and they are arranged throughout the structure to resist lateral forces in the zone IV region in accordance with Indian codes. In the current study, two models of two distinct types—one with and one without shear-wall structural systems—were modeled in seismic zone IV to examine how well the two types of structures performed when exposed to earthquake structural stresses. The same structural plan dimensions are shared by both models.

According to the analysis, buildings with shear walls have lower maximum displacement, base shear, and storey drift than buildings without shear walls. When subjected to earthquake and wind loading, buildings with shear walls perform better than those without.

Reddy T et al (2021) In this research, a G+15 R.C.C. building was modeled and studied in two parts: one with a shear wall system and the other without one. In order to determine the effective lateral load system during an earthquake in high seismic zones, software based analysis was performed using E-TABS. The building's performance was assessed in terms of base shear, storey drifts, and lateral displacement. The performance of regular buildings is proved to be better to irregular buildings based on the analysis and comparison done for the project.

According to ETABS' analysis, it was found that displacement for both regular and irregular buildings with shear walls is smaller than without shear walls. Furthermore, compared to shear walls, the displacement for regular and irregular buildings is greater. We learned that compared to a building without shear walls, one with shear walls can offer a high level of resistance to the effects of wind and seismic forces.

Cosgun T et al (2022) The study aimed to evaluate the retrofitting technique's effectiveness by comparing the seismic performance of two existing multi-storey RC buildings over an extended period. Seismic performance analysis was conducted based on the Turkish Building Earthquake Code (TBEC-2018) recommendations. The key difference between the buildings was the retrofit technique applied to one of them, which involved adding new shear walls. Following a code-based approach, the performance evaluation results were compared to determine the retrofit's impact on the building's seismic resilience.

The structural system geometry, member dimensions, and plan layout of the buildings were verified by conducting on-site measurements and comparing them with the original design projects. Material properties were determined through field studies and laboratory investigations. Finite element software was used to model the buildings, and numerical simulations were conducted to assess their seismic performance levels according to the applicable building code.

The analysis involved a comparative study of the retrofit approach's effectiveness by evaluating the structural behavior and seismic performance levels of both buildings. The findings emphasized the significance of specific parameters to be considered during the retrofitting process, especially when incorporating new shear walls in RC structures. The retrofitting process enhanced the structure's stiffness and reduced displacements. However, the results also revealed a failure under a shear-critical condition, highlighting the need for further improvements in the retrofitting strategy.

The main aim of this study was to assess the effectiveness of a retrofitting technique and evaluate the seismic performance of two mid-height inactive RC buildings. This evaluation involved numerical analysis supported by field studies and laboratory tests. One of the buildings underwent a recent retrofitting process, which included adding shear walls to mitigate seismic effects and decrease storey drifts. By comparing the seismic performances of both buildings, the study determined the efficacy of the retrofitting application.

The analysis results indicated a decrease in storey drifts and partial transfer of shear forces to the newly added shear walls in the retrofitted building (Fig. 2.15). However, the retrofitted building did not entirely achieve the desired seismic performance level. Furthermore, significant shear forces were observed to be transferred to the shear walls, along with storey drifts. The retrofitting technique led to a reduction in the structural ductile behavior, posing potential risks during seismic events.

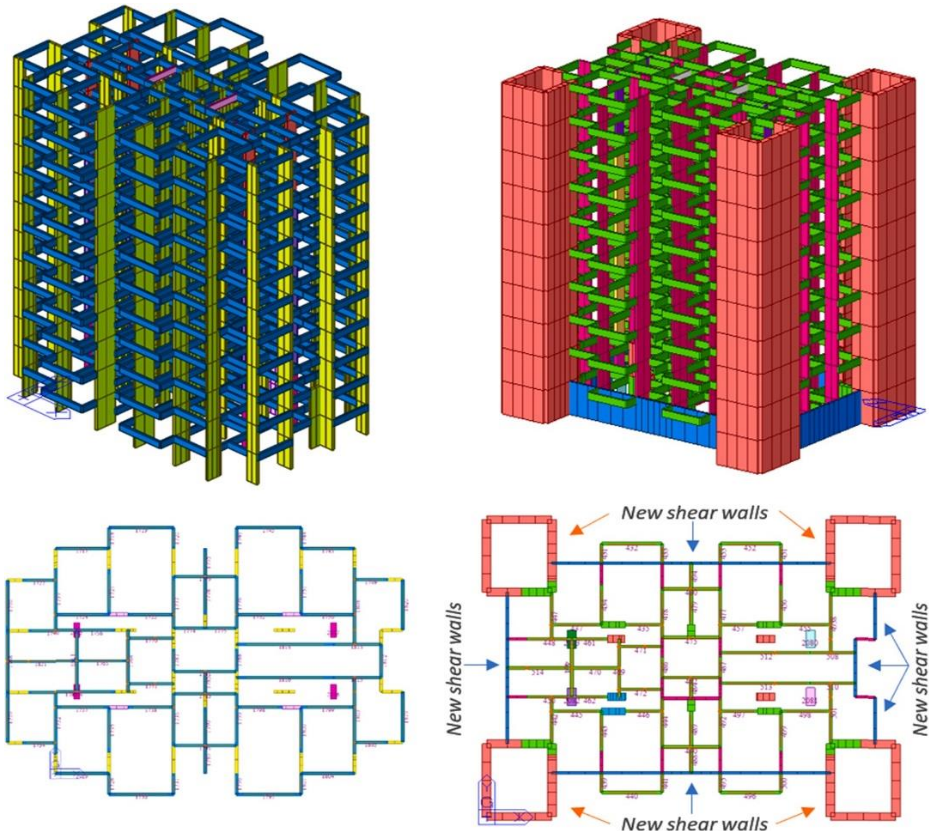


Fig. 2.15 Original Buidling Model and Building Model with new Shear Walls (Cosgun T et al (2022))

To ensure the best practice in retrofitting techniques, it is recommended to incorporate compatible shear walls that can effectively support the existing frame systems, thereby minimizing irregularities and excessive torsional forces. It is essential to ensure that the centers of mass and rigidity overlap as much as possible to avoid the generation of excessive shear forces.

3. METHODOLOGY

The building considered for analysis is an existing residential S+6 building built in the district of Rohtak in Haryana. The existing building was constructed in the year 2013, hence the provisions followed in design were in accordance with IS1893:2002. In this study, this existing structure has been modelled in STAAD-Pro and a complete analysis is carried out following the latest code of “Criteria of Earthquake Resistant Design of Structures” IS1893:2016 Part 1 (Sixth Revision).

After analysing when seen that the existing model did not comply with the clauses in the latest code i.e. IS1893:2016 Part 1 (Sixth Revision), structural failures are observed. Then different retrofitting techniques shall be applied in the building and analysed for various parameters Base Shear, Time Period, Lateral Displacement, Storey Drift, Bending Moment and Shear Force. This study gives an overview of which retrofitting procedure would be better if applied to an existing building which is built by following provisions of the previous revision of seismic code i.e. IS1893:2002.

3.1 Design Procedure

With the help of Design Basis Report of the existing building, a model in Staad was prepared and analysed. The following steps were followed for modelling:

1. Geometry: Started by creating a 3D model of the building geometry in STAAD.Pro by following the architectural drawing details. All the structural elements such as beams, columns, and any other components relevant to the building design were created as per the architectural plans shown in Fig.3.1 and for elevation the data given in Table-3.1 was followed. The model of the existing building created can be seen in Fig.3.2

Table 3.1: Elevation Detail

S. No.	Description	Detail
1	Total height of building	23.9 m
2	Height of each storey	3.2 m
3	No. of Storeys	S+6
4	Depth of foundation	1.5 m

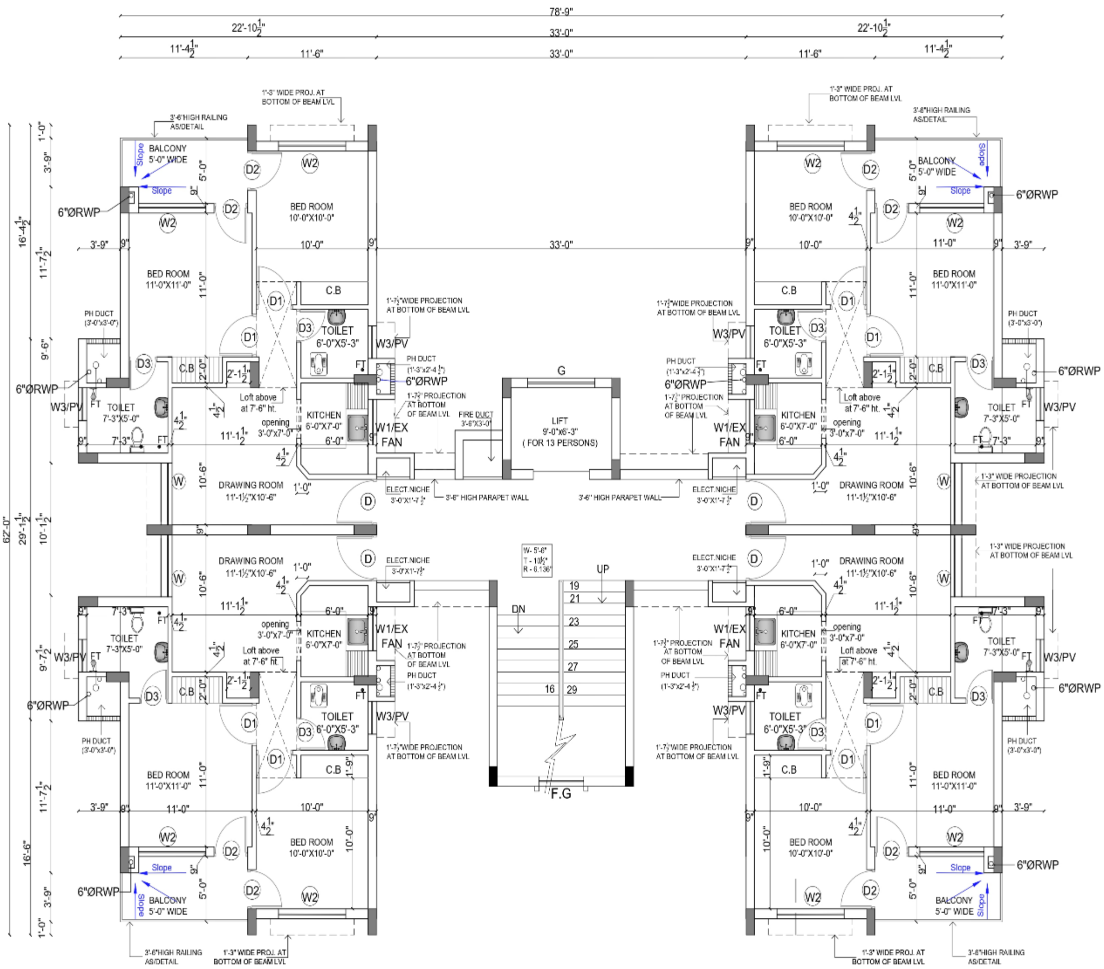


Fig. 3.1: Architectural Plan of the existing building

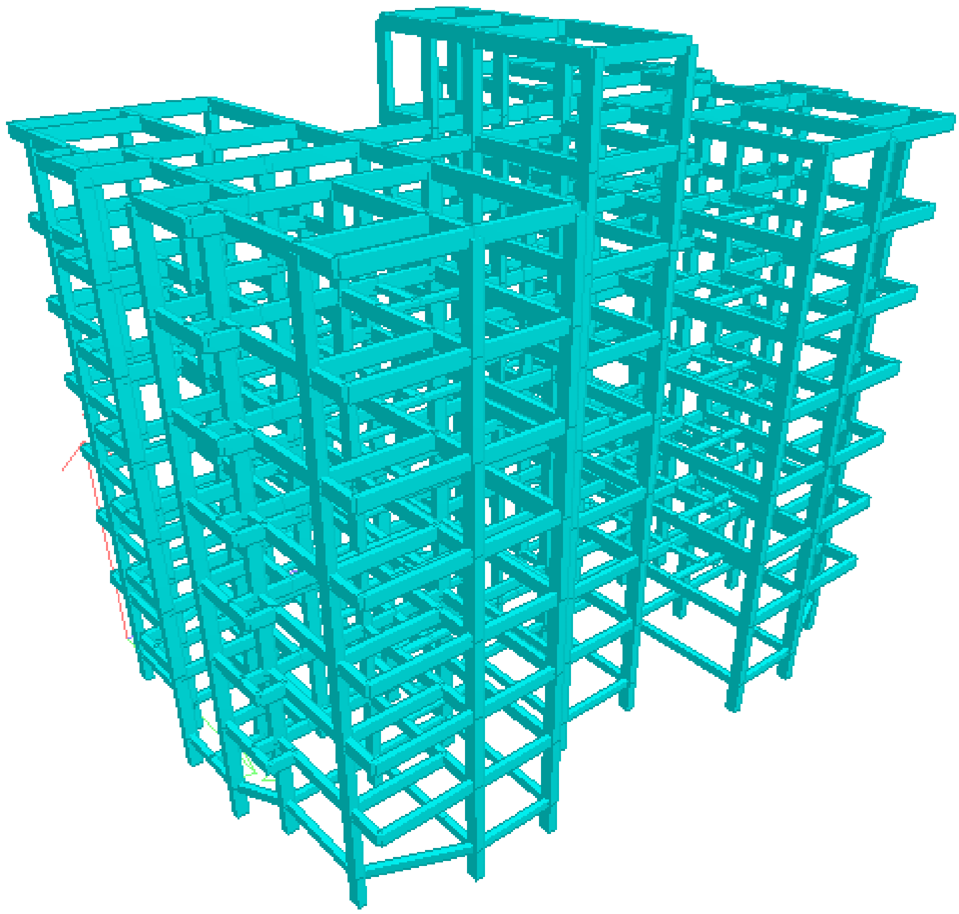


Fig. 3.2: 3D Model in StaadPro

2. Section Properties and Material Properties: After defining the geometry, appropriate properties to the structural elements were assigned as per the structural drawings. This includes specifying the section properties for beams, columns, such as their dimensions, moments of inertia, and other relevant characteristics. Material properties such as grade of Concrete and Steel, their modulus of elasticity, density, and any applicable design codes or standards were assigned according to Table 3.2 and as per the architectural drawings (Fig 3.1).

Table 3.2: Sectional Property Detail

S. No.	Description	Detail
1	Size of beams	225 x 500 mm 300 x 180 mm (CB)
2	Size of columns	300 x 450 mm
3	Thickness of slab	125 mm
4	Thickness of external walls	230 mm
5	Thickness of internal walls	115 mm
6	Wall Infill Material	ACC Block

Material Properties mentioned in Table 3.3 were adopted:

Table 3.3: Material Property Detail

S. No.	Description	Detail
1	Grade of Concrete	M-25
2	Grade of Steel	Fe500
3	Density of Concrete	25 kN/m3

The seismic parameters used in the existing model are mentioned in Table 3.4

Table 3.4: Seismic Parameters Detail

S. No.	Description	Detail
1	Seismic zone	IV
2	Soil condition	Medium
3	Response reduction factor	5
4	Importance factor	1
5	Type of Structure	5
6	Damping Ratio	0.05

3. Define Supports and Constraints: This involves setting the boundary conditions at the foundation or connection points of the structure, including fixed supports, pinned supports, or rollers. By defining these supports, accurate simulation of the structural behavior and load transfer mechanism can be achieved. In this study all the columns are provided with fixed support at the base.

4. Apply Loads: Various types of loads that act on the building, including dead loads, live loads, wind loads, seismic loads, and any other relevant loads specified by the design codes or project requirements are applied. STAAD.Pro provides tools to apply these loads, to input their magnitudes, directions, and distribution patterns. The loads applied on existing building were provided as mentioned in its DBR, also shown here in Fig. 3.3.

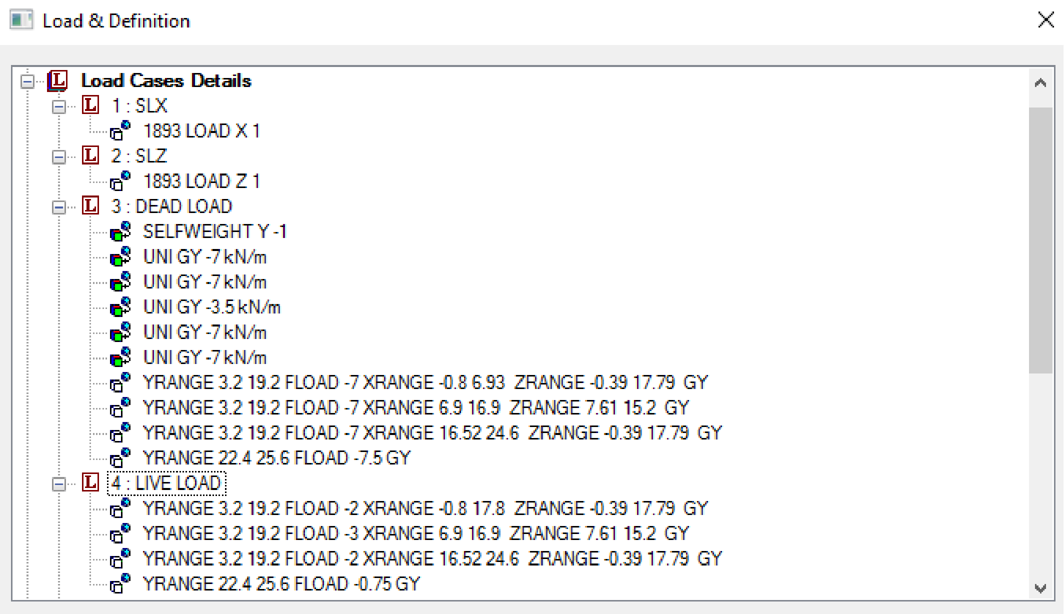


Fig. 3.3: Loading given in StaadPro

5. Run Analysis: STAAD.Pro offers different analysis options, including static, dynamic, or time history analysis. The software will calculate the internal forces and deformations within the structure based on the applied loads and structural properties. In the old model static analysis had been performed and in the revised building model dynamic Analysis by response spectrum method has been performed. In fig. 3.4, a plan is shown highlighting the column and beams which were studied in the results.

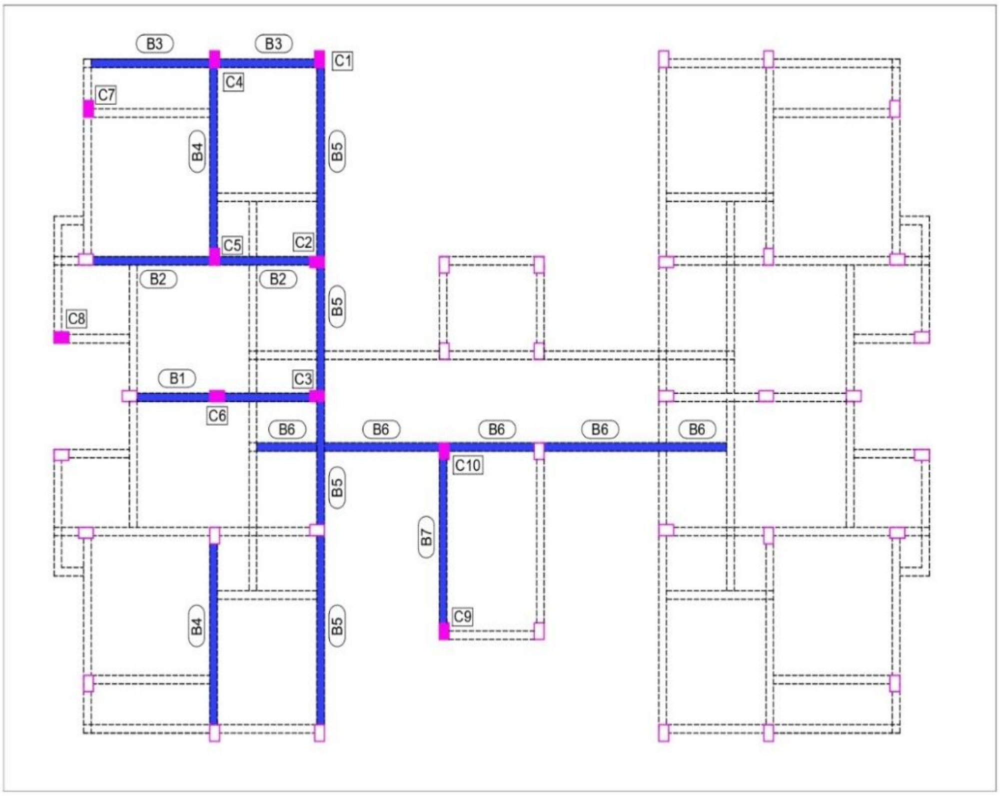


Fig. 3.4: Plan of building highlighting the members studied in the results

6. Analysis of output Results: The software provides visual representations of the structural response, including mode shapes, diagrams of bending moments, shear forces, deflections, and axial forces. Once the analysis is complete, the results were analysed for the selected members and discussed in the subsequent chapters.

3.2 Seismic Evaluation and Model Categorization

Model 1- Existing building with seismic code IS1893:2002.

Model 1 is the existing building model, no changes were made in this model. Results from this model were analysed and compared with revised models. The main difference in this model is it was analysed and designed using Seismic code IS1893:2002. (Fig 3.5)

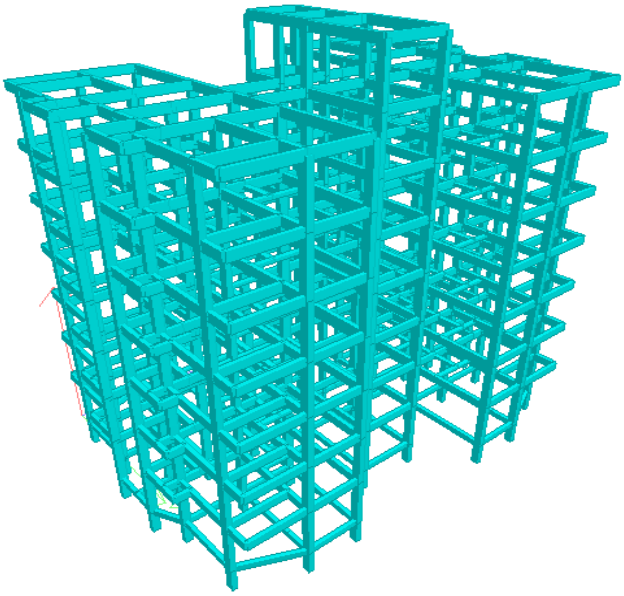


Fig. 3.5 (a) Model 1 – 3D model in StaadPro

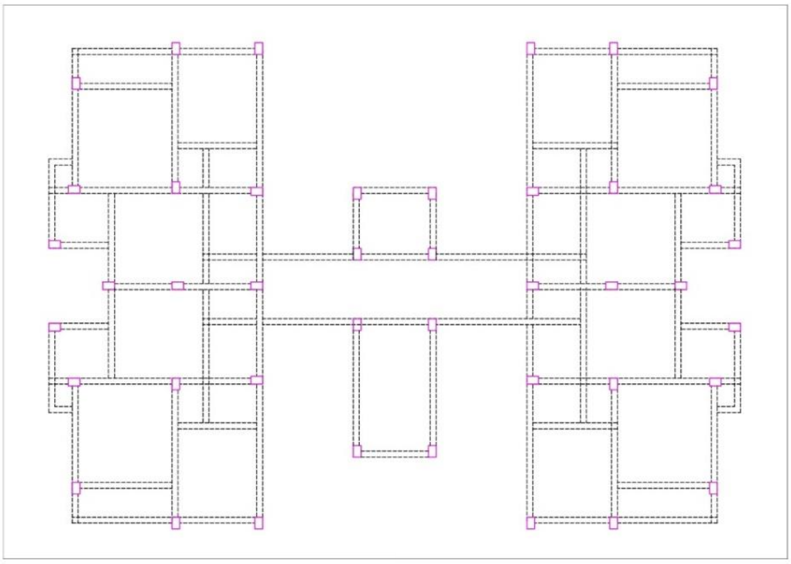


Fig. 3.5 (b) Model 1 – Plan of the building

Model 2 – Model as per Seismic Code IS1893:2016.

In this model the latest seismic code i.e. IS1893:2016 was used to analyse and design the existing building (fig.3.7). Some member failures were observed (fig.3.6) and lateral displacement was more than permissible limit of 0.004 times the height of building. After the analysis various retrofitting measures were studied and suggested. Further different models were designed with different retrofitting procedures and analysed in this study.

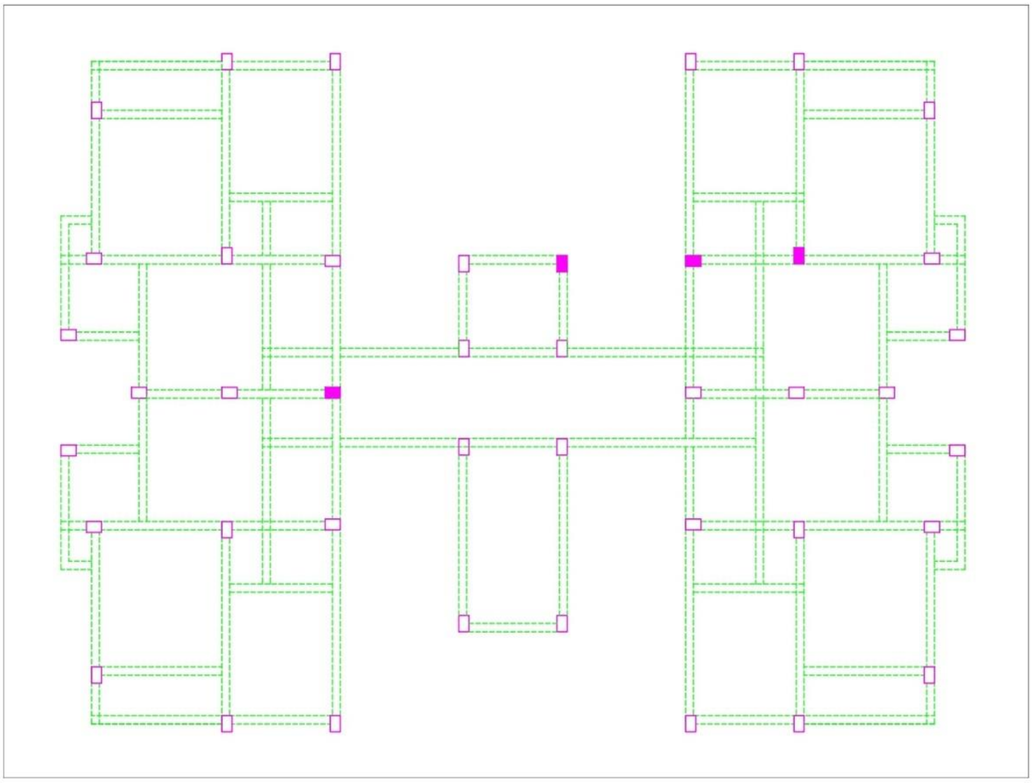


Fig.3.6 Plan showing failed members.

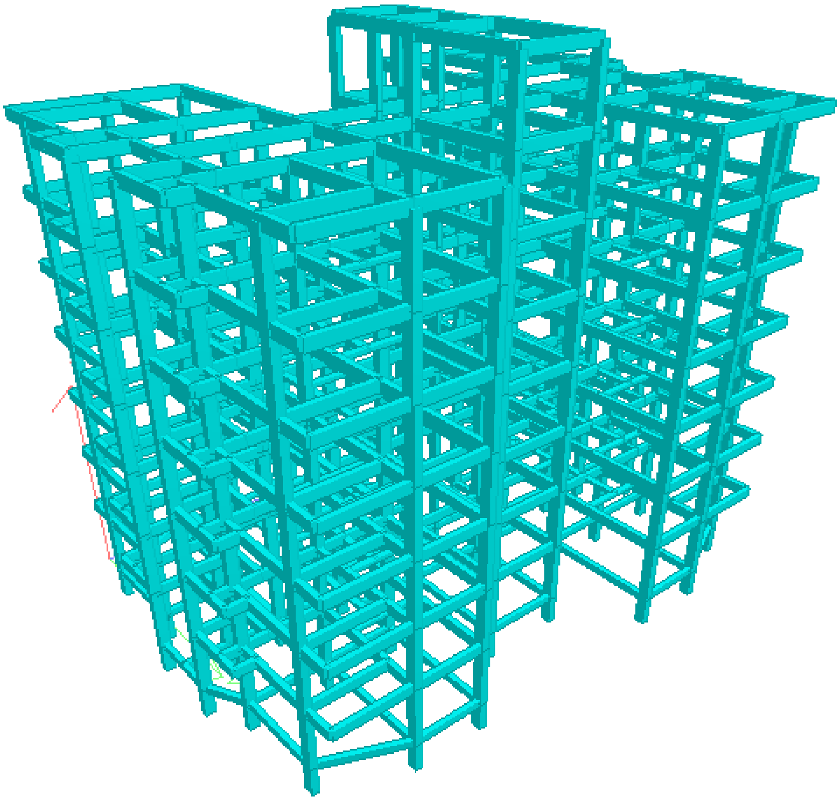


Fig. 3.7 (a) Model 2 – 3D model in StaadPro

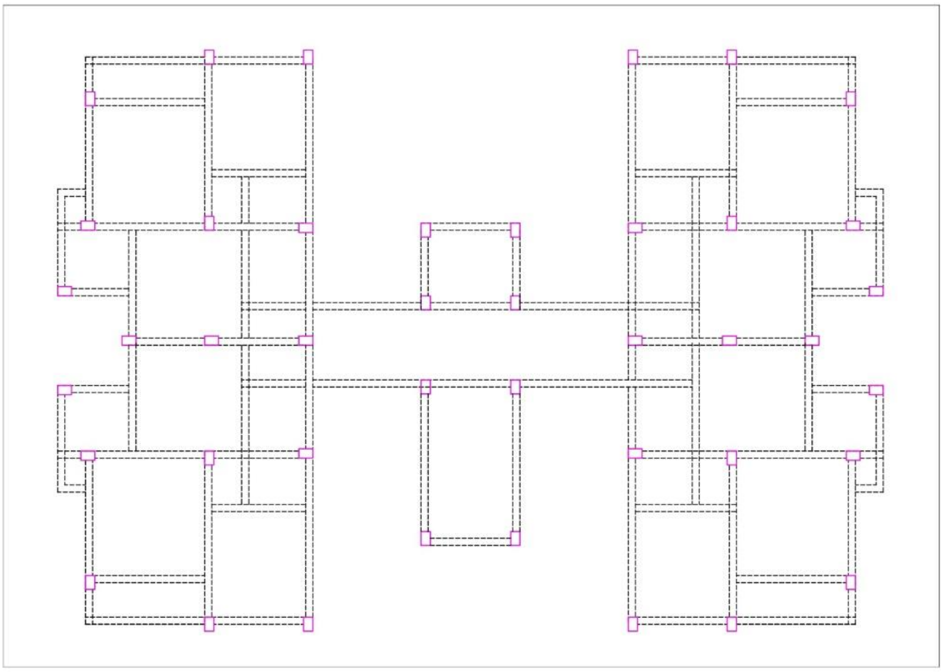


Fig. 3.7 (b) Model 2 – Plan of the building

Model 3 – Model Retrofitted with Jacketing technique

In this model, the members experiencing maximum stress and failing in designing in Model-2 were considered and accordingly different combinations for jacketing of beams and columns were designed & analyzed and the best combination was implemented (Table 3.5). Fig. 3.9 shows the location of these beams and columns in building plan. Design of jacketing of column is explained below along with typical arrangement of bars in a jacketed column (fig.3.8):

- Design of jacketed column.

1	**Existing Parameters**			
	Type of Column	Rectangle		
	Side of Column, L	450.000	mm	
	Side of Column, B	300.000	mm	
	Grade of Concrete, fck	25.000	N/mm²	
	Grade of Steel, fy	500.000	N/mm²	
	Diameter of Longitudinal Reinforcement Bar	25	16	mm
	No. of Bars	8	2	nos
	Total Area of Steel	4329.115	mm²	
	Assumed Cover to Reinforcement Bars, d'	50.000	mm	
2	**Existing Capacity of the member**			
	Gross Area of the Column	135000.000	mm²	
	Area of Steel, Ast	4329.115	mm²	
	Area of Concrete, Ac	130670.885	mm²	
	Existing axial load capacity, Pu (0.4 x fck x Ac) + (0.67 x fy x Ast)	**2756.962**	**kN**	
	Ratio of Cover to Dia of Column, d'/D	0.111		
	Percentage of Steel, pt	3.207	%	
	pt/fck	0.128		
	Pu/(fck*D²)	0.545		
	Mu/(fck*D³) Using SP 16	0.040		
	Existing Bending Moment Capacity, Mu	**91.125**	**kNm**	

3. Output Results from STAAD Model for Various Load Combinations in existing structure

Load Combinations	Fx	Fy	Fz	Mx	My	Mz	Remarks
	(kN)	(kN)	(kN)	(kNm)	(kNm)	(kNm)	
1.5 (DL + LL)	1756.62	2.42	-1.534	0	2	2	**SAFE**
1.2 (DL + LL + ELX)	1649.633	66.934	3.466	2	3	51	**SAFE**
1.2 (DL + LL - ELX)	1018.279	63.569	5.635	2	7	148	**DEFICIENT**
1.2 (DL + LL + ELZ)	1466.911	6.129	51.47	2	2	4	**SAFE**
1.2 (DL + LL - ELZ)	1201.001	2.763	53.638	2	80	10	**SAFE**
1.5 (DL + ELX)	1976.445	83.351	4.512	3	8	183	**DEFICIENT**
1.5 (DL - ELX)	1180.095	79.778	6.865	2	9	185	**DEFICIENT**
1.5 (DL + ELZ)	1740.886	7.344	64.515	3	3	5	**SAFE**
1.5 (DL - ELZ)	1408.498	3.771	66.869	3	100	12	**DEFICIENT**
0.9 DL + 1.5 ELX	1345.137	82.636	4.982	3	8	183	**DEFICIENT**
0.9 DL - 1.5 ELX	548.787	80.493	6.394	2	8	185	**DEFICIENT**
0.9 DL + 1.5 ELZ	1112.44	6.629	64.986	3	99	4	**DEFICIENT**
0.9 DL - 1.5 ELZ	777.19	4.486	66.398	3	100	12	**DEFICIENT**

4 Retrofit Design Parameter

Thickness of Jacket, t	100.000	mm
Size of Jacketed Column, D'	650.000	mm
Size of Jacketed Column, D'	500.000	mm
Grade of Concrete in the Jacket, fck'	30.000	N/mm²
Grade of Concrete used in Calculations, fck'	25.000	N/mm²
Grade of Steel, fy'	500.000	N/mm²
Grade of Steel used in Calculations, fy'	500.000	N/mm²
Cover to Reinforcement, d"	50.000	mm

5. Output Results from updated STAAD Model for Various Load Combinations with RETROFITTED SIZES

Load Combinations	Fx	Fy	Fz	Mx	My	Mz
	(kN)	(kN)	(kN)	(kNm)	(kNm)	(kNm)
1.5 (DL + LL)	2001.144	0.032	3.214	0	4	2
1.2 (DL + LL + ELZ)	1662.505	4.463	71.655	2	59	4
1.2 (DL + LL - ELZ)	1386.126	4.855	76.374	2	168	7
1.2 (DL + LL + ELX)	2069.002	120.24	5.422	5	16	303
1.2 (DL + LL - ELX)	988.96	120.63	10.141	5	18	299
1.5 (DL + ELZ)	1988.214	5.366	89.833	3	209	14
1.5 (DL - ELZ)	1631.077	6.282	95.203	3	211	9
1.5 (DL + ELX)	2484.672	150.08	7.042	7	20	379
1.5 (DL - ELX)	1134.619	151	12.411	7	22	373
0.9 DL + 1.5 ELZ	1264.355	5.549	90.907	3	209	13
0.9 DL - 1.5 ELZ	907.219	6.098	94.129	3	210	10
0.9 DL + 1.5 ELX	1760.813	150.27	-8.116	7	21	378
0.9 DL - 1.5 ELX	410.761	150.82	11.337	7	22	375

6 Check for axial load capacity of the retrofitted column

Existing Axial load capacity	2756.962	kN
Maximum axial load as per above Table	2484.672	kN
Area of column required	186117.753	mm²
Area of column provided	325000.000	mm²
		OK

7 Check for longitudinal reinforcement of the column

Area of steel required as per STAAD	6000.000	mm²	
Equivalent Area of steel in the existing core	4329.115	mm²	
Balance area of steel required in the jacket portion	1670.885	mm²	
Min. area of steel to be provided in the jacket as per IS 15988:2013	2227.847	mm²	
Dia of bar to be provided in the jacket	20.000	mm	
No. of Bars required	7.095	nos	
PROVIDED NO. OF BARS	**10.000**	**nos**	OK

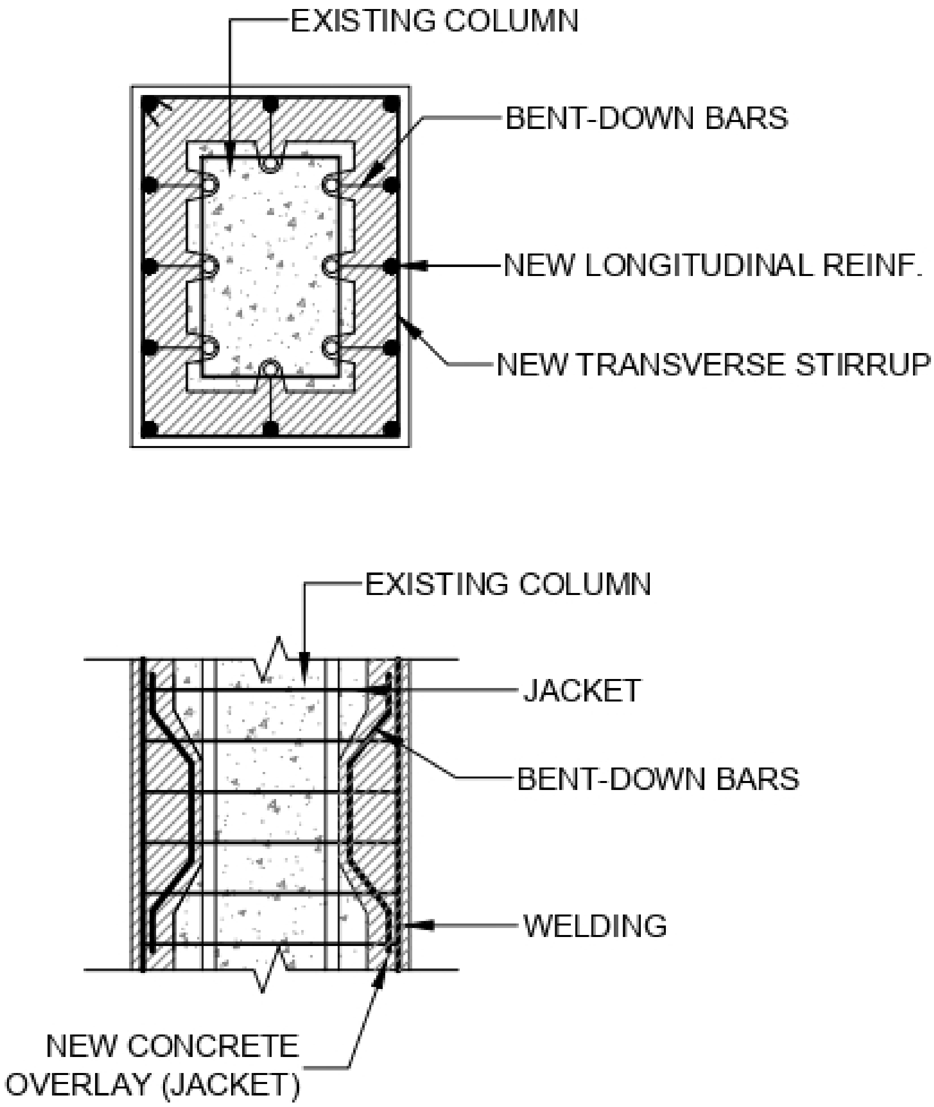


Fig.3.8 Typical Jacketing Detail (Ref. IS 15988:2013)

Table 3.5: Jacketing Detail			
S.No.	**Component**	**Old Size (mm)**	**New Size (mm)**
1	Beam B1, B2, B4	225 x 500	375 x 575
2	Column C2	450 x 300	650 x 500
3	Column C5	300 x 450	500 x 650

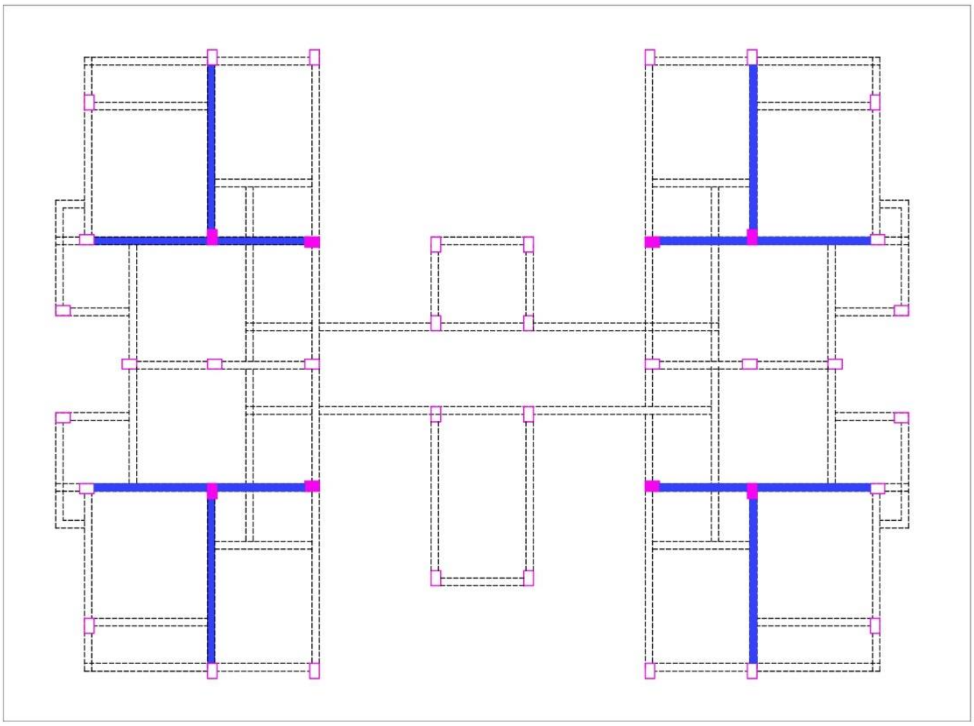


Fig. 3.9: Model 3 – Plan highlighting the columns and beams jacketed

Model 4 – Retrofitted Model with Shear Wall as per Seismic Code IS1893:2016.

In this model, the members experiencing maximum stress and failing in design in Model-2 were considered and accordingly shear wall was designed and different configuration as per the requirement were analyzed. Fig 3.11 below shows the location of shear wall which was adopted in Model-4. Below is design of shear wall between C6 & C3 columns and fig.3.10 gives a typical detail of layout of reinforcement in a shear wall when inserted between existing columns.

- **Design of Shear Wall**

Factored Axial Load	Pu	=	1154	kN
Design moment @ axis 3-3	M_{33}	=	15	kNm
Design moment @ axis 2-2	M_{22}	=	76	kNm
Length of Shear Wall	L_w	=	3230	mm
Thickness	t_w	=	200	mm
	h_w	=	3300	mm
Grade of Concrete	f_{ck}	=	20	N/mm^2
Grade of Steel	f_y	=	500	N/mm^2
Elastic Modulus of Steel	E_s	=	200000	N/mm^2
Clear Cover		=	40	mm
Stirrup size		=	8	mm
Area of Steel	A_{st}	=	2435.363	mm^2

12 mm dia bar @ 150 mm c/c on both face

No. of bars	=	26
Reinf. ratio , p=Ast/(tw*Lw)	=	0.003769911

Calculation for Moment Capacity of the wall
As per IS 13920-2016 Annex-A

λ = Pu/(fck*tw*Lw)
0.08932

φ = 0.87 fy p/fck
0.082

β = [(0.002+0.87fy/Es)]/0.0035
1.19286

$Xu/Lw = (\varphi+\lambda)/(2\varphi+0.36)$
0.32694

$Xu^*/Lw = 0.0035/[(0.0035+(0.002+0.87fy/Es)]$
0.45603

CASE 1

Now , we have Xu/Lw 0.32694 < Xu*/Lw 0.45603

(a) For Xu/Lw < Xu*/Lw

Moment Capacity @ Axis 3-3

$M_u/f_{ck}t_wL_w^2 = \varphi[(1+\lambda/\varphi)(1/2-0.416Xu/Lw)-(Xu/Lw)2*(0.168+\beta2/3)]$
0.056727642

$M_{u3\text{-}3c}$ = Moment*fck*tw*Lw2
2367.335 kNm

CASE 2

(b) For Xu*/Lw < Xu/Lw < 1 *(As per IS 13920:2016 Annex-A)*

$M_u/f_{ck}t_wL_w^2 = \alpha_1(x_o/L_w) - \alpha_2((x_o/L_w)\text{^}2)-\alpha_3-\lambda/2$

Where, $\alpha_1 = [0.36+\varphi(1-\beta/2-1/2\beta)]$
0.35872

$\alpha_2 = [0.15+\varphi/2*(1-\beta+\beta2/3-1/3\beta)]$
0.15008

$\alpha_3 = \varphi/6\beta((1/Xu/Lw)-3))$
0.00067

$\alpha_4 = (\varphi/\beta-\lambda)$
-0.0206

$\alpha_5 = (\varphi/2\beta)$
0.0489

Now, $\alpha_1X^2+\alpha_4X-\alpha_5=0$ Where, X=Xu/Lw

$$F = \frac{-\alpha_4 \pm \sqrt{[\alpha_4^2- 4\alpha_1(-\alpha_5)]}}{2\alpha_1}$$

= 0.39903 and -0.3417

i.e. Xu/Lw = 0.39903 and -0.3417

Again	$M_u/f_{ck}t_wL_w^2$	=	$\alpha_1(x_o/L_w)$ - $\alpha_2((x_o/L_w)$^2)-α_3-λ/2	
		=	0.07391	-0.185409759
So,	$M_u/f_{ck}t_wL_w^2$	=	0.07391	
	M_u	=	**3084.45**	kNm

OURVALID CASE is (a) For Xu/Lw < Xu*/Lw

So, the Moment Capacity @ Axis 3-3 = 2367.335 kNm

Moment Capacity @ Axis 2-2

	Pt/fck	=	0.01885			
	Pu/fckbD	=	0.08932	Where, b=	3230	mm
				D=	200	mm
	d'	=	56	mm		
	D	=	200			
	d'/D	=	0.28			
Adopt	d'/D	=	0.2			

From the Chart of SP-16 for Pu/fckbD = 0.033 and Pt/fck = 0.03

$Mu/fckbD^2$ = 0.04

Mu2-2c = 0.06*fckbD2

1E+08 Nmm

103.36 kNm

As per IS 456:2000 Clause 39.6

It should be $\left(\frac{Mux}{Mux1}\right)^{\alpha n} + \left(\frac{Muy}{Muy1}\right)^{\alpha n} \leq 1$

Puz = 0.45fck*Ac + 0.75 fy*Asc

6705343 kN

Pu/Puz = 0.001

Hence, αn = 1

0.00634 + 0.735294118 = 0.74163036 ≤ 1

OK

Therefore provide 12 mm dia bar @ 150 mm c/c on both face

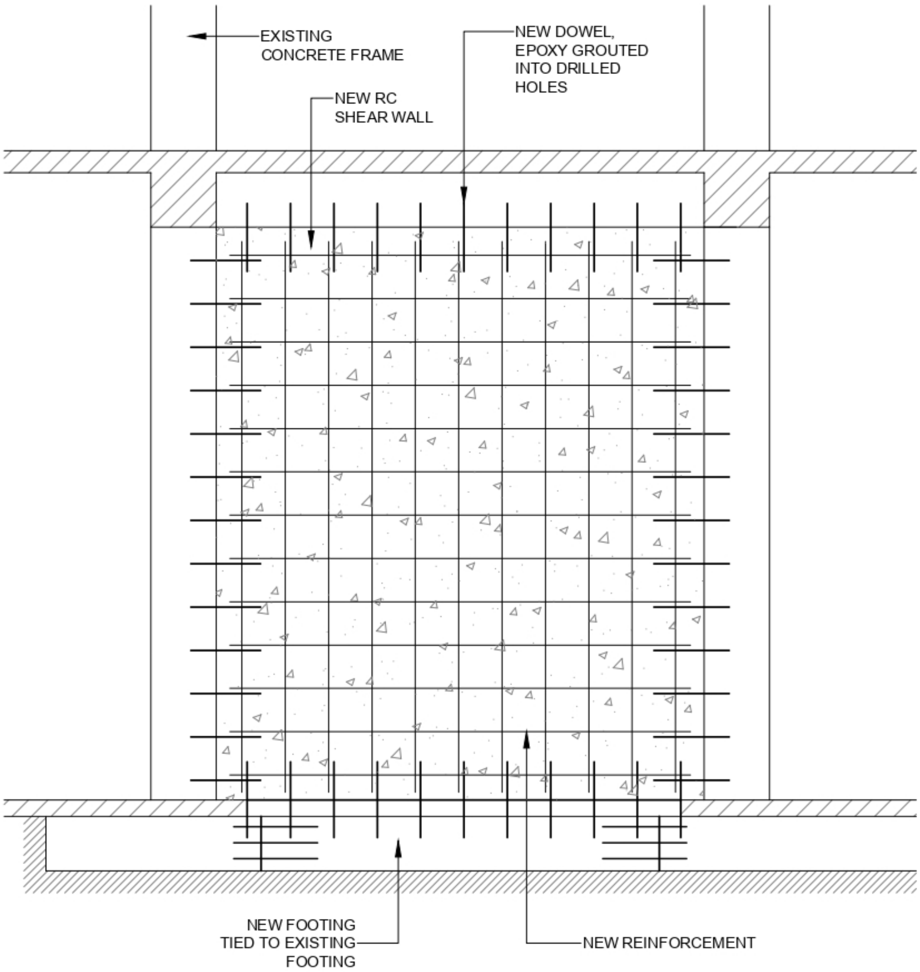


Fig. 3.10 Shear Wall Re-inforcement detail (Ref. IS 15988:2013)

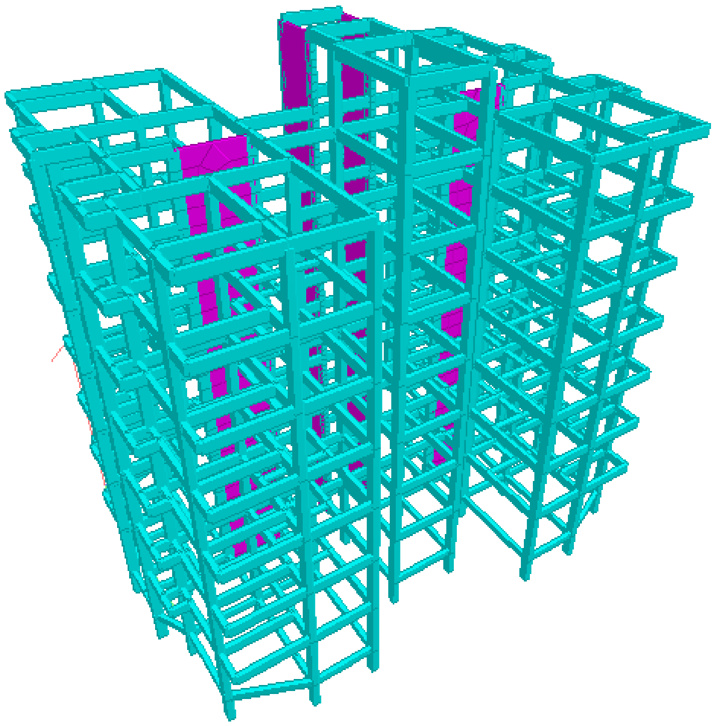


Fig. 3.11(a): Model 4 – 3D model in StaadPro showing location of Shear Wall

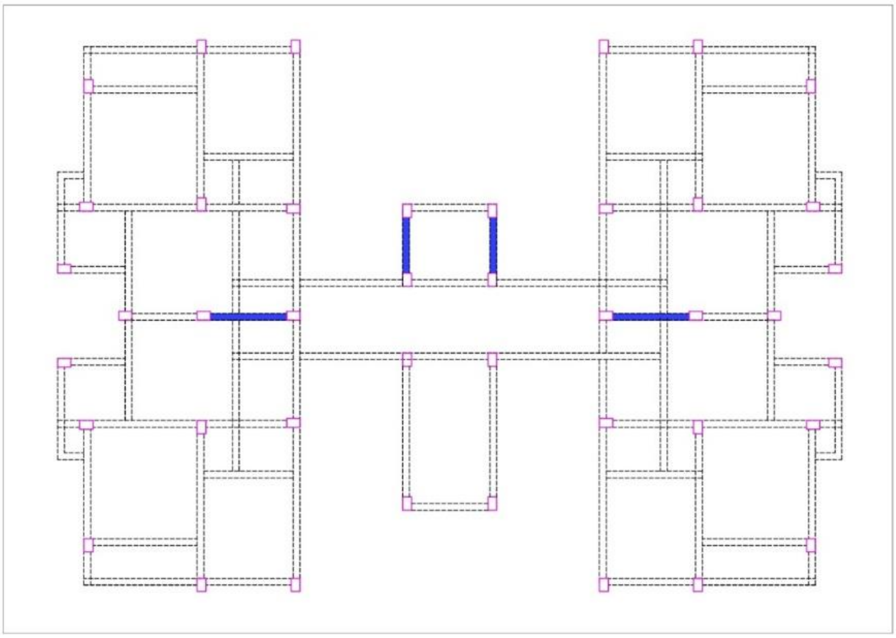


Fig. 3.11(b): Model 4 – Plan highlighting the location of Shear Wall

Model 5 – Retrofitted Model with Bracing with Seismic Code IS1893:2016

In this model, the members experiencing maximum stress and failing in design in Model-2 were considered and accordingly steel bracing was designed for different configuration and best option was provided between different columns. (Fig. 3.14) From base to 3rd floor ISA 150x115x10 back-to-back is provided and from 3rd floor to 6th floor ISA 150x75x12 back-to-back is provided. Typical section of a back to back bracing is shown in fig.3.12 along with connection detail in fig.3.13.As per the requirement the design of the bracings is shown below:

- Bracing of ISA 150x115x10 back-to-back provided from base till 3rd floor.

Intensity of load acting on the bracing	=	406	kN (C)
Height of storey	=	3.3	m
Bay Area	=	3.23	m
Length of Bracing	=	4.62	m

Assuming ISA section (150x115x10),

A	=	25.52	cm^2	
		2552	mm^2	
Ixx	=	573.3*2		
		1146.60	cm^4	
Iyy	=	(293.4+25.52*2.82*2.82) *2		
		992.69	cm^4	
r	=	$\sqrt{I/A}$		
		4.41	cm	
l/r	=	104.706		

Corresponding to the Kl/r value calculated above,

Stress reduction factor from table 8 in IS 800:2007

χ	=	0.45		
0.45*250	=	112.5	N/mm^2	
A_{req}	=	3608.889	mm^2	
A_{prov}	=	5104	mm^2	OK

- Bracing of ISA 150x75x12 back-to-back provided from 3rd to 6th floor.

Intensity of load acting on the bracing		=	200	kN
Height of storey		=	3.3	m
Bay Area		=	3.23	m
Length of Bracing		=	4.62	m
Assuming ISA section				
(150x75x12)				
	A	=	30.38	cm^2
			3038	mm^2
	Ixx	=	587.2*2	
			1174.40	cm^4
	Iyy	=	(99.5+30.38*1.69*1.69)*2	
			372.54	cm^4
	r	=	√I/A	
			2.48	cm
	l/r	=	186.4866	

Corresponding to the Kl/r value calculated above,
Stress reduction factor from Table 8 in IS 800:2007

χ	=	0.205		
0.205*250	=	51.25	N/mm^2	
A_{req}	=	3902.439	mm^2	
A_{prov}	=	6076	mm^2	OK

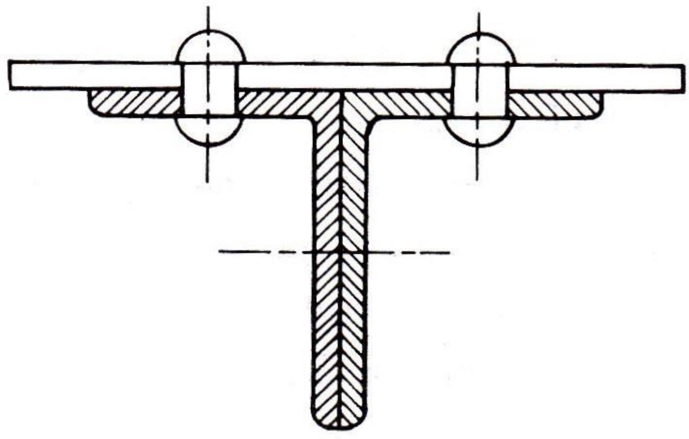


Fig.3.12 Typical Angle section back-to-back

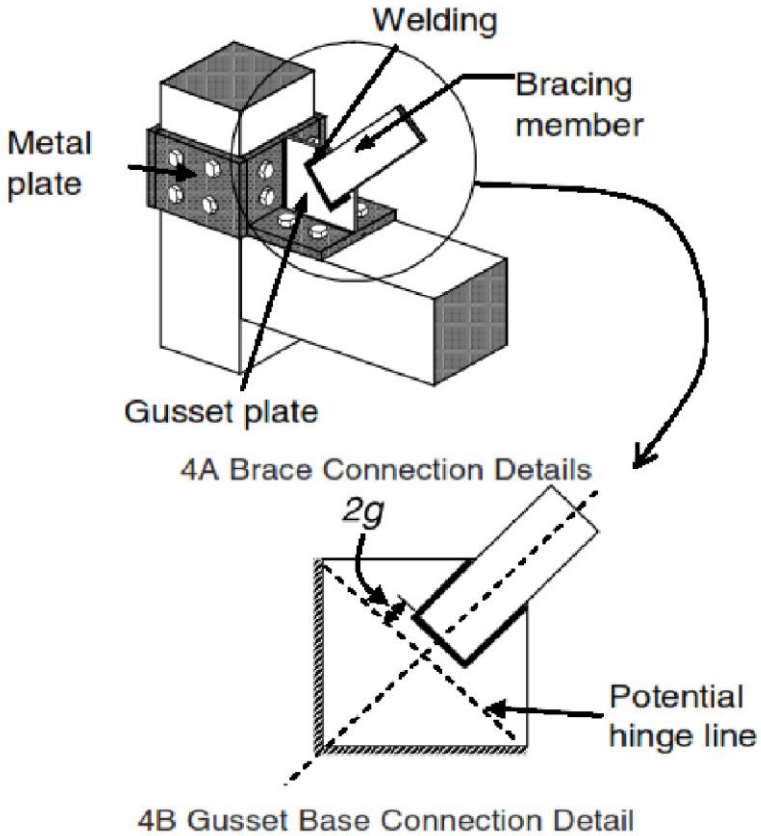


Fig.3.13 Brace Connection Detail (Ref: IS 15988:2013)

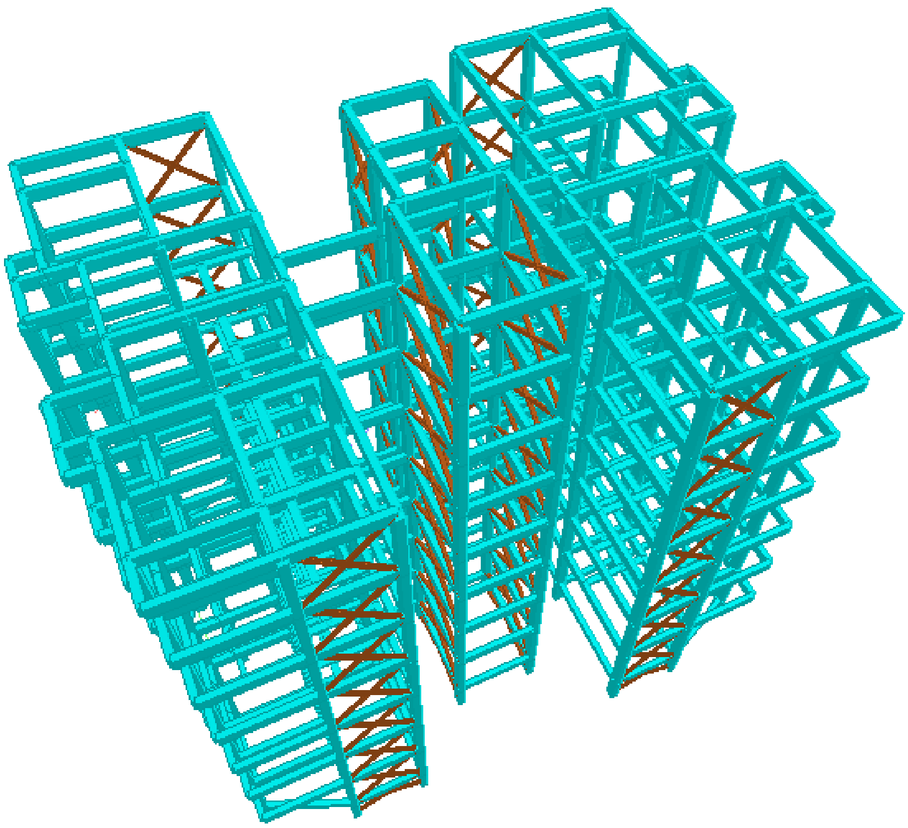


Fig. 3.14(a): Model 5 – 3D model in StaadPro showing Bracing locations

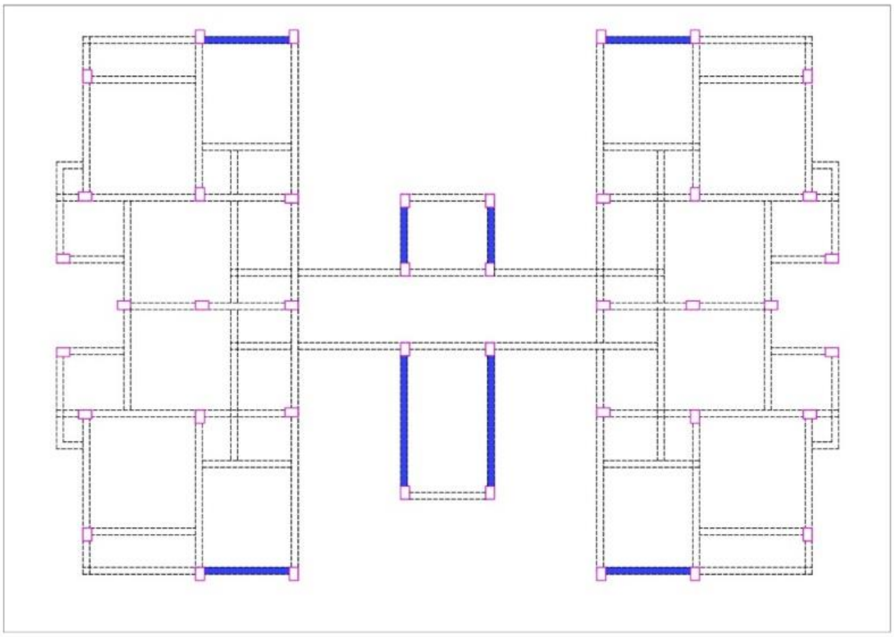


Fig. 3.14(b): Model 5 – Plan highlighting the location of Steel Bracing.

A brief description of the models, their nomenclature and color code used in graphs further in this study is summarized in Table 3.6.

Table 3.6: Details of the Models

S.No.	Model Name	Description	Color Code
1.	Model-1/ IS1893:2002	Original Model of the existing building, with seismic code IS1893:2002	
2.	Model-2/ IS1893:2016	Model of the existing Building, with latest Seismic Code IS1893:2016	
3.	Model-3/ Jacketing	2016 model retrofitted by providing jacketing.	
4.	Model-4/ Shear Wall	2016 model retrofitted by providing ShearWall.	
5.	Model-5/ Bracing	2016 model retrofitted by providing Bracing.	

4. PARAMETRIC ANALYSIS AND RESULTS

4.1 TIME PERIOD (in Dynamic Analysis)

It refers to the characteristic period of vibration of a structure, which is the time taken for the structure to complete one full cycle of oscillation. The time period of a structure depends on various factors, including its mass, stiffness, and the distribution of mass and stiffness throughout the structure. Lighter and stiffer structures tend to have shorter time periods, while heavier and more flexible structures have longer time periods.

Determining the time period of a structure is important as it influences its response to dynamic loads. During an earthquake, structures with longer time periods tend to experience larger displacements and higher internal forces, while structures with shorter time periods tend to have smaller displacements and lower internal forces.

In structural analysis, the natural frequencies of a structure are calculated, which are related to the time-period, to assess its dynamic response. By understanding the time-period and natural frequencies, one can evaluate the potential for resonance phenomena, where the dynamic forces coincide with the natural frequency of the structure, leading to amplified responses and potential structural failure. As the dynamic response was studied, Model 1 which was designed with Static Analysis does not give any value. The results obtained are shown in Table 4.1 and Fig 4.1.

Table 4.1 (a) - Time period (sec)

IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
Not calculated in static analysis	2.35	2.19	2.17	1.99

Table 4.1 (b) - Percentage change in Time Period w.r.t. IS1893:2016 Model

Jacketing	Shear Wall	Bracing
-6.92 %	-7.77 %	-15.05 %

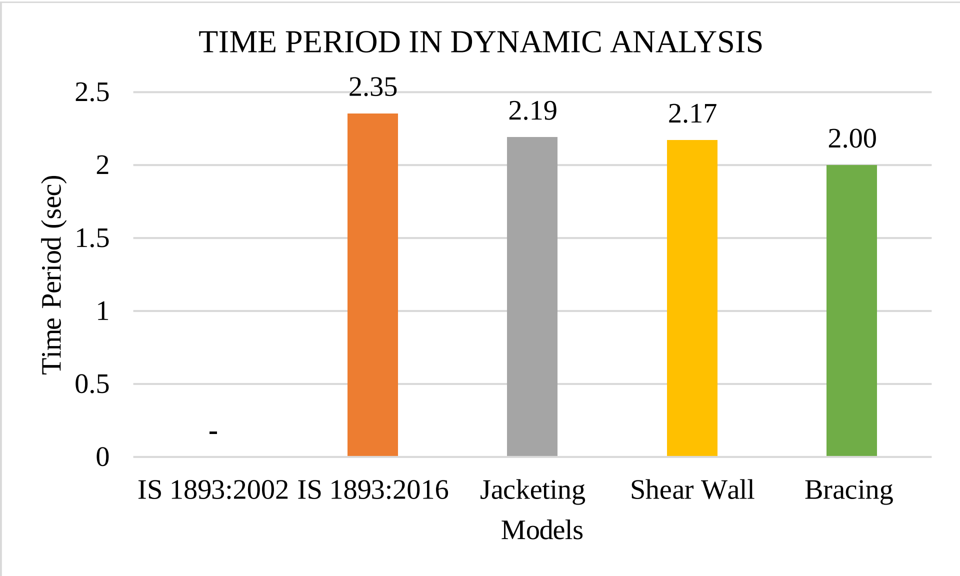


Fig. 4.1 - Time Period in Dynamic Analysis

The result obtained for Model-1 are not calculated because static analysis was performed in it. Percentage decrease in values of time period can be seen in table 4.1(b). Reduction in Time Period was seen in all the models, Model-5 with bracings has the least time period, it had 15 % reduction compared to Model-2.

4.2 BASE SHEAR

In RCC (reinforced concrete) frame structures, base shear is an important parameter used to determine the seismic forces that the building must be designed to resist. During an earthquake, the ground moves horizontally and the structure experiences significant lateral forces. These forces cause the building to sway and deform, potentially leading to collapse. Base shear is the total lateral force that the foundation of the building must resist during an earthquake.

Base shear is a critical design parameter because it determines the size and strength of the foundation and the lateral load-resisting system of the building. The magnitude of base shear is influenced by various factors such as the building height, weight, stiffness, and the properties of the soil at the foundation level.

According to IS 1893:2016, the base shear (V) can be determined using the following equation:

$V = Ah \times Wi$

where: V represents the base shear,

Ah is the design horizontal peak ground acceleration coefficient, and

Wi is the seismic weight of the structure.

The design horizontal peak ground acceleration coefficient (Ah) is determined based on the site seismicity, soil conditions, and the response spectrum. The seismic weight (Wi) considers the mass of the structure and its dynamic characteristics.
In the study we have taken the following factors in our model (table 4.2)

Table 4.2: Seismic Parameters

S.No.	Model Name	Zone Factor	Response Reduction Factor	Importance Factor	Type of Soil	Type of Structure
1.	Model-1/ IS1893:2002	IV	5	1	Medium	3*
2.	Model-2/ IS1893:2016	IV	5	1	Medium	5
3.	Model-3/ Jacketing	IV	5	1	Medium	5
4.	Model-4/ Shear Wall	IV	5	1	Medium	5
5.	Model-5/ Bracing	IV	5	1	Medium	5

(ST-3 in IS1893:2002 is same as ST-5 in IS1893:2016)*

In this study, we first discussed the dynamic base shear in Table 4.3,4.6 and Fig.4.2 & Fig.4.5, then the multiplying factor (Vb/VB) in Table 4.4,4.7 and Fig.4.3 & Fig.4.6, and then the Design Base Shear in Table 4.5,4.8 and Fig 4.4 & Fig.4.7.

➢ In X-Direction:

Table 4.3(a): Dynamic Base Shear (kN) in X-direction

IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
Not calculated in Static Analysis	489.58	568.62	644.39	570.16

Table 4.3(b) - Percentage change in Dynamic Base shear w.r.t. IS1893:2016 model

Jacketing	Shear Wall	Bracing
+16.14 %	+31.62 %	+16.46 %

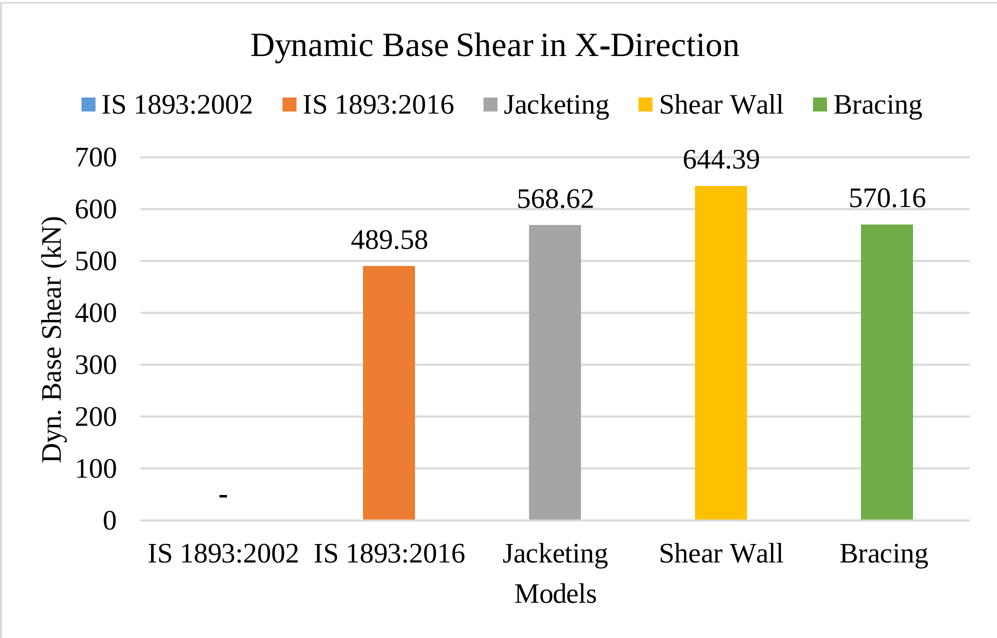


Fig.4.2: Dynamic Base Shear in X-Direction

Table 4.4(a) - Multiplying factor Vb/VB in X-Direction

IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
Not calculated in Static Analysis	3.86	3.43	3.07	3.35

Table 4.4(b) - Percentage change in multiplying factor w.r.t. IS1893:2016 model

Jacketing	Shear Wall	Bracing
-11.22 %	-20.44 %	-13.32 %

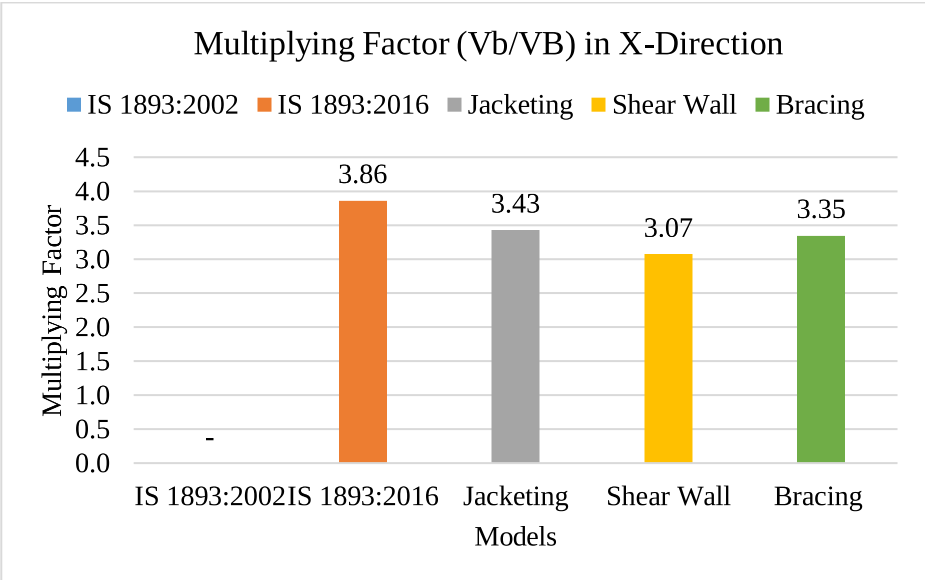


Fig.4.3: Multiplying Factor (Vb/VB) in X-Direction

Table 4.5 – Design Base Shear (kN) in X-Direction

IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
1890.28	1890.28	1949.68	1979.78	1908.21

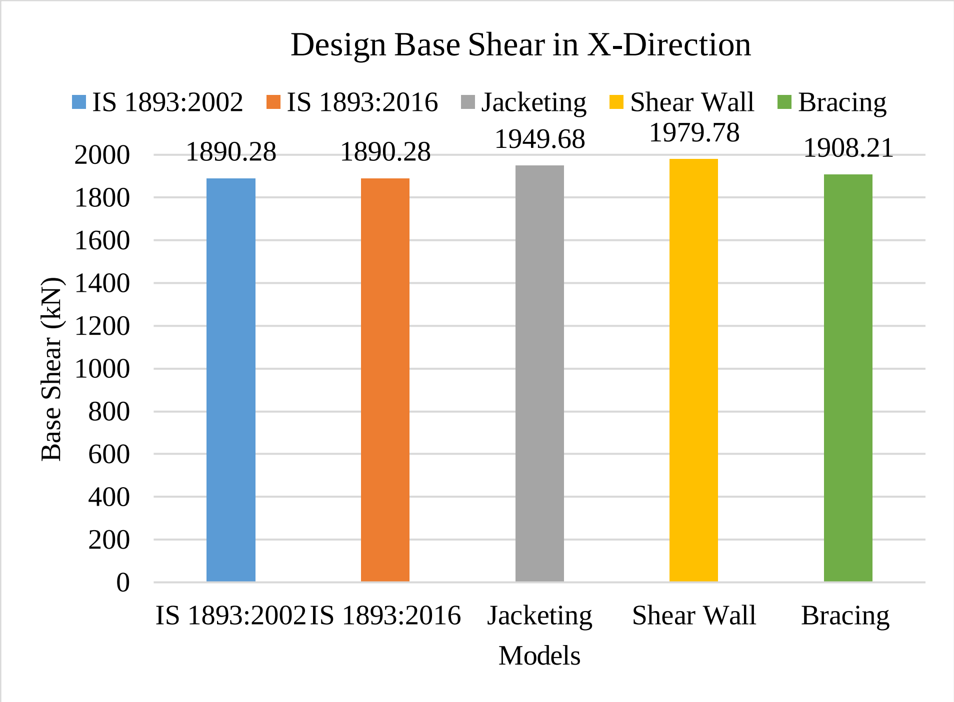


Fig.4.4: Design Base Shear in X-Direction

➢ In Z-Direction :

Table 4.6(a) – Dynamic Base Shear (kN) in Z-Direction

IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
Not calculated in Static Analysis	449.07	496.71	499.08	520.38

Table 4.6 (b) - Percentage change in Dynamic Base shear w.r.t. IS1893:2016 model

Jacketing	Shear Wall	Bracing
+10.61 %	+11.14 %	+15.88 %

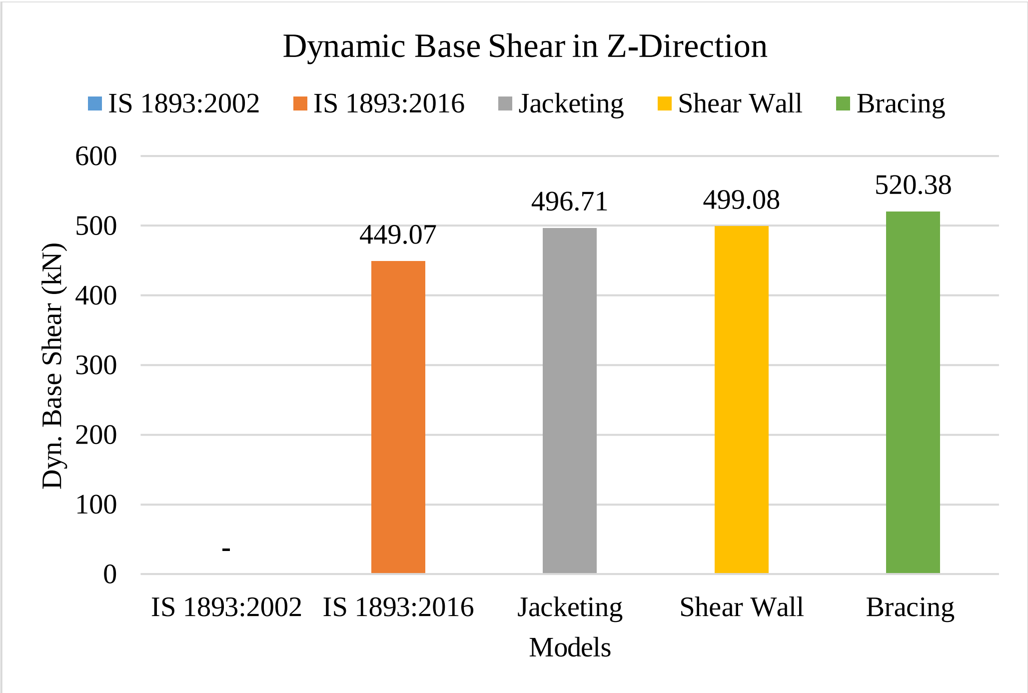


Fig.4.5: Dynamic Base Shear in Z-Direction

Table 4.7(a) - Multiplying factor Vb/VB in Z-Direction

IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
Not calculated in Static Analysis	4.00	3.73	3.77	3.49

Table 4.7(b) - Percentage change in Multiplying factor w.r.t. IS1893:2016 model

Jacketing	Shear Wall	Bracing
-6.75 %	-5.82 %	-12.89 %

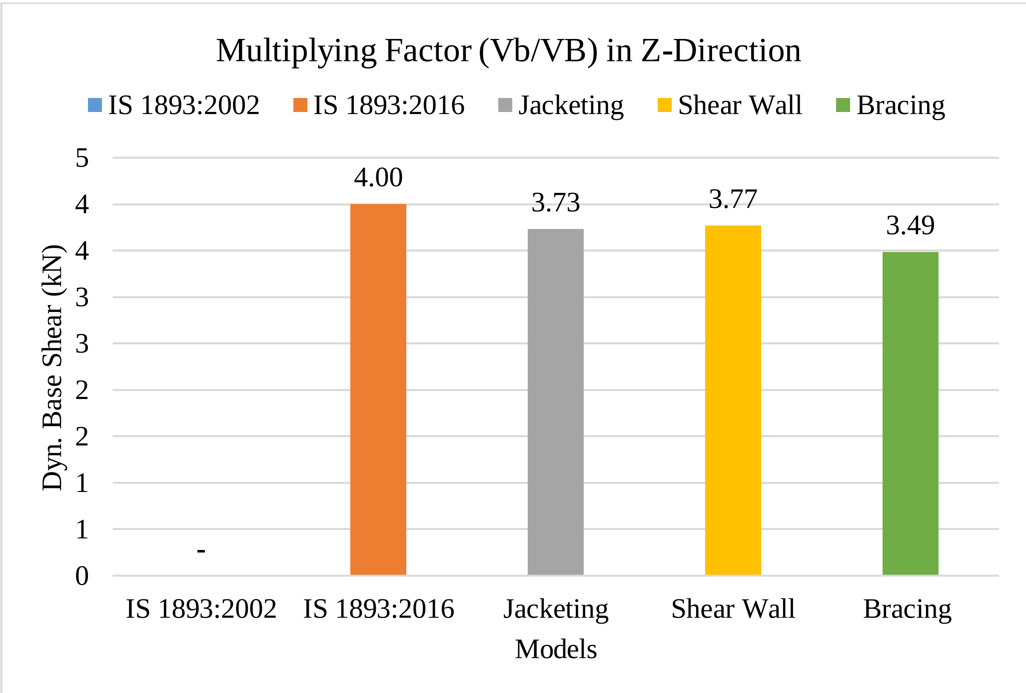


Fig.4.6 Multiplying Factor (Vb/VB) in Z-Direction

Table 4.8 – Design Base Shear (kN) in Z-Direction

IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
1796.58	1796.58	1853.03	1881.64	1813.61

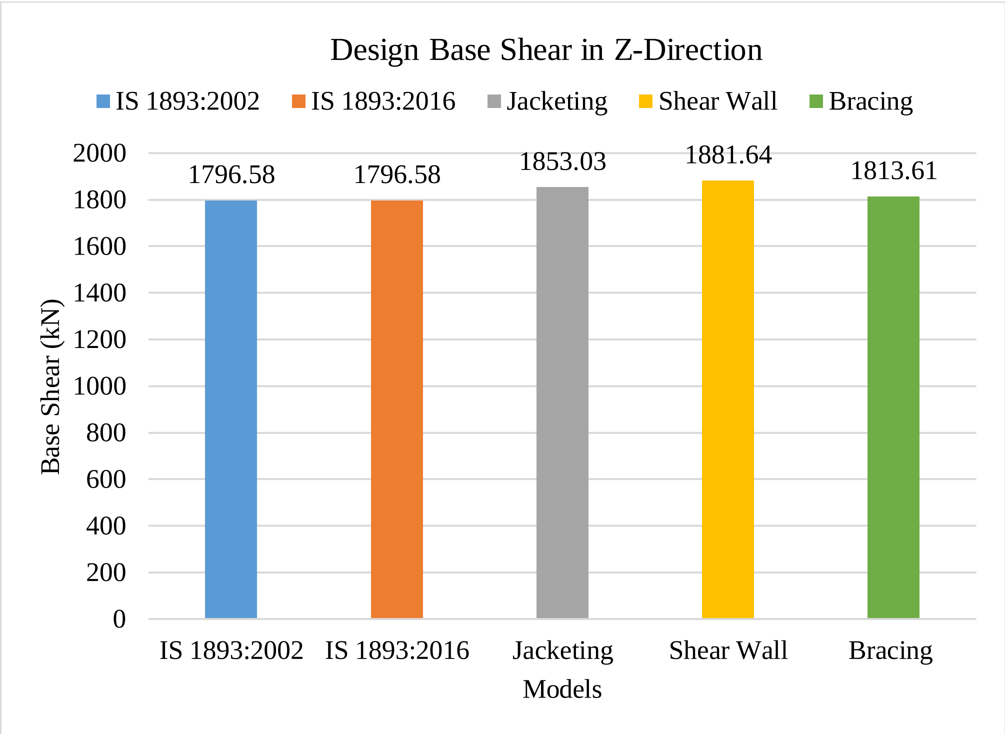


Fig.4.7: Design Base Shear in Z-Direction

In Model-1 the values for Dynamic Base Shear and Multiplying Factor Vb/VB are zero because static analysis was performed in it. From above analysis, it is observed that Design Shear increase in the retrofitted models by 2-5 %. Increase in seismic weight of the building is one of the contributing factors.

4.3 LATERAL DISPLACEMENT

Lateral displacement is the horizontal displacement experienced by a building or structure under lateral loads. These loads can arise from various sources, such as earthquakes or dynamic loads. Lateral displacement is typically measured at specific points or nodes within the structure, such as the roof or the building's base. Analyzing lateral displacement is crucial for designing safe and resilient structures. Excessive lateral displacement can lead to structural damage, deformation, and even collapse, posing a threat to the integrity of the building. It can also result in non-structural damage, such as damage to partitions, cladding, or utility systems, impacting the functionality and usability of the structure. To mitigate excessive lateral displacement, various design strategies can be employed. These may include incorporating robust lateral load resisting systems, such as shear walls, bracing systems, or moment-resisting frames. These systems are designed to efficiently distribute and dissipate lateral forces, minimizing the overall displacement of the structure. In dynamic analysis as per IS1893:2016 the lateral displacement increases significantly as cracked sections are considered in this analysis and reduction in moment of inertia of members is done for Structural Analysis (30% for columns and 65% for beams). In this study dynamic analysis are performed on all the models except model 1 and the obtained results for Lateral Displacement in X and Z-Direction for the selected columns (fig.4.8) are discussed below from Table 4.9 – 4.22 and Fig. 4.9 – 4.22:

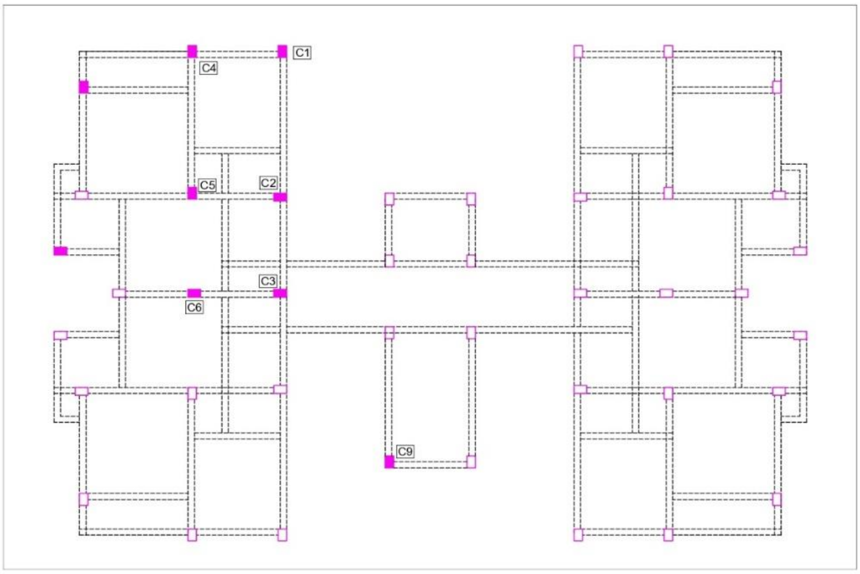


Fig.4.8 Plan highlighting the columns studied

➢ **LATERAL DISPLACMENT IN X-DIRECTION** (Load Combination LC 1: SLX)

Table 4.9(a) - Lateral Displacement (mm) in X-Direction in column C-1

Node No.	Ht (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
28	0	0	0	0	0	0
268	1.50	1.693	3.483	2.953	2.063	0.554
272	4.70	11.307	22.270	18.849	13.351	2.915
276	7.90	20.782	39.994	33.600	23.757	6.529
280	11.10	30.052	55.769	46.547	32.943	10.976
284	14.30	38.632	69.271	57.568	41.130	15.895
288	17.50	45.896	80.339	66.641	48.525	20.999
292	20.70	51.120	88.545	73.459	54.753	26.088
296	23.90	53.846	93.444	77.588	58.935	31.061

Table 4.9(b) - Percentage change in Lateral Displacement w.r.t. IS1893:2016 model in Column C-1

Node No.	Ht (m)	Jacketing (%)	Shear Wall (%)	Bracing (%)
28	0	0	0	0
268	1.50	-15.22	-40.77	-84.09
272	4.70	-15.36	-40.05	-86.91
276	7.90	-15.99	-40.60	-83.68
280	11.10	-16.54	-40.93	-80.32
284	14.30	-16.89	-40.62	-77.05
288	17.50	-17.05	-39.60	-73.86
292	20.70	-17.04	-38.16	-70.54
296	23.90	-16.97	-36.93	-66.76

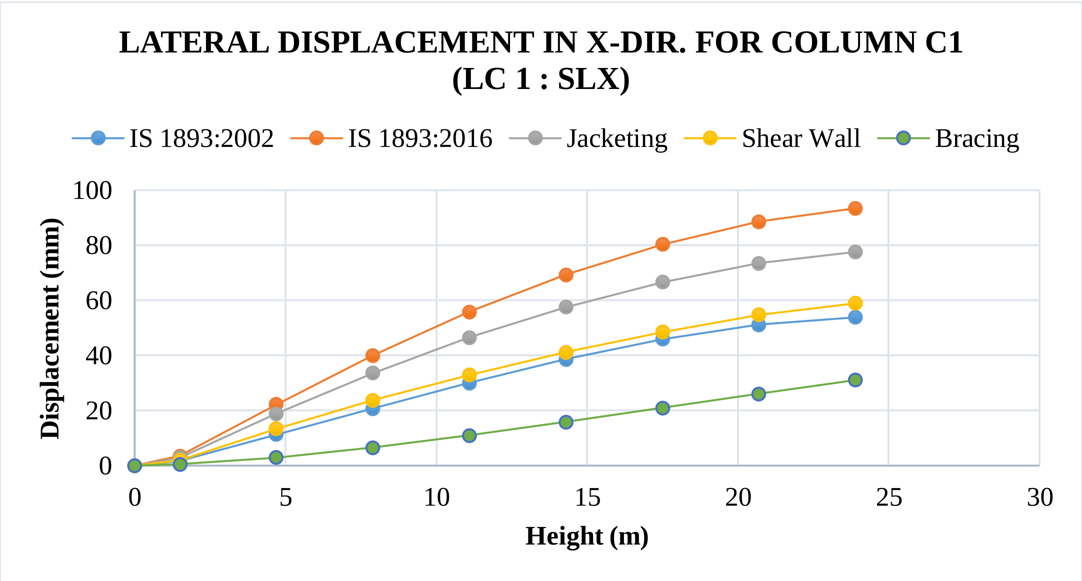


Fig.4.9 Lateral Displacement in X-Direction in column C-1

Table 4.10(a) - Lateral Displacement (mm) in X-Direction in column C-2

Node No.	Ht (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
3	0	0	0	0	0	0
45	1.50	1.691	3.176	2.764	1.228	2.092
63	4.70	10.933	20.373	16.464	8.216	13.301
81	7.90	20.330	37.578	29.422	15.504	24.591
99	11.10	29.417	52.879	40.990	22.461	34.754
117	14.30	37.762	65.881	50.891	28.940	43.6
135	17.50	44.759	76.352	58.855	34.705	50.993
153	20.70	49.678	83.864	64.545	39.226	56.465
171	23.90	52.063	88.151	67.838	41.933	59.591

Table 4.10(b) - Percentage change in Lateral Displacement w.r.t. IS1893:2016 model in column C-2

Node No.	Ht (m)	Jacketing (%)	Shear Wall (%)	Bracing (%)
3	0	0	0	0
45	1.50	-12.97	-61.34	-34.13
63	4.70	-19.19	-59.67	-34.71
81	7.90	-21.70	-58.74	-34.56
99	11.10	-22.48	-57.52	-34.28
117	14.30	-22.75	-56.07	-33.82
135	17.50	-22.92	-54.55	-33.21
153	20.70	-23.04	-53.23	-32.67
171	23.90	-23.04	-52.43	-32.40

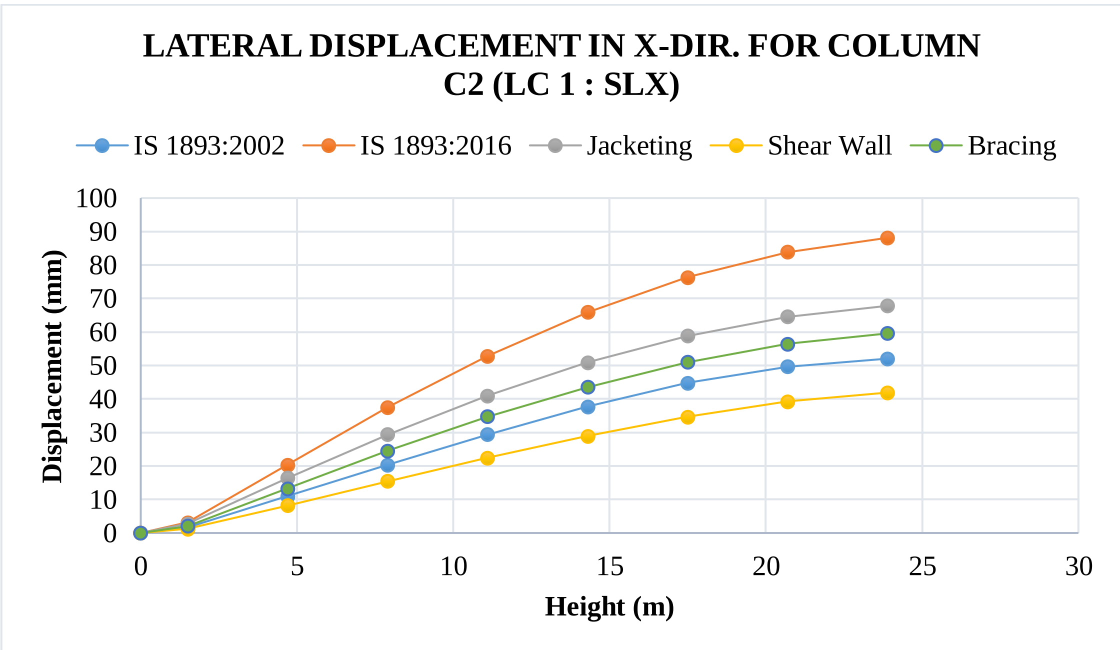


Fig.4.10 Lateral Displacement in X-Direction in column C-2

Table 4.11(a): Lateral Displacement (mm) in X-Direction in column C-3

Node No.	Ht (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
6	0	0	0	0	0	0
48	1.50	1.658	3.016	2.671	0.617	2.296
66	4.70	10.843	19.447	15.872	3.478	14.815
84	7.90	20.051	35.857	27.855	7.795	27.332
102	11.10	28.928	50.628	38.733	12.917	38.682
120	14.30	37.026	63.233	48.115	18.343	48.602
138	17.50	43.780	73.353	55.690	23.726	56.95
156	20.70	48.571	80.615	61.169	28.871	63.426
174	23.90	51.004	84.764	64.368	33.734	67.603

Table 4.11(b) - Percentage change in Lateral Displacement w.r.t. IS1893:2016 model in Column C-3

Node No.	Ht (m)	Jacketing (%)	Shear Wall (%)	Bracing (%)
6	0	0	0	0
48	1.50	-11.44	-79.54	-23.87
66	4.70	-18.38	-82.12	-23.82
84	7.90	-22.32	-78.26	-23.77
102	11.10	-23.49	-74.49	-23.60
120	14.30	-23.91	-70.99	-23.14
138	17.50	-24.08	-67.66	-22.36
156	20.70	-24.12	-64.19	-21.32
174	23.90	-24.06	-60.20	-20.25

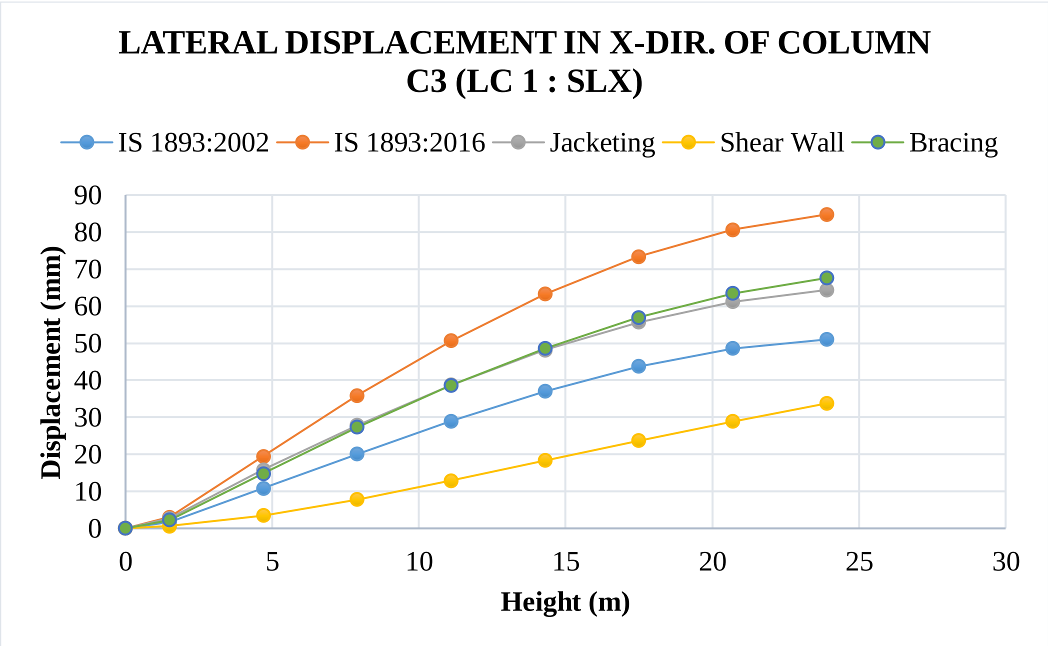


Fig.4.11 Lateral Displacement in X-Direction in column C-3

Table 4.12(a) - Lateral Displacement (mm) in X-Direction in column C-4

Node No.	Ht (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
27	0	0	0	0	0	0
267	1.50	1.689	3.481	2.951	2.059	0.548
271	4.70	11.308	22.273	18.851	13.353	2.905
275	7.90	20.783	39.995	33.601	23.757	6.524
279	11.10	30.054	55.771	46.548	32.943	10.975
283	14.30	38.634	69.273	57.569	41.129	15.897
287	17.50	45.899	80.341	66.642	48.524	21.004
291	20.70	51.124	88.548	73.460	54.754	26.096
295	23.90	53.845	93.444	77.588	58.933	31.06

Table4.12(b) - Percentage change in Lateral Displacement w.r.t. IS1893:2016 model in Column C-4

Node No.	Ht (m)	Jacketing (%)	Shear Wall (%)	Bracing (%)
27	0	0	0	0
267	1.50	-15.23	-40.85	-84.26
271	4.70	-15.36	-40.05	-86.96
275	7.90	-15.99	-40.60	-83.69
279	11.10	-16.54	-40.93	-80.32
283	14.30	-16.90	-40.63	-77.05
287	17.50	-17.05	-39.60	-73.86
291	20.70	-17.04	-38.16	-70.53
295	23.90	-16.97	-36.93	-66.76

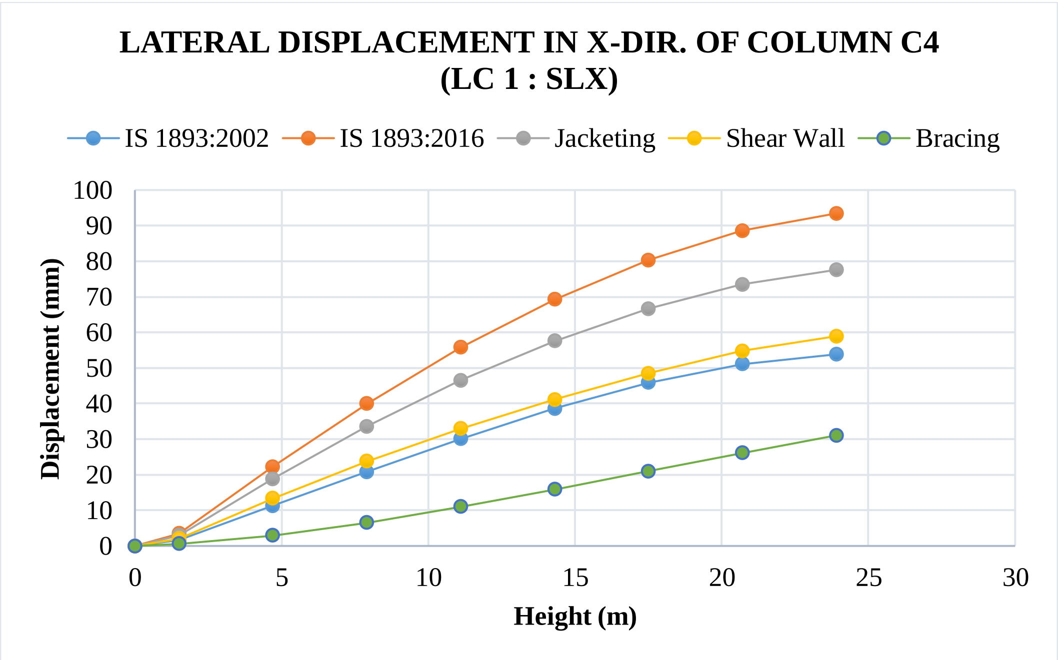


Fig.4.12 Lateral Displacement in X-Direction in column C-4

Table 4.13(a) - Lateral Displacement (mm) in X-Direction in column C-5

Node No.	Ht (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
2	0	0	0	0	0	0
44	1.50	1.690	3.179	2.790	1.224	2.098
62	4.70	10.938	20.381	16.481	8.224	13.3
80	7.90	20.329	37.577	29.420	15.512	24.583
98	11.10	29.418	52.880	40.991	22.471	34.743
116	14.30	37.763	65.881	50.892	28.952	43.587
134	17.50	44.760	76.352	58.856	34.717	50.977
152	20.70	49.682	83.866	64.549	39.239	56.448
170	23.90	52.063	88.152	67.837	41.940	59.579

Table 4.13(b) - Percentage change in Lateral Displacement w.r.t. IS1893:2016 model in Column C-5

Node No.	Ht (m)	Jacketing (%)	Shear Wall (%)	Bracing (%)
2	0	0	0	0
44	1.50	-12.24	-61.50	-34.00
62	4.70	-19.14	-59.65	-34.74
80	7.90	-21.71	-58.72	-34.58
98	11.10	-22.48	-57.51	-34.30
116	14.30	-22.75	-56.05	-33.84
134	17.50	-22.91	-54.53	-33.23
152	20.70	-23.03	-53.21	-32.69
170	23.90	-23.05	-52.42	-32.41

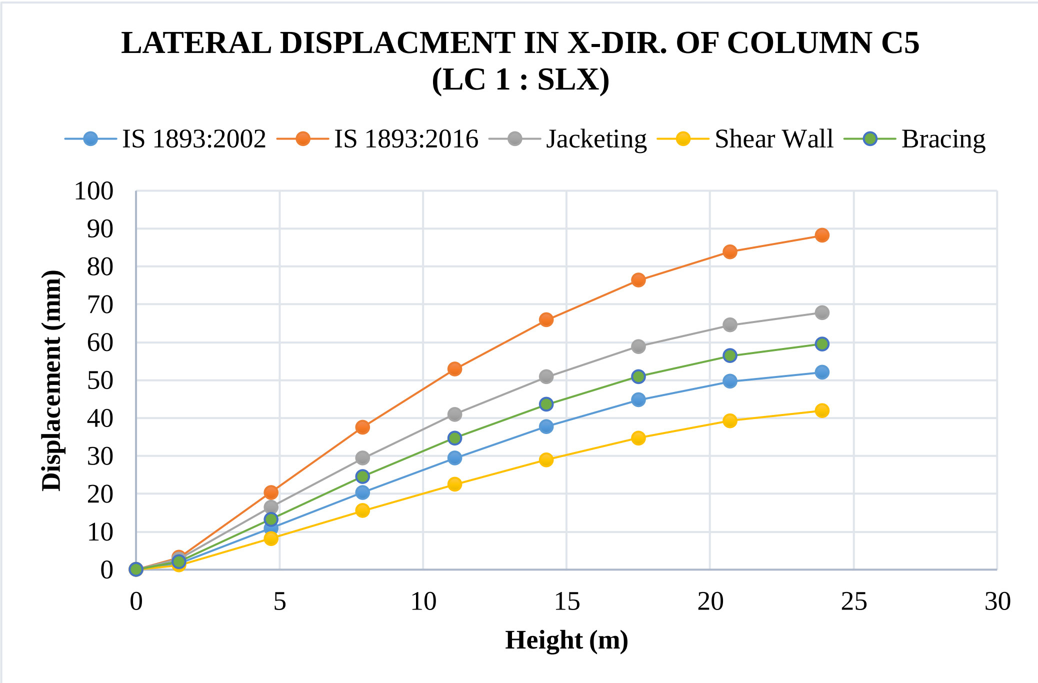


Fig.4.13 Lateral Displacement in X-Direction in column C-5

Table 4.14(a) - Lateral Displacement (mm) in X-Direction in column C-6

Node No.	Ht (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
5	0	0	0	0	0	0
47	1.50	1.657	3.014	2.678	0.609	2.294
65	4.70	10.840	19.438	15.870	3.468	14.812
83	7.90	20.046	35.849	27.849	7.777	27.333
101	11.10	28.921	50.621	38.726	12.895	38.687
119	14.30	37.016	63.225	48.107	18.319	48.61
137	17.50	43.765	73.344	55.682	23.699	56.96
155	20.70	48.549	80.602	61.158	28.851	63.433
173	23.90	50.994	84.757	64.359	33.607	67.597

Table 4.14(b) - Percentage change in Lateral Displacement w.r.t. IS1893:2016 model in Column C-6

Node No.	Ht (m)	Jacketing (%)	Shear Wall (%)	Bracing (%)
5	0	0	0	0
47	1.50	-11.15	-79.79	-23.89
65	4.70	-18.36	-82.16	-23.80
83	7.90	-22.32	-78.31	-23.76
101	11.10	-23.50	-74.53	-23.58
119	14.30	-23.91	-71.03	-23.12
137	17.50	-24.08	-67.69	-22.34
155	20.70	-24.12	-64.21	-21.30
173	23.90	-24.07	-60.35	-20.25

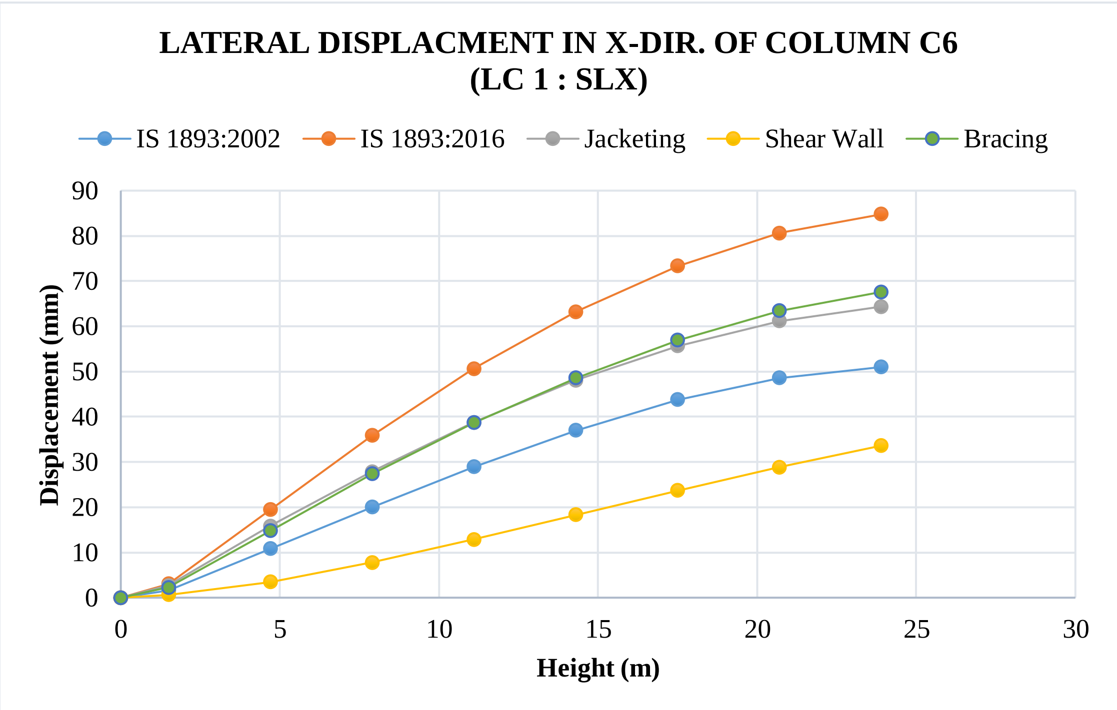


Fig4.14 Lateral Displacement in X-Direction in column C-6

Table 4.15(a) - Lateral Displacement (mm) in X-Direction in column C-10

Node No.	Ht (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
21	0	0	0	0	0	0
189	1.50	1.640	3.071	2.524	0.873	2.299
195	4.70	10.919	19.755	15.807	5.707	14.723
199	7.90	20.122	36.079	27.837	11.223	26.906
203	11.10	29.045	50.825	38.665	17.183	37.997
207	14.30	37.208	63.393	47.978	23.155	47.68
211	17.50	44.038	73.511	55.509	28.784	55.832
215	20.70	48.897	80.839	61.000	33.644	62.174
219	23.90	51.425	85.449	64.705	36.820	66.956
759	27.10	53.694	89.182	68.551	38.299	68.551

Table 4.15(b) - Percentage change in Lateral Displacement w.r.t. IS1893:2016 model in Column C-10

Node No.	Ht (m)	Jacketing (%)	Shear Wall (%)	Bracing (%)
21	0	0	0	0
189	1.50	-17.81	-71.57	-25.14
195	4.70	-19.98	-71.11	-25.47
199	7.90	-22.84	-68.89	-25.42
203	11.10	-23.93	-66.19	-25.24
207	14.30	-24.32	-63.47	-24.79
211	17.50	-24.49	-60.84	-24.05
215	20.70	-24.54	-58.38	-23.09
219	23.90	-24.28	-56.91	-21.64

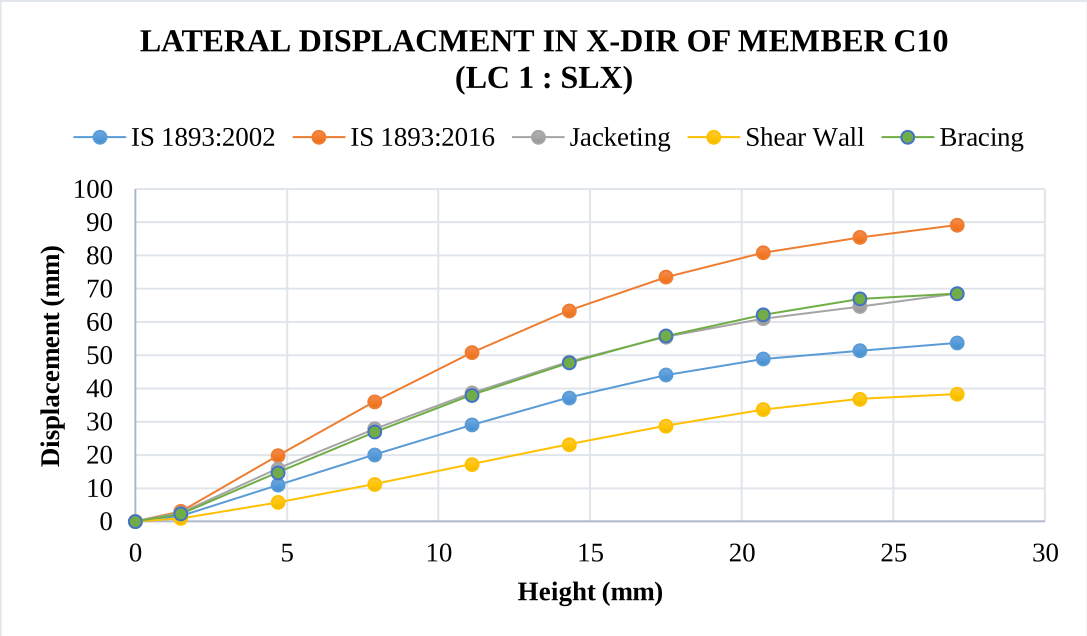


Fig.4.15 Lateral Displacement in X-Direction in column C-10

- **LATERAL DISPLACEMENT IN Z-DIRECTION** (Load Combination LC 2: SLZ)

Table 4.16(a) - Lateral Displacement (mm) in Z-Direction in column C-1

Node No.	Ht (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
28	0	0	0	0	0	0
268	1.50	1.675	3.316	2.792	2.383	2.118
272	4.70	11.119	21.848	18.237	15.906	13.936
276	7.90	21.226	41.45	34.822	30.521	26.275
280	11.10	31.106	59.291	50.045	43.904	37.251
284	14.30	40.182	74.566	63.129	55.372	46.418
288	17.50	47.741	86.888	73.675	64.587	53.59
292	20.70	52.939	95.643	81.193	70.976	58.295
296	23.90	55.288	100.416	85.435	74.175	60.248

Table 4.16(b) - Percentage change in Lateral Displacement w.r.t. IS1893:2016 model in Column C-1

Node No.	Ht (m)	Jacketing (%)	Shear Wall (%)	Bracing (%)
28	0	0	0	0
268	1.50	-15.80	-28.14	-36.13
272	4.70	-16.53	-27.20	-36.21
276	7.90	-15.99	-26.37	-36.61
280	11.10	-15.59	-25.95	-37.17
284	14.30	-15.34	-25.74	-37.75
288	17.50	-15.21	-25.67	-38.32
292	20.70	-15.11	-25.79	-39.05
296	23.90	-14.92	-26.13	-40.00

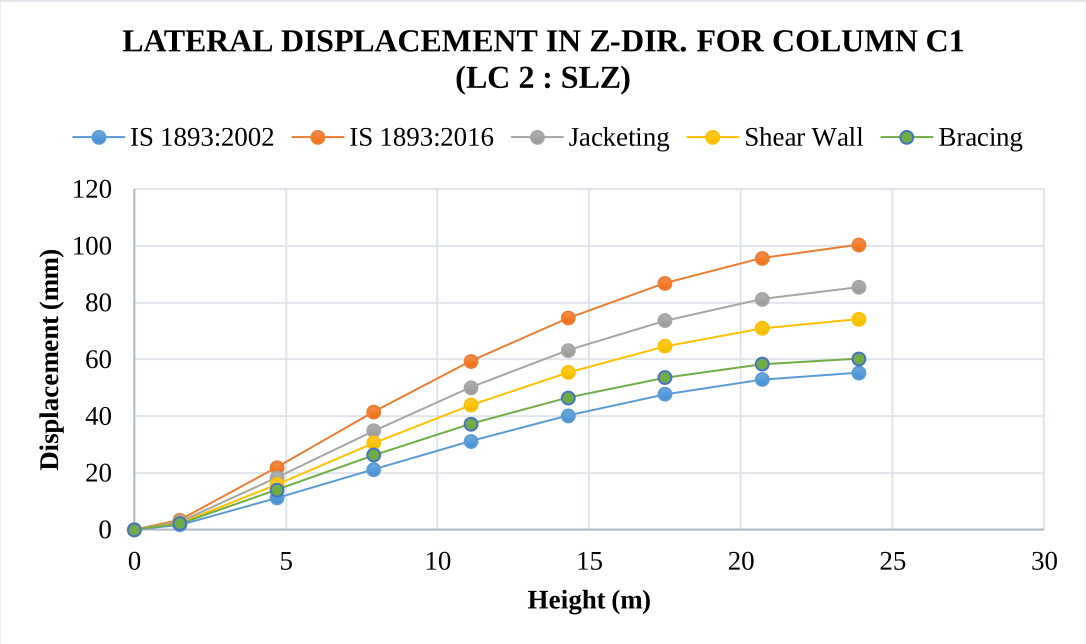


Fig.4.16 Lateral Displacement in Z-Direction in column C-1

Table 4.17(a) - Lateral Displacement (mm) in Z-Direction in column C-2

Node No.	Ht (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
3	0	0	0	0	0	0
45	1.50	1.663	3.322	2.796	2.385	2.122
63	4.70	11.134	21.873	18.25	15.919	13.95
81	7.90	21.222	41.444	34.819	30.513	26.265
99	11.10	31.103	59.287	50.044	43.896	37.241
117	14.30	40.178	74.561	63.127	55.362	46.406
135	17.50	47.738	86.882	73.673	64.577	53.577
153	20.70	52.944	95.646	81.195	70.973	58.288
171	23.90	55.281	100.407	85.429	74.159	60.22

Table 4.17(b) - Percentage change in Lateral Displacement w.r.t. IS1893:2016 model in Column C-2

Node No.	Ht (m)	Jacketing (%)	Shear Wall (%)	Bracing (%)
3	0	0	0	0
45	1.50	-15.83	-28.21	-36.12
63	4.70	-16.56	-27.22	-36.22
81	7.90	-15.99	-26.38	-36.63
99	11.10	-15.59	-25.96	-37.19
117	14.30	-15.34	-25.75	-37.76
135	17.50	-15.20	-25.67	-38.33
153	20.70	-15.11	-25.80	-39.06
171	23.90	-14.92	-26.14	-40.02

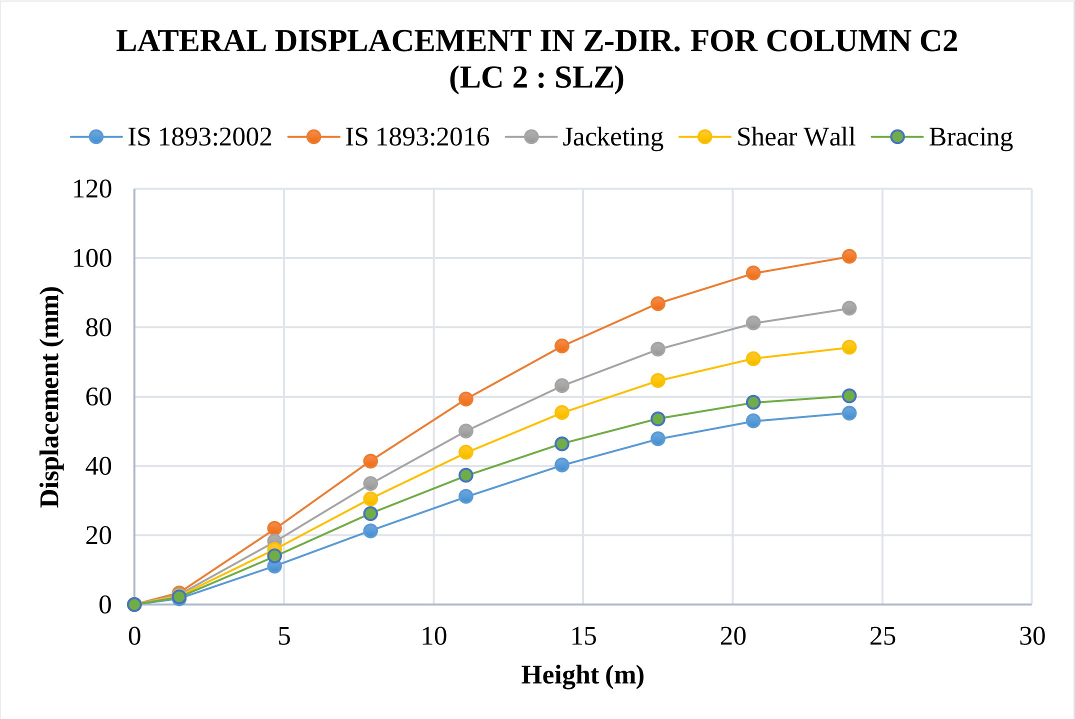


Fig.4.17 Lateral Displacement in Z-Direction in column C-2

Table 4.18(a) - Lateral Displacement (mm) in Z-Direction in column C-3

Node No.	Ht. (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
6	0	0	0	0	0	0
48	1.50	1.658	3.321	2.796	2.381	2.121
66	4.70	11.135	21.875	18.257	15.917	13.949
84	7.90	21.219	41.44	34.819	30.509	26.257
102	11.10	31.1	59.283	50.043	43.892	37.23
120	14.30	40.173	74.555	63.126	55.358	46.394
138	17.50	47.733	86.876	73.672	64.572	53.565
156	20.70	52.942	95.64	81.197	70.969	58.278
174	23.90	55.28	100.404	85.429	74.152	60.199

Table 4.18(b) - Percentage change in Lateral Displacement w.r.t. IS1893:2016 model in Column C-3

Node No.	Ht (m)	Jacketing (%)	Shear Wall (%)	Bracing (%)
6	0	0	0	0
48	1.50	-15.81	-28.30	-36.13
66	4.70	-16.54	-27.24	-36.23
84	7.90	-15.98	-26.38	-36.64
102	11.10	-15.59	-25.96	-37.20
120	14.30	-15.33	-25.75	-37.77
138	17.50	-15.20	-25.67	-38.34
156	20.70	-15.10	-25.80	-39.07
174	23.90	-14.91	-26.15	-40.04

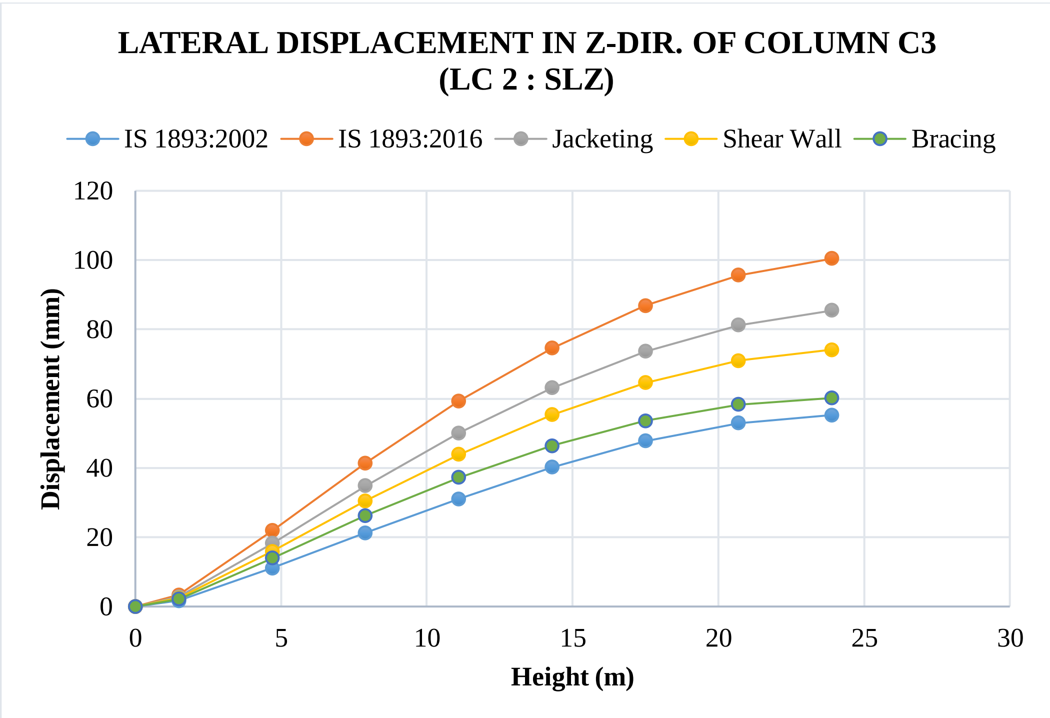


Fig.4.18 Lateral Displacement in Z-Direction in column C-3

Table 4.19(a) - Lateral Displacement (mm) in Z-Direction in column C-4

Node No.	Ht (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
27	0	0	0	0	0	0
267	1.50	1.539	2.932	2.732	2.332	2.187
271	4.70	10.761	20.627	17.98	16.844	15.294
275	7.90	20.992	40.494	34.747	33.226	29.84
279	11.10	30.86	58.313	50.05	47.922	42.63
283	14.30	39.843	73.323	63.093	60.267	53.204
287	17.50	47.276	85.357	73.577	70.088	61.643
291	20.70	52.307	94.012	81.137	77.022	67.821
295	23.90	54.571	99.21	85.738	81.046	71.76

Table 4.19(b) - Percentage change in Lateral Displacement w.r.t. IS1893:2016 model in Column C-4

Node No.	Ht (m)	Jacketing (%)	Shear Wall (%)	Bracing (%)
27	0	0	0	0
267	1.50	-6.82	-20.46	-25.41
271	4.70	-12.83	-18.34	-25.85
275	7.90	-14.19	-17.95	-26.31
279	11.10	-14.17	-17.82	-26.89
283	14.30	-13.95	-17.81	-27.44
287	17.50	-13.80	-17.89	-27.78
291	20.70	-13.70	-18.07	-27.86
295	23.90	-13.58	-18.31	-27.67

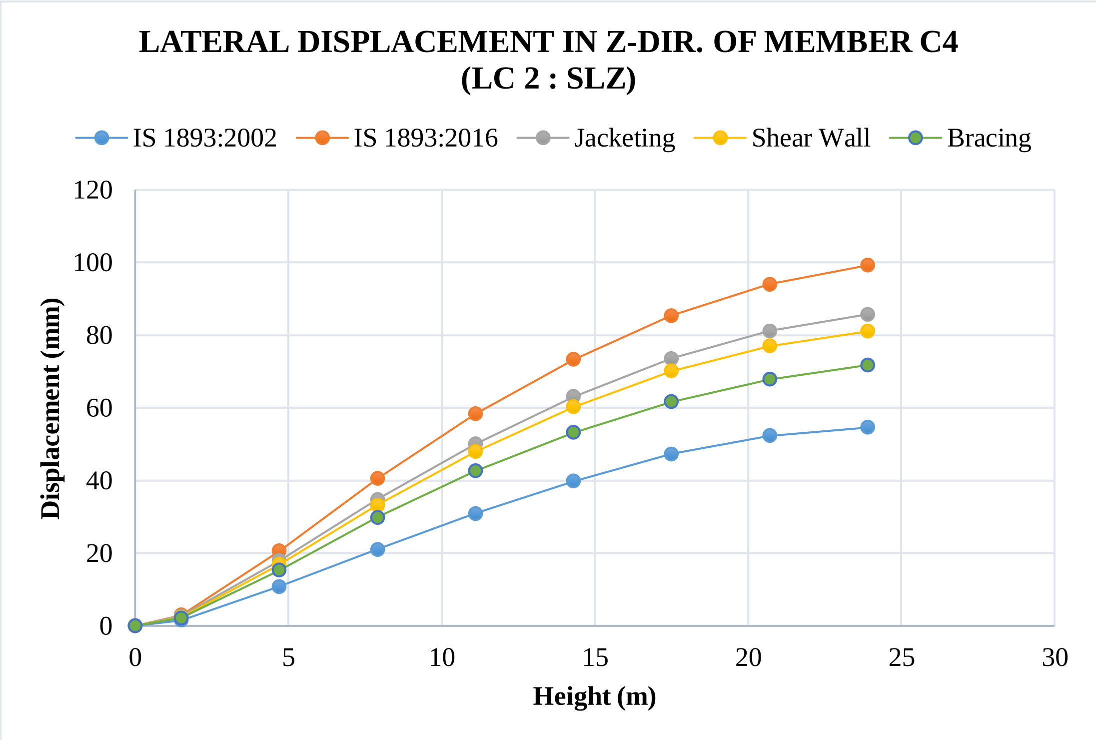


Fig.4.19 Lateral Displacement in Z-Direction in column C-4

Table 4.20(a) - Lateral Displacement (mm) in Z-Direction in column C-5

Node No.	Ht (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
2	0	0	0	0	0	0
44	1.50	1.502	2.897	2.689	2.29	2.159
62	4.70	10.76	20.627	17.965	16.844	15.294
80	7.90	20.995	40.497	34.751	33.228	29.843
98	11.10	30.86	58.314	50.051	47.921	42.63
116	14.30	39.844	73.324	63.095	60.266	53.205
134	17.50	47.278	85.357	73.579	70.086	61.643
152	20.70	52.311	94.012	81.137	77.02	67.819
170	23.90	54.566	99.208	85.74	81.042	71.763

Table 4.20(b) - Percentage change in Lateral Displacement w.r.t. IS1893:2016 model in Column C-5

Node No.	Ht (m)	Jacketing (%)	Shear Wall (%)	Bracing (%)
2	0	0	0	0
44	1.50	-7.18	-20.95	-25.47
62	4.70	-12.91	-18.34	-25.85
80	7.90	-14.19	-17.95	-26.31
98	11.10	-14.17	-17.82	-26.90
116	14.30	-13.95	-17.81	-27.44
134	17.50	-13.80	-17.89	-27.78
152	20.70	-13.70	-18.07	-27.86
170	23.90	-13.58	-18.31	-27.66

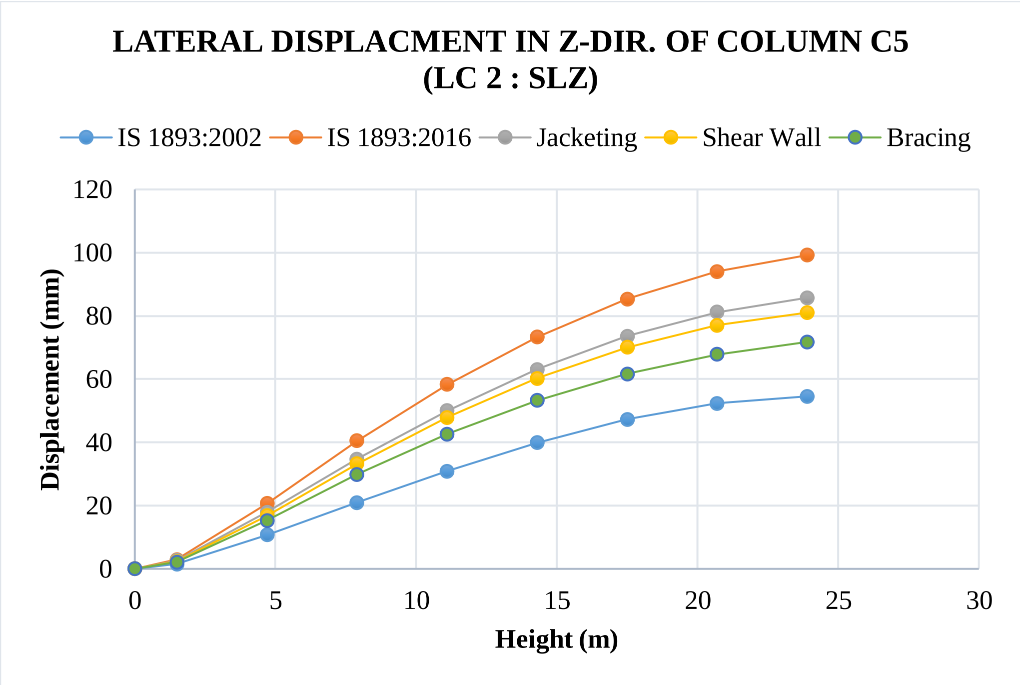


Fig.4.20 Lateral Displacement in Z-Direction in column C-5

Table 4.21(a) - Lateral Displacement (mm) in Z-Direction in column C-6

Node No.	Ht (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
5	0	0	0	0	0	0
47	1.50	1.474	2.874	2.669	2.253	2.141
65	4.70	10.630	20.485	17.973	15.443	15.133
83	7.90	21.088	40.679	34.923	31.446	29.933
101	11.10	30.901	58.439	50.210	45.797	42.705
119	14.30	39.877	73.432	63.246	57.877	53.304
137	17.50	47.339	85.496	73.748	67.501	61.807
155	20.70	52.388	94.141	81.324	74.264	67.825
173	23.90	54.516	99.324	85.841	78.438	70.908

Table 4.21(b) - Percentage change in Lateral Displacement w.r.t. IS1893:2016 model in Column C-6

Node No.	Ht (m)	Jacketing (%)	Shear Wall (%)	Bracing (%)
5	0	0	0	0
47	1.50	-7.13	-21.61	-25.50
65	4.70	-12.26	-24.61	-26.13
83	7.90	-14.15	-22.70	-26.42
101	11.10	-14.08	-21.63	-26.92
119	14.30	-13.87	-21.18	-27.41
137	17.50	-13.74	-21.05	-27.71
155	20.70	-13.61	-21.11	-27.95
173	23.90	-13.57	-21.03	-28.61

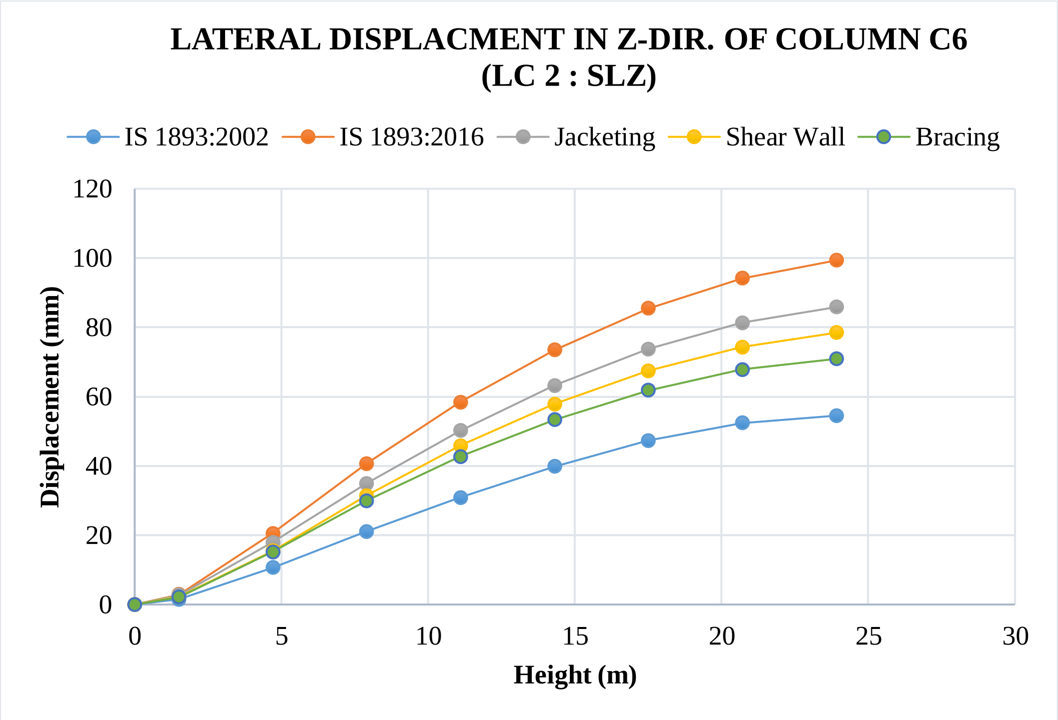


Fig.4.21 Lateral Displacement in Z-Direction in column C-6

Table 4.22(a) - Lateral Displacement (mm) in Z-Direction in column C-10

Node No.	Ht (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
21	0	0	0	0	0	0
189	1.50	1.711	3.178	2.826	2.426	0.608
195	4.70	10.94	21.018	18.634	15.987	2.623
199	7.90	20.865	40.756	35.971	30.775	5.606
203	11.10	30.603	58.864	51.68	43.957	9.321
207	14.30	39.617	74.239	64.862	54.606	13.533
211	17.50	47.374	86.574	75.4	62.396	18.039
215	20.70	53.36	95.731	83.309	67.034	22.672
219	23.90	57.358	101.703	88.641	68.299	27.302
759	27.10	59.771	105.14	91.928	67.078	31.87

Table 4.22(b) - Percentage change in Lateral Displacement w.r.t. IS1893:2016 model in Column C-10

Node No.	Ht (m)	Jacketing (%)	Shear Wall (%)	Bracing (%)
21	0	0	0	0
189	1.50	-11.08	-23.66	-80.87
195	4.70	-11.34	-23.94	-87.52
199	7.90	-11.74	-24.49	-86.24
203	11.10	-12.20	-25.32	-84.17
207	14.30	-12.63	-26.45	-81.77
211	17.50	-12.91	-27.93	-79.16
215	20.70	-12.98	-29.98	-76.32
219	23.90	-12.84	-32.84	-73.16

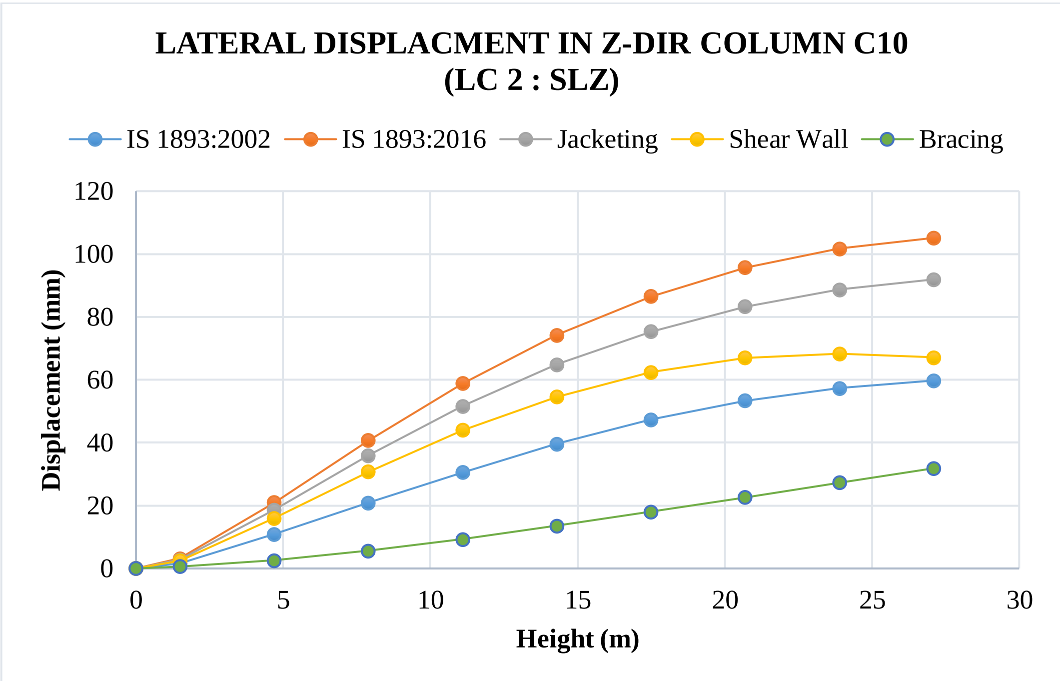


Fig.4.22 Lateral Displacement in Z-Direction in column C-10

The retrofitted models experienced less lateral displacement than the existing model during dynamic analysis. It was due to the reason the models were retrofitted with techniques which resist the lateral force of the members and subsequently the deflections are reduced.

- In Model 3 i.e. model with jacketing, the members showed a decrease of 10-30% in lateral displacement.
- In Model 4 i.e. model with shear wall, the members showed an average decrease in lateral displacement of 30-50%.
- In Model 5 i.e. model with steel bracing, members showed an average decrease in lateral displacement of 40-60%.
- In some members in Model 4 & Model 5 the value decreased by 80%, it is due to the reason those members were directly connected to the displacement resisting shear wall or bracings.

4.4 STOREY DRIFT

Storey drift is the relative horizontal displacement between adjacent floors or storeys of a building during lateral loading. It plays a critical role in assessing the deformations and differential movements experienced by different levels of a structure under dynamic loads such as earthquakes, wind forces, or other lateral forces.

Analyzing storey drift is vital, it helps to assess the structural response to lateral forces. By evaluating its magnitude, we can identify potential areas of concern and design appropriate measures to control and limit excessive differential movement between storeys. Excessive storey drift can have detrimental effects on a structure. It can lead to increased stresses and strains in structural elements, potentially resulting in structural damage, such as cracking, deformation, or even collapse. To mitigate the negative effects of storey drift, various design strategies are practiced. These include the selection of appropriate lateral load resisting systems such as shear walls, bracing systems, or moment-resisting frames. By implementing robust and well-designed lateral load resisting systems, the structure can effectively distribute and dissipate lateral forces, reducing storey drift and enhancing the overall stability and performance of the structure. Dynamic analysis is performed using softwares, to accurately predict and assess the anticipated storey drift under different loading conditions. This allows for optimization of the structural design and identification of critical areas where additional measures, such as stiffness or strength enhancements, may be necessary. It also helps in retrofitting works to be done on existing structures. In this study storey drift was calculated for all the models in both x and z direction. The results are shown below in Table 4.23 & 4.24 and Fig. 4.23 & 4.24:

- **Storey Drift in X-Direction** (Load Combination LC 1: SLX)

Table 4.23 – Storey Drift (mm) in X-Direction

Floor Level	Ht (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
Base	0	0	0	0	0	0
Stilt Floor	1.5	1.69	3.18	2.76	1.23	2.09
1st Floor	4.7	9.24	17.20	13.70	6.99	11.21
2nd Floor	7.9	9.40	17.21	12.96	7.29	11.29
3rd Floor	11.1	9.09	15.30	11.57	6.96	10.16
4th Floor	14.3	8.35	13.00	9.90	6.48	8.85
5th Floor	17.5	7.00	10.47	7.96	5.77	7.39
6th Floor	20.7	4.92	7.51	5.69	4.52	5.47
7th Floor	23.9	2.39	4.29	3.29	2.71	3.13

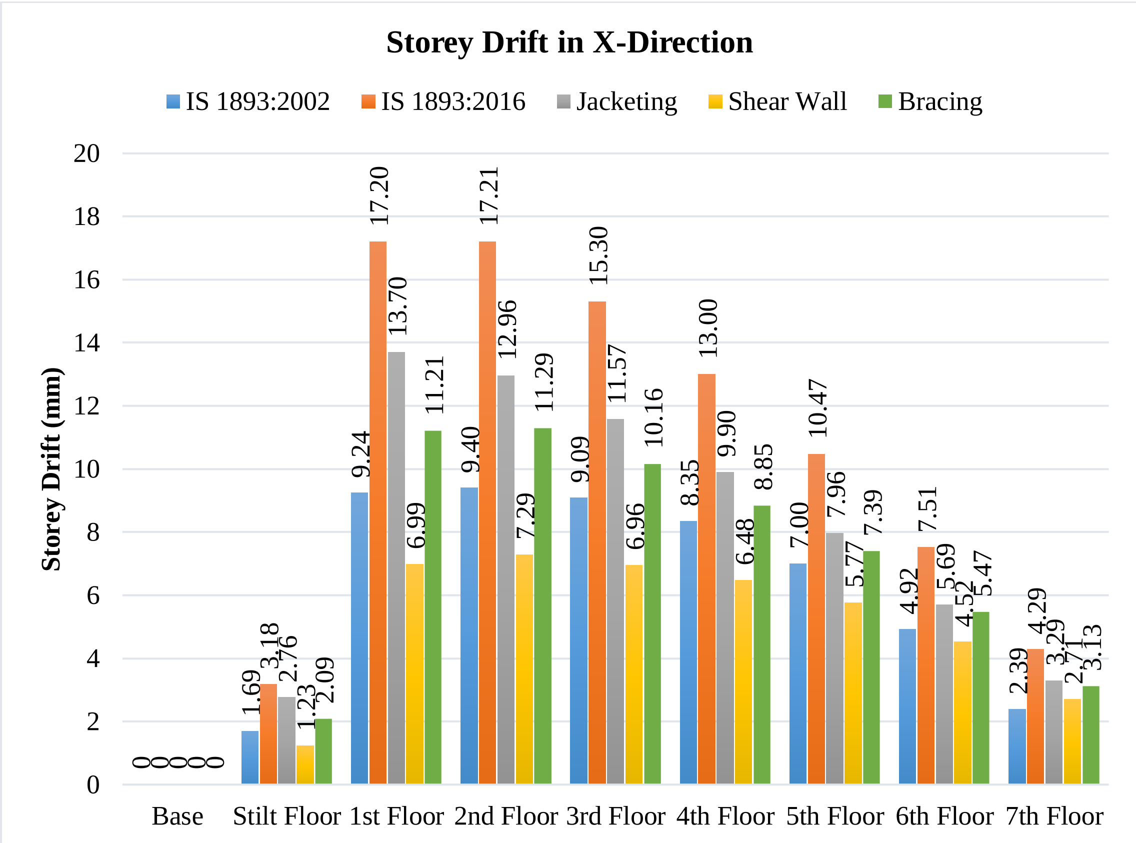


Fig.4.23 Storey Drift in X-Direction

- **Storey Drift in Z-Direction** (Load Combination LC 1: SLZ)

Table 4.24 – Storey Drift (mm) in Z-Direction

Floor Level	Ht (m)	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
Base	0	0	0	0	0	0
Stilt Floor	1.5	1.66	3.32	2.80	2.39	2.12
1st Floor	4.7	9.47	18.55	15.45	13.53	11.83
2nd Floor	7.9	10.09	19.57	16.57	14.59	12.32
3rd Floor	11.1	9.88	17.84	15.23	13.38	10.98
4th Floor	14.3	9.08	15.27	13.08	11.47	9.17
5th Floor	17.5	7.56	12.32	10.55	9.22	7.17
6th Floor	20.7	5.21	8.76	7.52	6.40	4.71
7th Floor	23.9	2.34	4.76	4.23	3.19	1.93

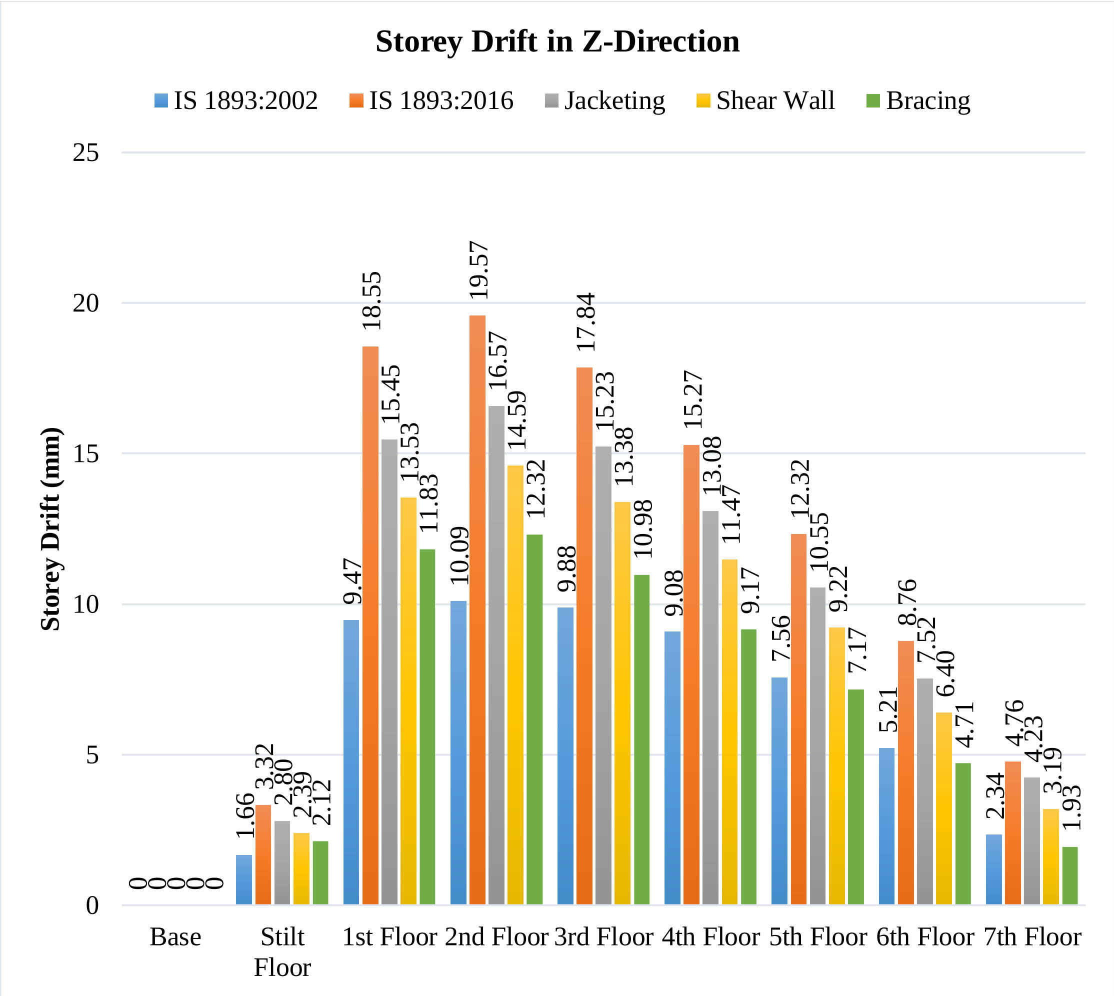


Fig.4.24 Storey Drift in Z-Direction

4.5 BENDING MOMENT IN COLUMNS

Bending moment in columns refers to the internal forces and moments experienced by a column due to external loads and lateral loads applied to it. Columns are vertical structural members designed to primarily support axial loads, bending forces. Bending moments in columns typically occur due to earthquake lateral loads and some due to eccentric loading.

The magnitude of the bending moment in a column depends on several factors, including the applied loads, the location and eccentricity of the loads, the column's geometry, and the support conditions. The maximum bending moment is considered to determine the required column size, reinforcement, and overall structural stability also factors such as the column's material properties, its slenderness ratio, and the anticipated loads to ensure that the column can withstand the bending stresses and deformations without compromising the structural integrity. In this study the bending moment in columns has been given due diligence and its value has been calculated at every level for all the models and analysed, results for the selected columns (Fig.4.25) are discussed below from Table 4.25 – 4.39 and Fig.4.26 – 4.40:

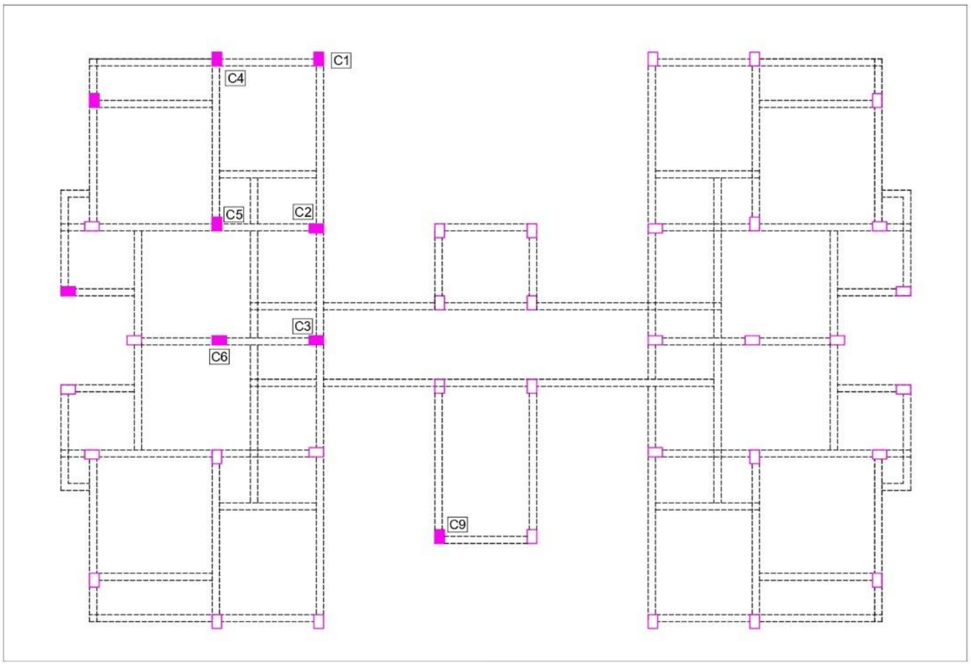


Fig.4.25 Plan highlighting the columns studied

- **Bending Moment about Z-Axis** (Load combination : LC 11 = 1.5 (DL + SLX))

Table 4.25(a) – Bending Moment (kNm) about Z-Axis in column C-1

Member No.	Storey Level	IS1893:2002	IS1893:2016	Jacketing	Shear Wall	Bracing
100	Foundation	71	99	78	59	9
101	Stilt Floor	95	96	81	61	4
102	1st Floor	98	95	78	61	10
103	2nd Floor	96	88	72	59	8
104	3rd Floor	91	77	68	61	8
105	4th Floor	81	63	62	61	8
106	5th Floor	63	47	51	53	9
107	6th Floor	35	29	32	36	10

Table 4.25(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 model in Column C-1

Member No.	Storey Level	Jacketing (%)	Shear Wall (%)	Bracing (%)
100	Foundation	-21.21	-40.40	-90.91
101	Stilt Floor	-15.63	-36.46	-95.83
102	1st Floor	-17.89	-35.79	-89.47
103	2nd Floor	-18.18	-32.95	-90.91
104	3rd Floor	-11.69	-20.78	-89.61
105	4th Floor	-1.59	-3.17	-87.30
106	5th Floor	8.51	12.77	-80.85
107	6th Floor	10.34	24.14	-65.52

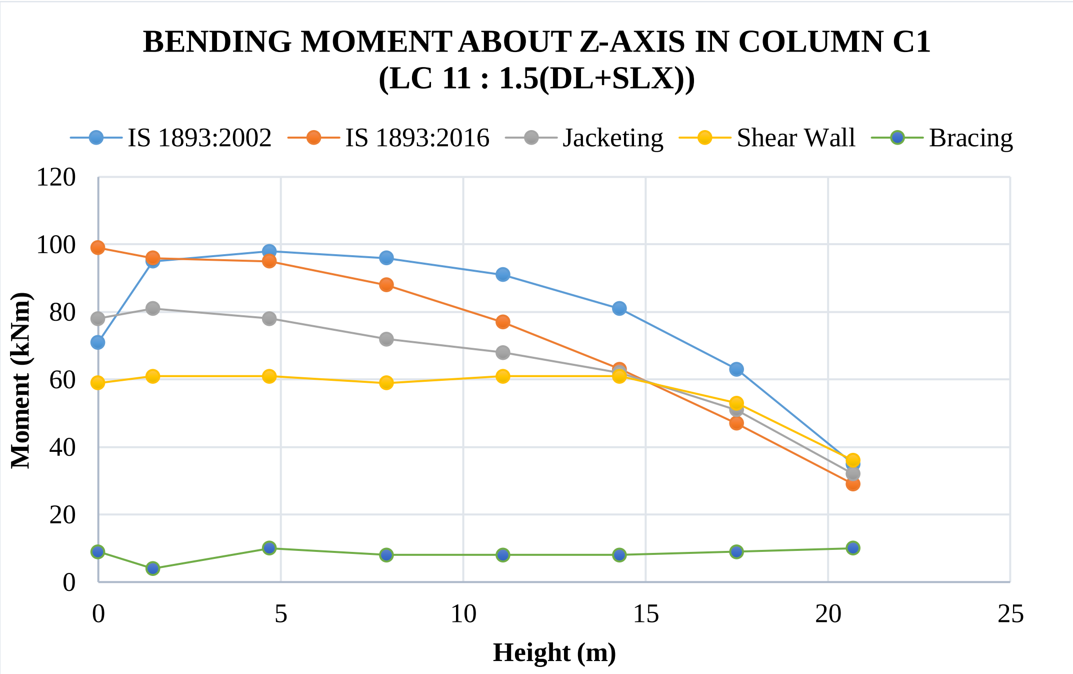


Fig.4.26 Bending Moment about Z-Axis in column C-1

Table 4.26(a) – Bending Moment (kNm) about Z-Axis in column C-2

Member No.	Storey Level	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
200	Foundation	144	186	356	69	124
201	Stilt Floor	139	150	224	65	99
202	1st Floor	144	130	168	70	94
203	2nd Floor	144	119	165	70	89
204	3rd Floor	138	103	146	70	86
205	4th Floor	126	84	123	66	80
206	5th Floor	105	64	90	56	66
207	6th Floor	63	46	47	39	47

Table 4.26(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 model in Column C-2

Member No.	Storey Level	Jacketing (%)	Shear Wall (%)	Bracing (%)
200	Foundation	91.40	-62.90	-33.33
201	Stilt Floor	49.33	-56.67	-34.00
202	1st Floor	29.23	-46.15	-27.69
203	2nd Floor	38.66	-41.18	-25.21
204	3rd Floor	41.75	-32.04	-16.50
205	4th Floor	46.43	-21.43	-4.76
206	5th Floor	40.63	-12.50	3.13
207	6th Floor	2.17	-15.22	2.17

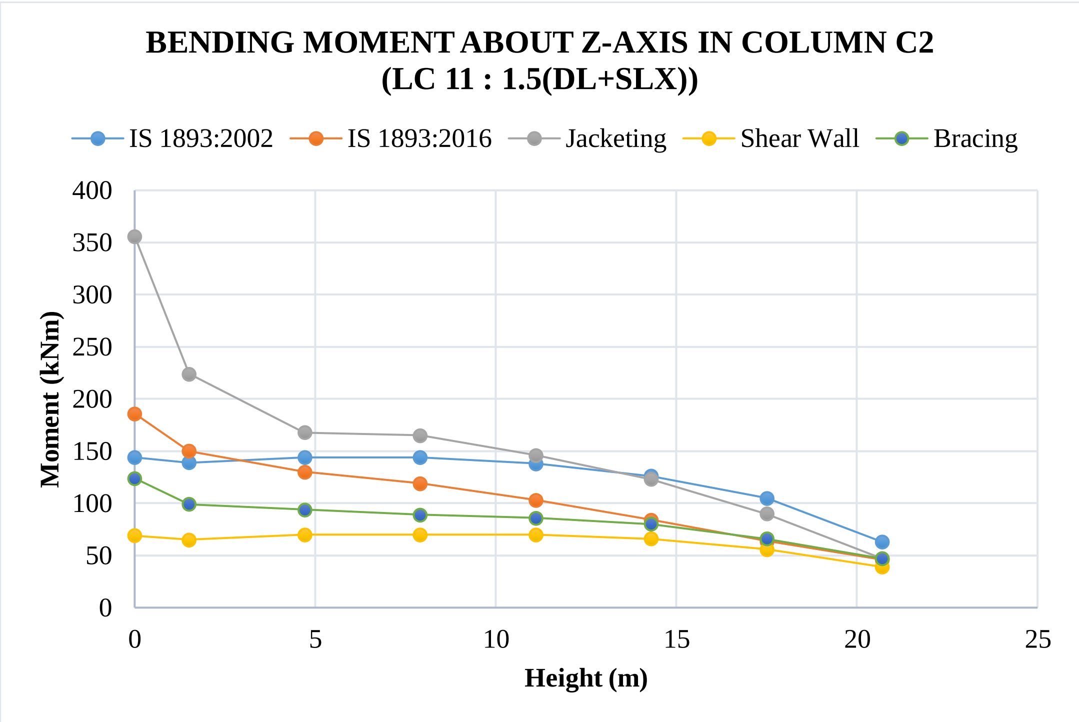


Fig.4.27 Bending Moment about Z-Axis in column C-2

Table4.27(a) – Bending Moment (kNm) about Z-Axis in column C-3

Member No.	Storey Level	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
300	Foundation	144	184	154	42	139
301	Stilt Floor	154	164	135	16	125
302	1st Floor	158	153	141	19	120
303	2nd Floor	154	137	130	19	110
304	3rd Floor	145	116	114	18	103
305	4th Floor	127	91	95	17	92
306	5th Floor	98	63	71	17	74
307	6th Floor	58	38	45	18	52

Table 4.27(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 model in Column C-3

Member No.	Storey Level	Jacketing (%)	Shear Wall (%)	Bracing (%)
300	Foundation	-16.30	-77.17	-24.46
301	Stilt Floor	-17.68	-90.24	-23.78
302	1st Floor	-7.84	-87.58	-21.57
303	2nd Floor	-5.11	-86.13	-19.71
304	3rd Floor	-1.72	-84.48	-11.21
305	4th Floor	4.40	-81.32	1.10
306	5th Floor	12.70	-73.02	17.46
307	6th Floor	18.42	-52.63	36.84

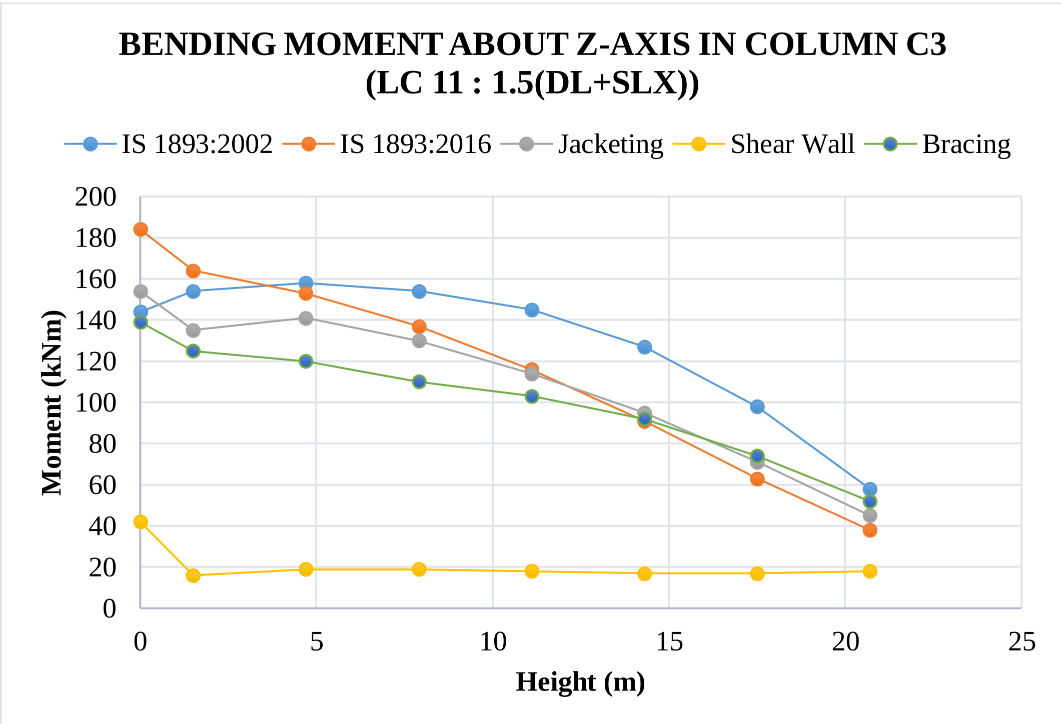


Fig.4.28 Bending Moment about Z-Axis in column C-3

Table 4.28(a) – Bending Moment (kNm) about Z-Axis in column C-4

Member No.	Storey Level	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
400	Foundation	83	111	89	66	22
401	Stilt Floor	116	118	100	74	9
402	1st Floor	119	124	98	74	16
403	2nd Floor	115	112	87	69	17
404	3rd Floor	106	97	80	70	19
405	4th Floor	91	77	71	69	19
406	5th Floor	66	54	55	58	18
407	6th Floor	32	30	30	37	19

Table 4.28(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 model in Column C-4

Member No.	Storey Level	Jacketing (%)	Shear Wall (%)	Bracing (%)
400	Foundation	-19.82	-40.54	-80.18
401	Stilt Floor	-15.25	-37.29	-92.37
402	1st Floor	-20.97	-40.32	-87.10
403	2nd Floor	-22.32	-38.39	-84.82
404	3rd Floor	-17.53	-27.84	-80.41
405	4th Floor	-7.79	-10.39	-75.32
406	5th Floor	1.85	7.41	-66.67
407	6th Floor	0.00	23.33	-36.67

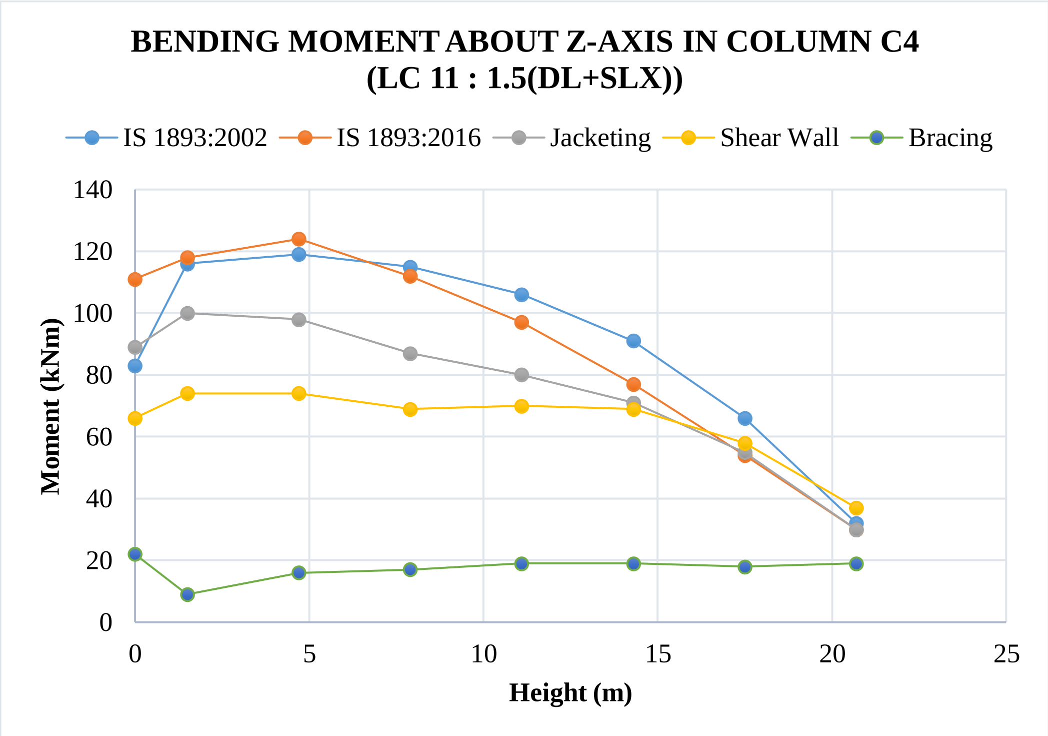


Fig.4.29 Bending Moment about Z-Axis in column C-4

Table 4.29(a) – Bending Moment (kNm) about Z-Axis in column C-5

Member No.	Storey Level	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
500	Foundation	89	101	204	37	67
501	Stilt Floor	121	136	230	59	92
502	1st Floor	131	153	235	67	103
503	2nd Floor	122	139	204	64	93
504	3rd Floor	109	121	178	62	87
505	4th Floor	88	97	147	58	79
506	5th Floor	56	68	106	47	63
507	6th Floor	26	32	46	22	32

Table 4.29(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 model in Column C-5

Member No.	Storey Level	Jacketing (%)	Shear Wall (%)	Bracing (%)
500	Foundation	101.98	-63.37	-33.66
501	Stilt Floor	69.12	-56.62	-32.35
502	1st Floor	53.59	-56.21	-32.68
503	2nd Floor	46.76	-53.96	-33.09
504	3rd Floor	47.11	-48.76	-28.10
505	4th Floor	51.55	-40.21	-18.56
506	5th Floor	55.88	-30.88	-7.35
507	6th Floor	43.75	-31.25	0.00

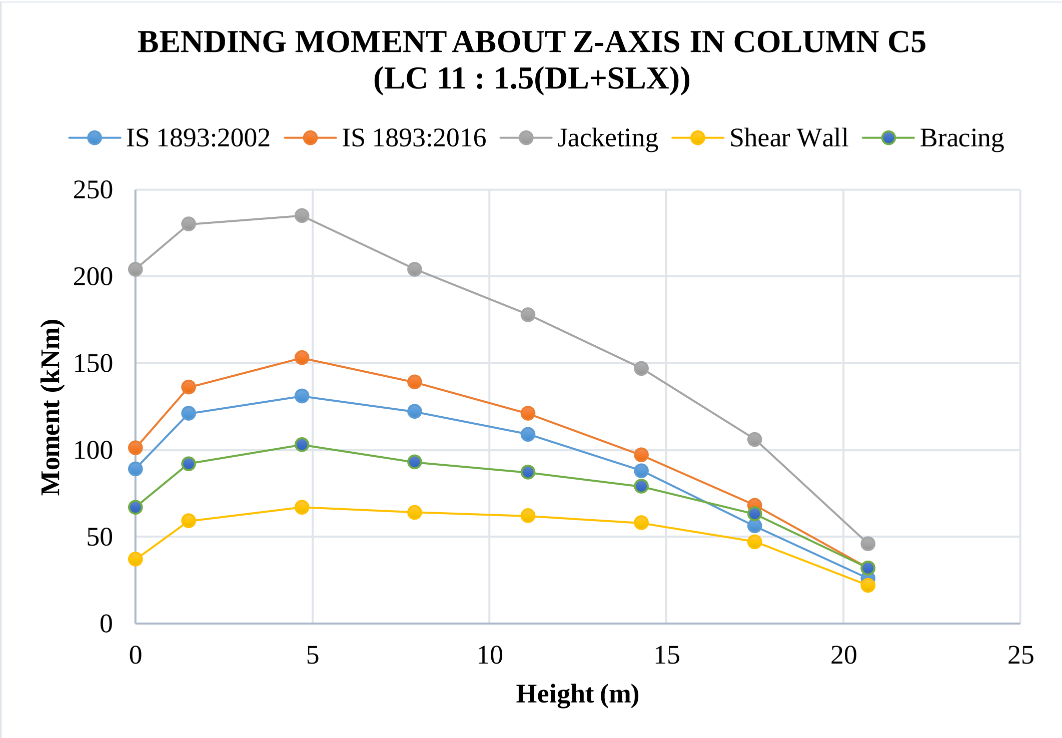


Fig.4.30 Bending Moment about Z-Axis in column C-5

Table 4.30(a) – Bending Moment (kNm) about Z-Axis in column C-6

Member No.	Storey Level	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
600	0.00	157	189	158	52	143
601	1.50	188	202	209	21	156
602	4.70	191	219	212	45	166
603	7.90	174	204	192	54	155
604	11.10	153	182	170	58	145
605	14.30	120	152	144	58	134
606	17.50	73	115	110	56	116
607	20.70	20	67	60	55	81

Table 4.30(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 model in Column C-6

Member No.	Storey Level	Jacketing (%)	Shear Wall (%)	Bracing (%)
600	Foundation	-16.40	-72.49	-24.34
601	Stilt Floor	3.47	-89.60	-22.77
602	1st Floor	-3.20	-79.45	-24.20
603	2nd Floor	-5.88	-73.53	-24.02
604	3rd Floor	-6.59	-68.13	-20.33
605	4th Floor	-5.26	-61.84	-11.84
606	5th Floor	-4.35	-51.30	0.87
607	6th Floor	-10.45	-17.91	20.90

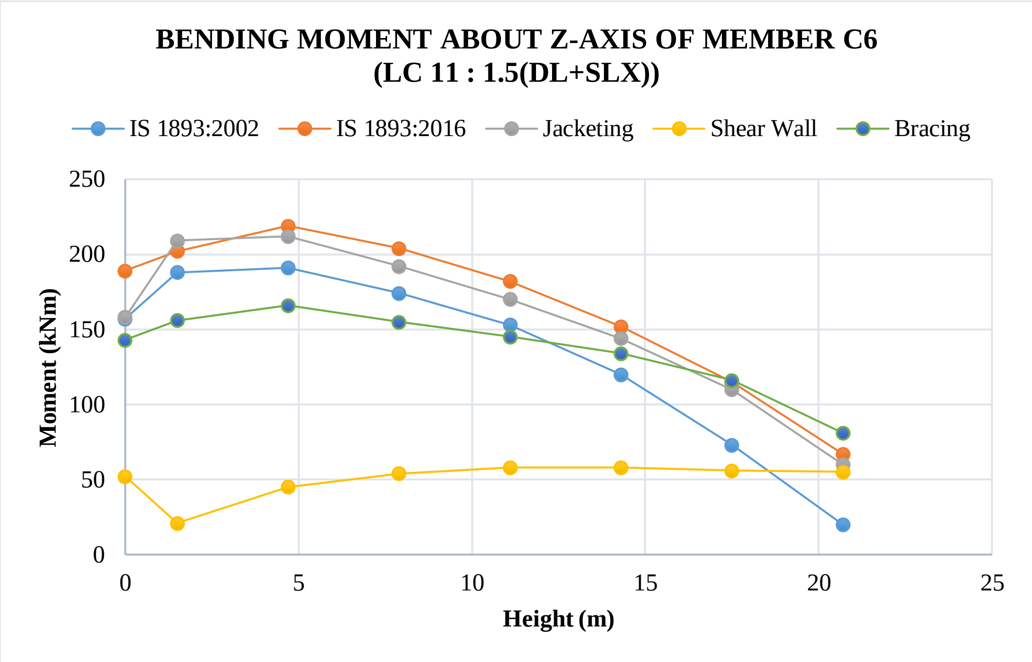


Fig.4.31 Bending Moment about Z-Axis in column C-6

Table 4.31(a) – Bending Moment (kNm) about Z-Axis in column C-8

Member No.	Storey Level	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
800	Foundation	132	177	140	64	122
801	Stilt Floor	97	105	87	28	79
802	1st Floor	71	108	85	38	87
803	2nd Floor	61	107	82	44	85
804	3rd Floor	50	101	79	47	83
805	4th Floor	36	92	75	48	83
806	5th Floor	14	79	69	48	80
807	6th Floor	35	58	58	45	64

Table 4.31(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 model in Column C-8

Member No.	Storey Level	Jacketing (%)	Shear Wall (%)	Bracing (%)
800	Foundation	-20.90	-63.84	-31.07
801	Stilt Floor	-17.14	-73.33	-24.76
802	1st Floor	-21.30	-64.81	-19.44
803	2nd Floor	-23.36	-58.88	-20.56
804	3rd Floor	-21.78	-53.47	-17.82
805	4th Floor	-18.48	-47.83	-9.78
806	5th Floor	-12.66	-39.24	1.27
807	6th Floor	0.00	-22.41	10.34

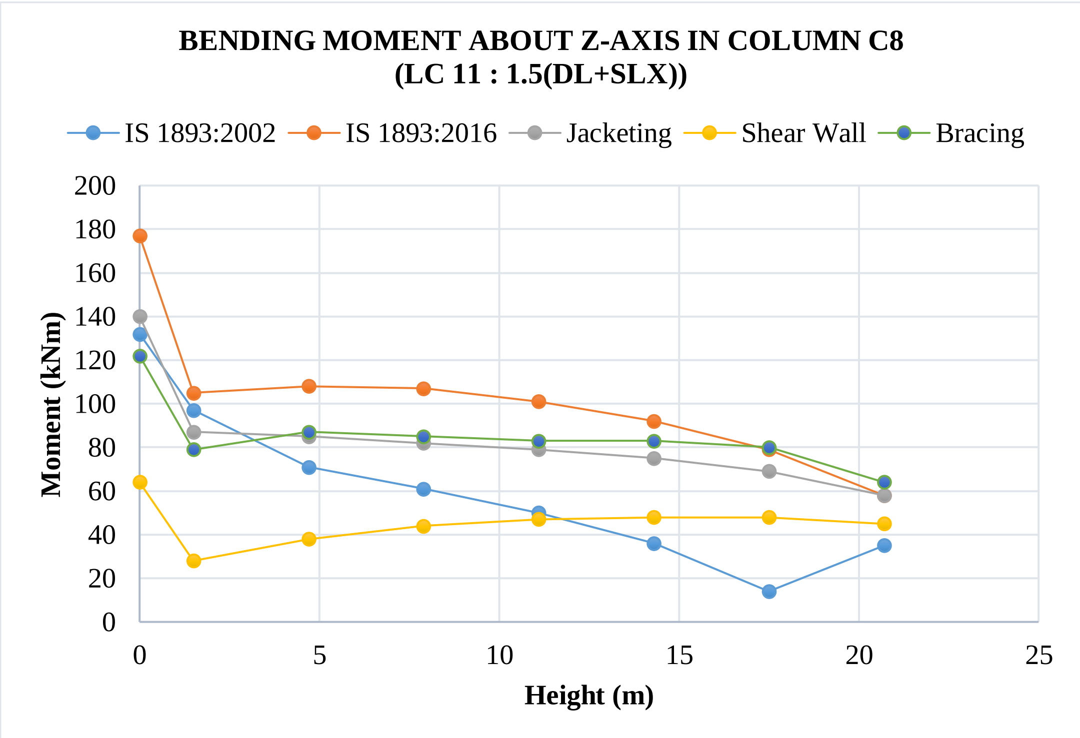


Fig.4.32 Bending Moment about Z-Axis in column C-8

Table 4.32(a) – Bending Moment (kNm) about Z-Axis in column C-10

Member No.	Storey Level	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
1000	Foundation	81	95	74	26	69
1001	Stilt Floor	107	107	87	36	77
1002	1st Floor	112	114	81	47	79
1003	2nd Floor	111	105	74	53	73
1004	3rd Floor	103	92	67	55	68
1005	4th Floor	88	74	58	55	61
1006	5th Floor	65	52	45	52	48
1007	6th Floor	33	28	27	40	34

Table 4.32(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 model in Column C-10

Member No.	Storey Level	Jacketing (%)	Shear Wall (%)	Bracing (%)
1000	Foundation	-22.11	-72.63	-27.37
1001	Stilt Floor	-18.69	-66.36	-28.04
1002	1st Floor	-28.95	-58.77	-30.70
1003	2nd Floor	-29.52	-49.52	-30.48
1004	3rd Floor	-27.17	-40.22	-26.09
1005	4th Floor	-21.62	-25.68	-17.57
1006	5th Floor	-13.46	0.00	-7.69
1007	6th Floor	-3.57	42.86	21.43

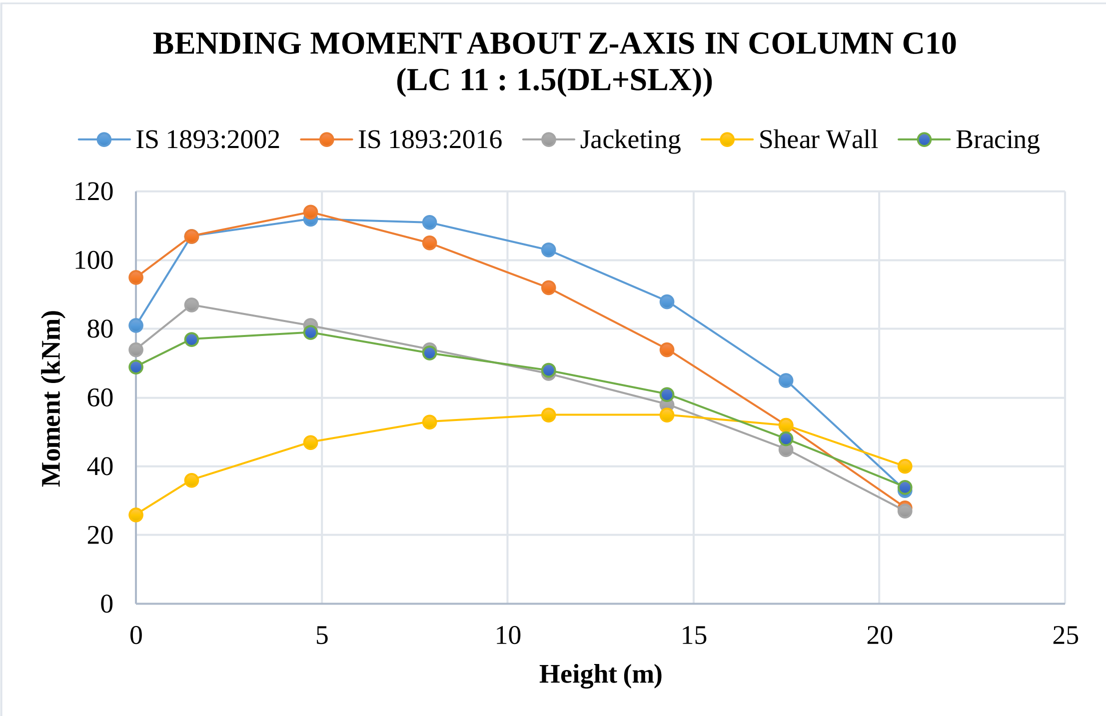


Fig.4.33 Bending Moment about Z-Axis in column C-10

- Bending Moment about Y-Axis (Load combination : LC 12 = 1.5 (DL + SLZ))

Table 4.33(a) – Bending Moment (kNm) about Y-Axis in column C-1

Member No.	Storey Level	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
100	Foundation	132	181	143	131	114
101	Stilt Floor	100	118	98	84	77
102	1st Floor	40	117	110	98	89
103	2nd Floor	41	121	106	98	88
104	3rd Floor	37	117	100	94	85
105	4th Floor	28	109	95	93	85
106	5th Floor	19	95	85	86	81
107	6th Floor	55	86	74	81	76

Table 4.33(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 model in Column C-1

Member No.	Storey Level	Jacketing (%)	Shear Wall (%)	Bracing (%)
100	Foundation	-20.99	-27.62	-37.02
101	Stilt Floor	-16.95	-28.81	-34.75
102	1st Floor	-5.98	-16.24	-23.93
103	2nd Floor	-12.40	-19.01	-27.27
104	3rd Floor	-14.53	-19.66	-27.35
105	4th Floor	-12.84	-14.68	-22.02
106	5th Floor	-10.53	-9.47	-14.74
107	6th Floor	-13.95	-5.81	-11.63

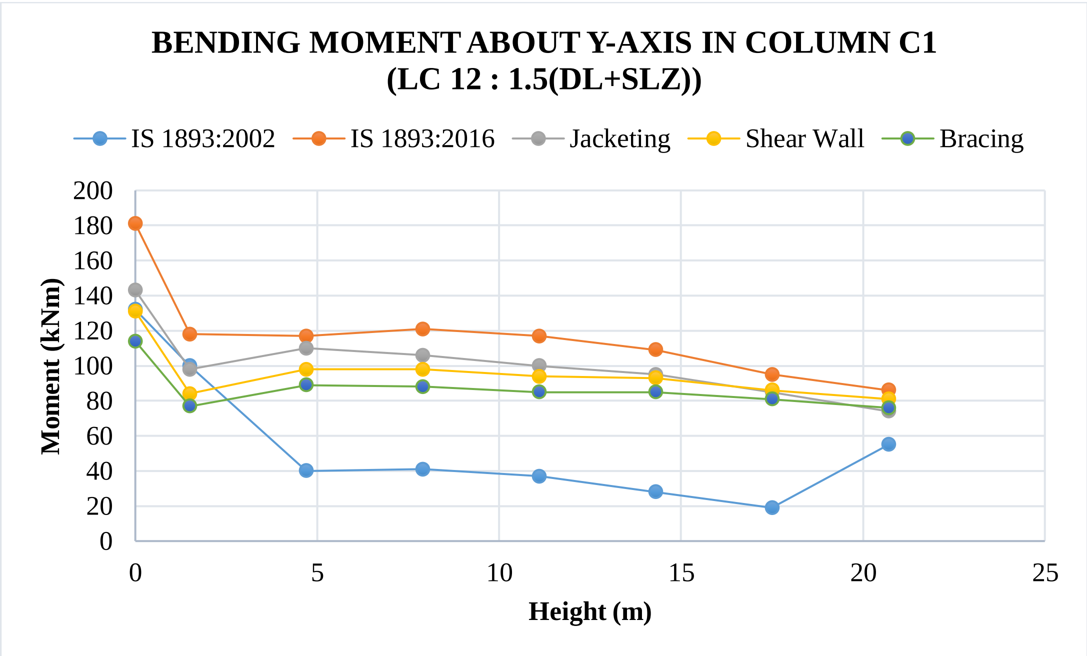


Fig.4.34 Bending Moment about Y-Axis in column C-1

Table 4.34(a) – Bending Moment (kNm) about Y-Axis in Column C-2

Member No.	Storey Level	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
200	Foundation	80	99	196	72	62
201	Stilt Floor	122	116	171	88	74
202	1st Floor	144	151	164	119	94
203	2nd Floor	138	135	141	110	82
204	3rd Floor	128	117	125	104	75
205	4th Floor	110	93	103	94	65
206	5th Floor	80	64	81	74	47
207	6th Floor	43	37	35	52	26

Table 4.34(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 model in Column C-2

Member No.	Storey Level	Jacketing (%)	Shear Wall (%)	Bracing (%)
200	Foundation	97.98	-27.27	-37.37
201	Stilt Floor	47.41	-24.14	-36.21
202	1st Floor	8.61	-21.19	-37.75
203	2nd Floor	4.44	-18.52	-39.26
204	3rd Floor	6.84	-11.11	-35.90
205	4th Floor	10.75	1.08	-30.11
206	5th Floor	26.56	15.63	-26.56
207	6th Floor	-5.41	40.54	-29.73

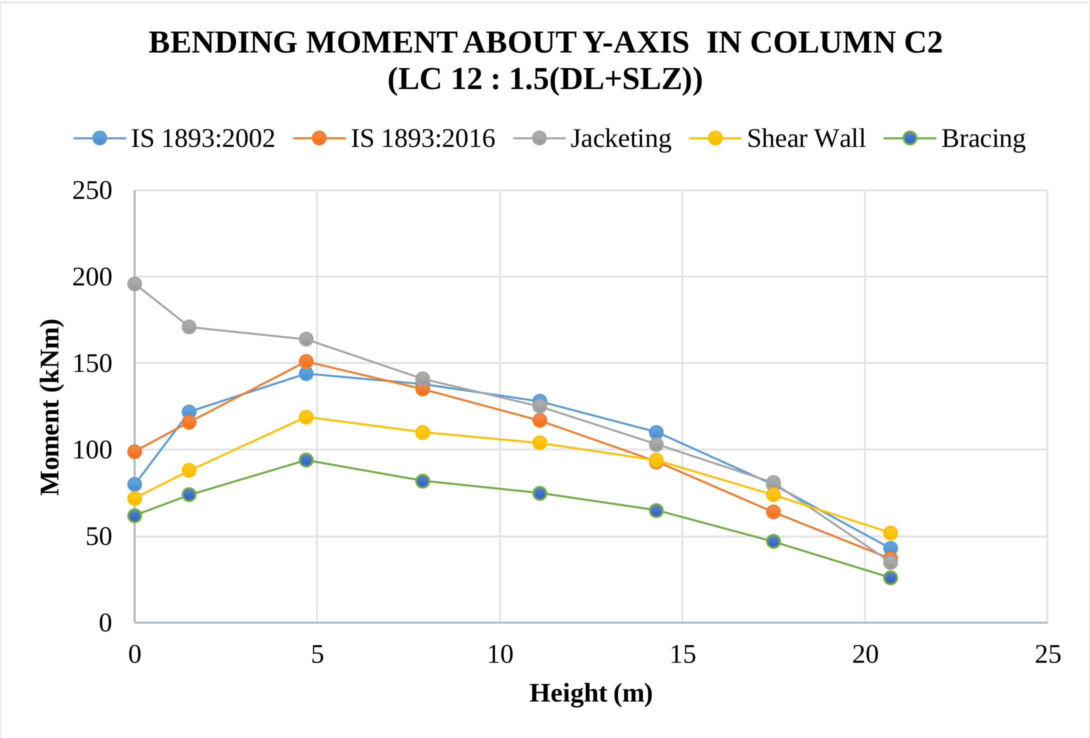


Fig.4.35 Bending Moment about Y-Axis in column C-2

Table 4.35(a) – Bending Moment (kNm) about Y-Axis in column C-3

Member No.	Storey Level	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
300	Foundation	86	105	87	72	67
301	Stilt Floor	116	122	106	78	79
302	1st Floor	132	147	123	103	90
303	2nd Floor	130	136	111	95	83
304	3rd Floor	120	119	99	85	75
305	4th Floor	102	95	85	73	65
306	5th Floor	72	66	64	52	49
307	6th Floor	32	34	37	24	23

Table 4.35(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 model in Column C-3

Member No.	Storey Level	Jacketing (%)	Shear Wall (%)	Bracing (%)
300	Foundation	-17.14	-31.43	-36.19
301	Stilt Floor	-13.11	-36.07	-35.25
302	1st Floor	-16.33	-29.93	-38.78
303	2nd Floor	-18.38	-30.15	-38.97
304	3rd Floor	-16.81	-28.57	-36.97
305	4th Floor	-10.53	-23.16	-31.58
306	5th Floor	-3.03	-21.21	-25.76
307	6th Floor	8.82	-29.41	-32.35

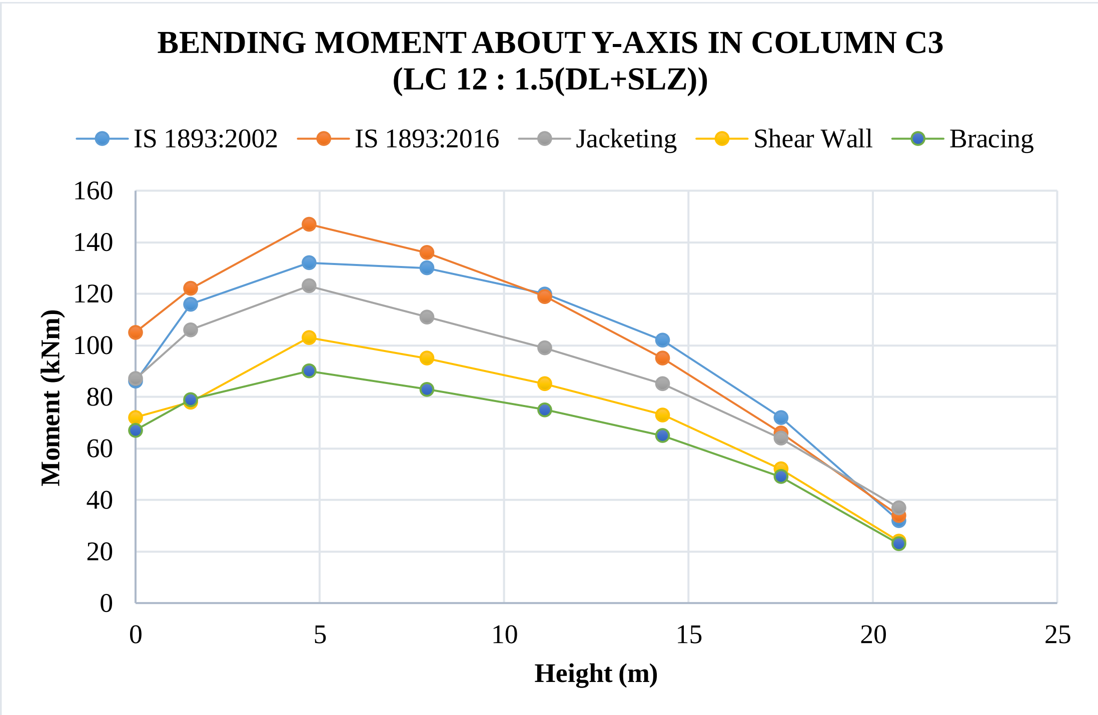


Fig.4.36 Bending Moment about Y-Axis in column C-3

Table 4.36(a) – Bending Moment (kNm) about Y-Axis in column C-5

Member No.	Storey Level	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
500	Foundation	131	165	341	130	123
501	Stilt Floor	164	173	271	150	132
502	1st Floor	175	167	227	145	134
503	2nd Floor	162	134	196	120	110
504	3rd Floor	155	117	181	115	106
505	4th Floor	142	96	158	104	97
506	5th Floor	114	73	119	83	79
507	6th Floor	84	61	75	68	68

Table 4.36(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 model in Column C-5

Member No.	Storey Level	Jacketing (%)	Shear Wall (%)	Bracing (%)
500	Foundation	106.67	-21.21	-25.45
501	Stilt Floor	56.65	-13.29	-23.70
502	1st Floor	35.93	-13.17	-19.76
503	2nd Floor	46.27	-10.45	-17.91
504	3rd Floor	54.70	-1.71	-9.40
505	4th Floor	64.58	8.33	1.04
506	5th Floor	63.01	13.70	8.22
507	6th Floor	22.95	11.48	11.48

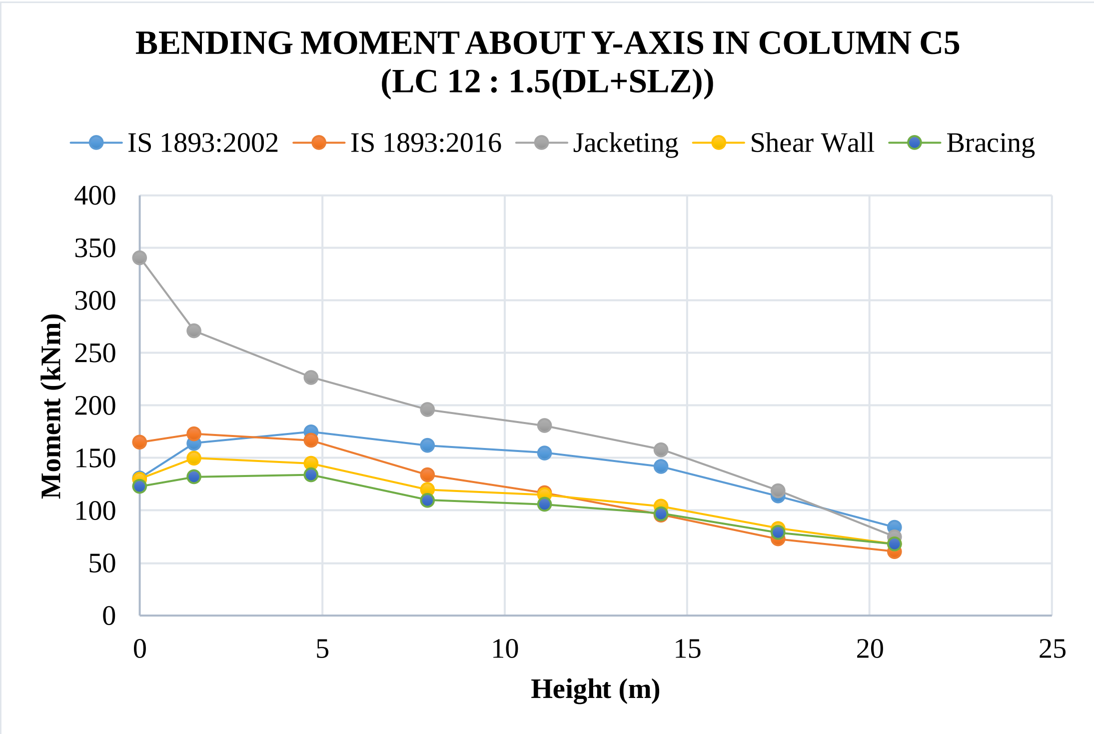


Fig.4.37 Bending Moment about Y-Axis in column C-5

Table 4.37(a) – Bending Moment (kNm) about Y-Axis in column C-6

Member No.	Storey Level	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
600	0.00	83	95	87	72	71
601	1.50	75	80	74	53	58
602	4.70	22	29	40	9	24
603	7.90	22	28	37	8	20
604	11.10	23	26	34	8	18
605	14.30	22	23	30	7	18
606	17.50	21	18	25	6	17
607	20.70	7	8	16	7	9

Table 4.37(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 model in Column C-6

Member No.	Storey Level	Jacketing (%)	Shear Wall (%)	Bracing (%)
600	Foundation	-8.42	-24.21	-25.26
601	Stilt Floor	-7.50	-33.75	-27.50
602	1st Floor	37.93	-68.97	-17.24
603	2nd Floor	32.14	-71.43	-28.57
604	3rd Floor	30.77	-69.23	-30.77
605	4th Floor	30.43	-69.57	-21.74
606	5th Floor	38.89	-66.67	-5.56
607	6th Floor	100.00	-12.50	12.50

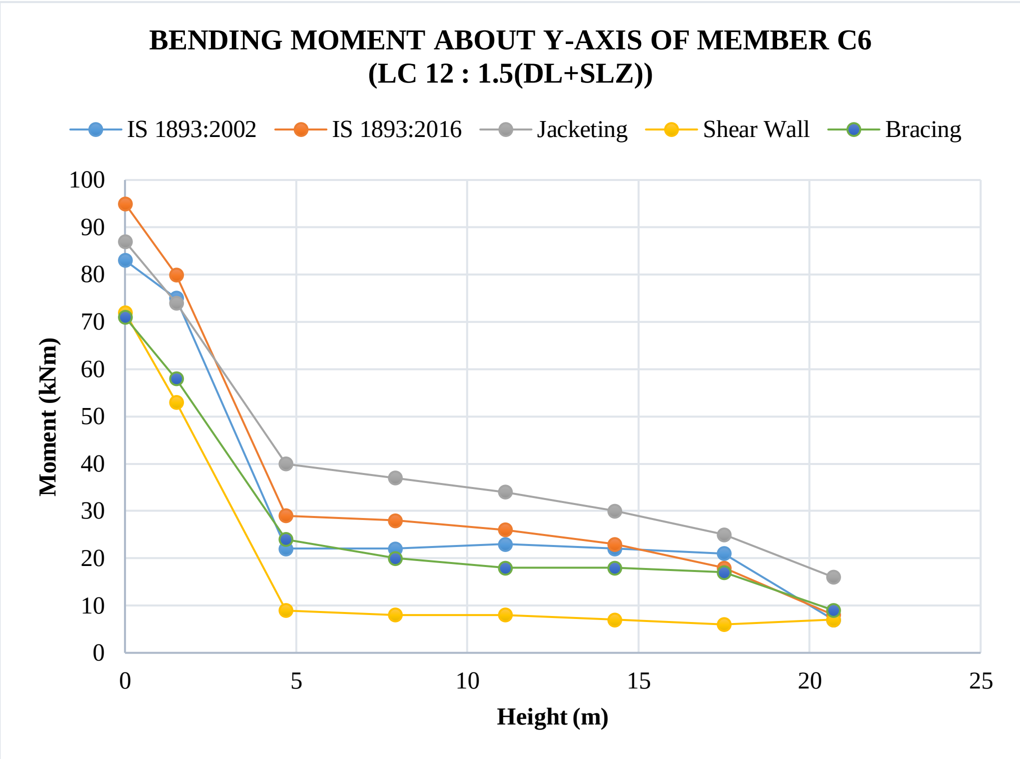


Fig.4.38 Bending Moment about Y-Axis in column C-6

Table 4.38(a) – Bending Moment (kNm) about Y-Axis in column C-8

Member No.	Storey Level	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
800	Foundation	84	98	82	89	83
801	Stilt Floor	104	106	92	99	92
802	1st Floor	98	93	82	87	81
803	2nd Floor	98	87	78	82	76
804	3rd Floor	93	77	73	75	72
805	4th Floor	81	62	66	65	65
806	5th Floor	63	46	54	51	51
807	6th Floor	36	27	36	30	31

Table 4.38(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 model in Column C-8

Member No.	Storey Level	Jacketing (%)	Shear Wall (%)	Bracing (%)
800	Foundation	-16.33	-9.18	-15.31
801	Stilt Floor	-13.21	-6.60	-13.21
802	1st Floor	-11.83	-6.45	-12.90
803	2nd Floor	-10.34	-5.75	-12.64
804	3rd Floor	-5.19	-2.60	-6.49
805	4th Floor	6.45	4.84	4.84
806	5th Floor	17.39	10.87	10.87
807	6th Floor	33.33	11.11	14.81

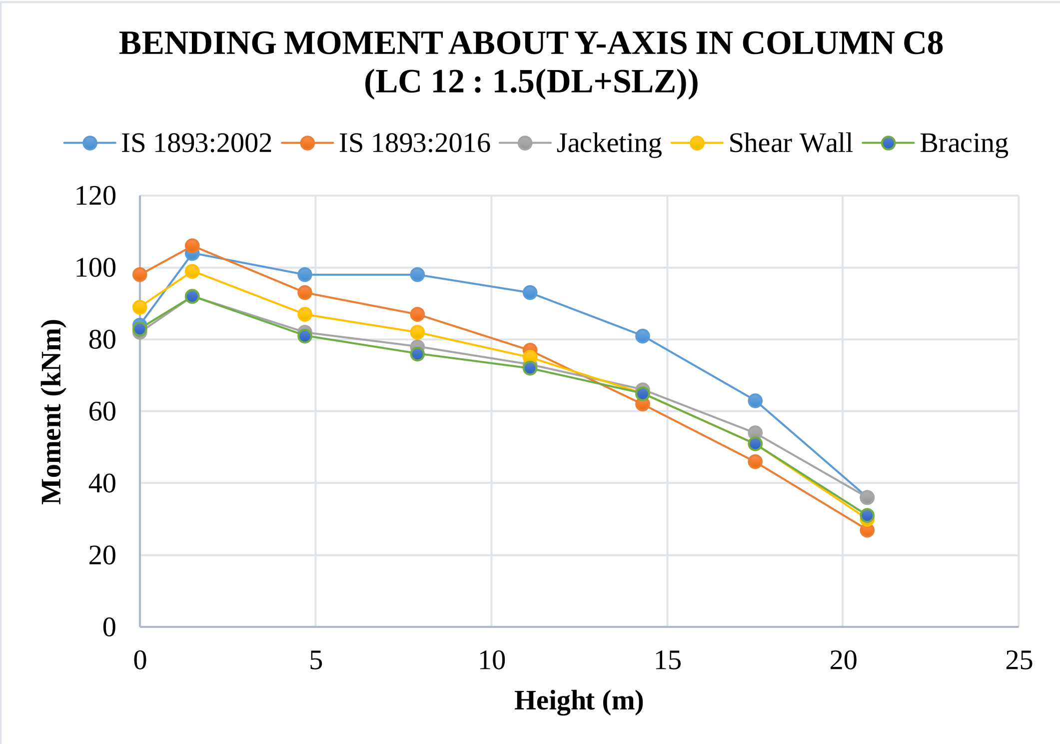


Fig.4.39 Bending Moment about Y-Axis in column C-8

Table 4.39(a) – Bending Moment (kNm) about Y-Axis in column C-10

Member No.	Storey Level	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
1000	Foundation	147	181	155	141	38
1001	Stilt Floor	115	133	114	101	23
1002	1st Floor	86	133	125	111	32
1003	2nd Floor	85	130	114	104	32
1004	3rd Floor	79	119	102	94	33
1005	4th Floor	67	103	91	83	33
1006	5th Floor	48	83	82	73	34
1007	6th Floor	25	51	59	48	27

Table 4.39(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 model in Column C-10

Member No.	Storey Level	Jacketing (%)	Shear Wall (%)	Bracing (%)
1000	Foundation	-14.36	-22.10	-79.01
1001	Stilt Floor	-14.29	-24.06	-82.71
1002	1st Floor	-6.02	-16.54	-75.94
1003	2nd Floor	-12.31	-20.00	-75.38
1004	3rd Floor	-14.29	-21.01	-72.27
1005	4th Floor	-11.65	-19.42	-67.96
1006	5th Floor	-1.20	-12.05	-59.04
1007	6th Floor	15.69	-5.88	-47.06

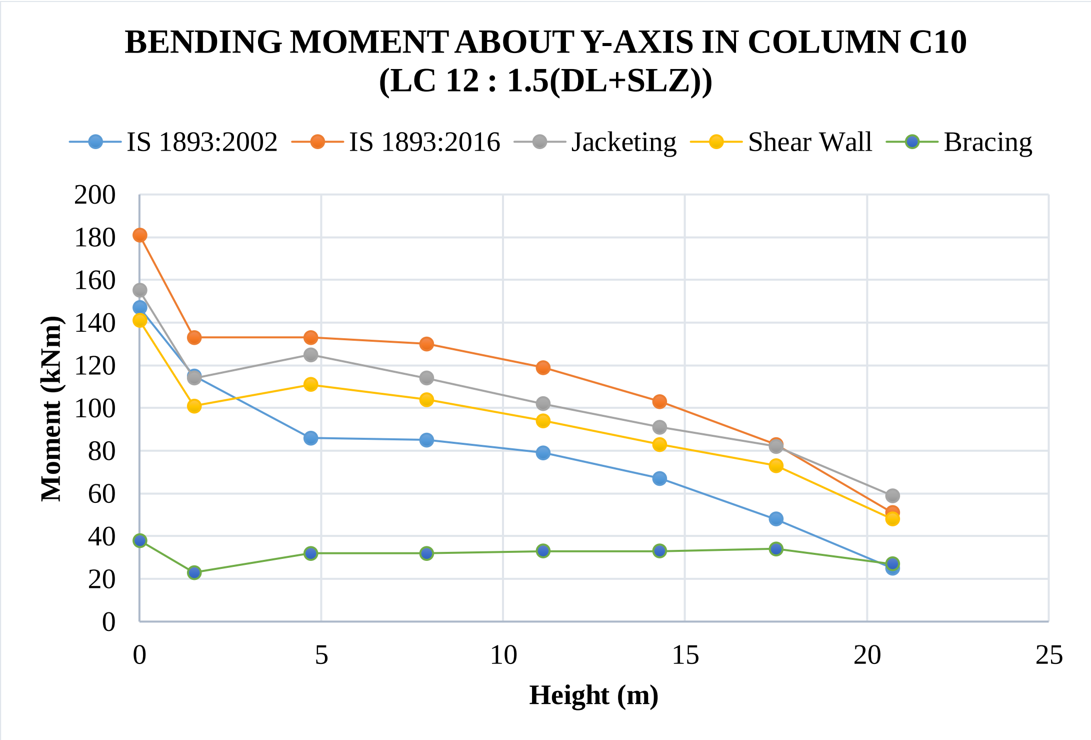


Fig.4.40 Bending Moment about Y-Axis in column C-10

From the above analysis of bending moment in columns following observations were made:

- In Model 3 i.e. model with jacketing, the columns which were jacketed attract more forces towards them and hence bending moment in those columns increases.
- In Model 4 i.e. model with shear wall, the columns showed an avg decrease in bending moment of 30-50% because the columns situated near the wall or in line with wall experienced less forces because the shear wall attracts most of the forces in its direction.
- In Model 5 i.e. model with steel bracing, the columns showed an avg decrease in bending moment of 20-40% because the columns situated near the wall or in line with wall experienced less forces because steel bracing attracts the forces in its direction.

4.6 BENDING MOMENT IN BEAMS

Bending moment refers to the internal forces and moments experienced by a beam due to external loads, mostly due external lateral loads applied to it. Beams are horizontal structural members designed to primarily resist bending forces. When loads are applied to a beam, they generate bending moments that cause the beam to deform and induce tensile and compressive stresses along its cross-section. It varies along its length and is influenced by factors such as the magnitude and distribution of the applied loads, the beam's span, and its support conditions. The maximum bending moment usually occurs at the point where the applied load or loads cause the greatest bending effect on the beam.

Analyzing the bending moment in beams is crucial for structural design. Maximum bending moment helps to determine the required beam size, material, and reinforcement. By ensuring that the beam can withstand the anticipated bending stresses without exceeding its strength and deflection limits, hence ensure the structural integrity and safety of the overall system.

In this study the main beam members were studied and analysed in all the models for the most critical load case. Beams highlighted i.e. B1, B2 & B4 also shown in fig.4.41 below are analysed and their results are shown in table 4.40-4.42 and fig.4.42-4.44:

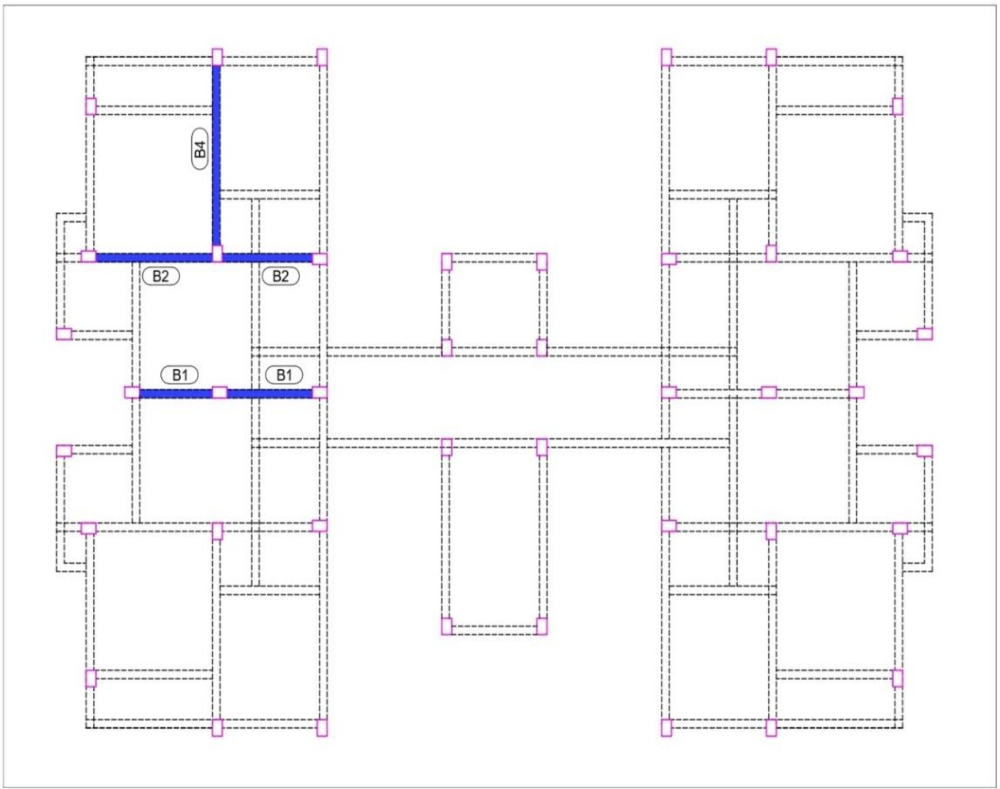


Fig.4.41 Plan highlighting the beams selected for results

Table 4.40(a)- Bending Moment (kNm) in Beam B1

Beam No. - Floor Lvl	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
B1-1	249	249	277	96	192
B1-3	224	223	226	47	185
B1-7	35	73	69	18	76

Table 4.40(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 Model in Beam B1

Beam No. - Floor Lvl	Jacketing (%)	Shear Wall (%)	Bracing (%)
B1-1	11.24 %	-61.45 %	-22.89 %
B1-3	1.35 %	-78.92 %	-17.04 %
B1-7	-5.48 %	-75.34 %	4.11 %

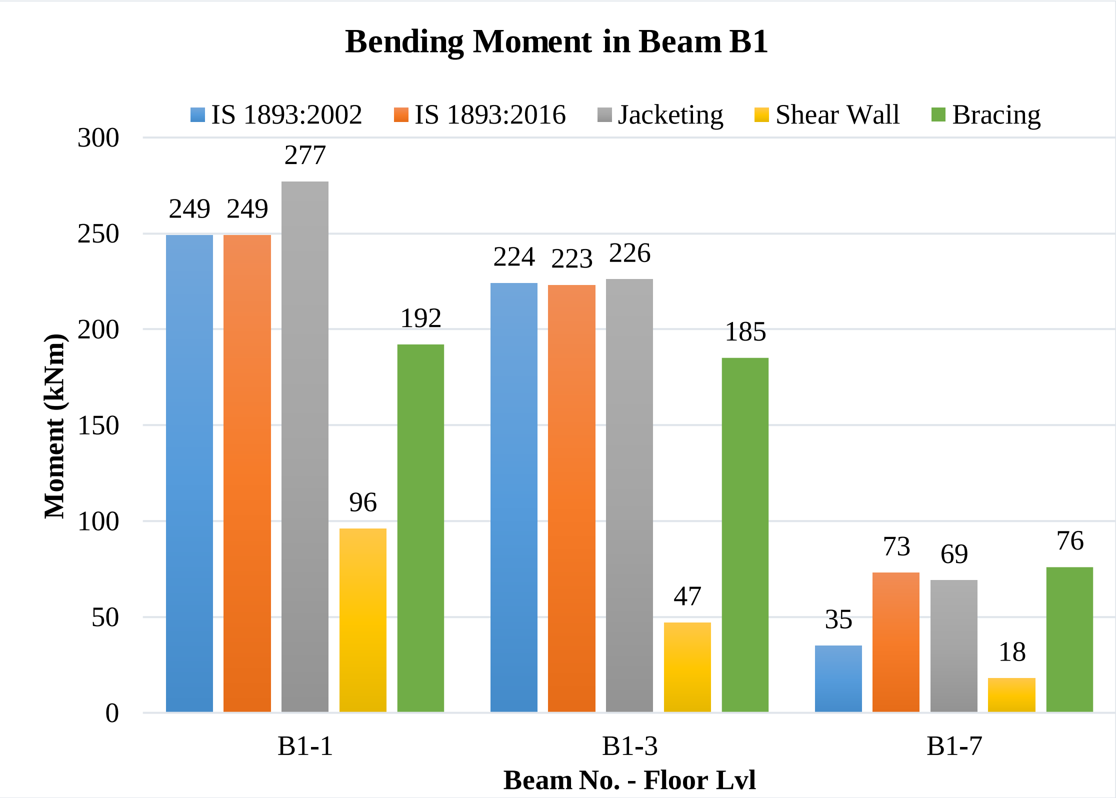


Fig.4.42 Bending Moment in Beam B1

Table 4.41(a) - Bending Moment (kNm) in Beam B2

Beam No. - Floor Lvl	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
B2-1	244	228	335	71	160
B2-3	234	211	287	103	159
B2-7	56	77	76	131	76

Table 4.41(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 Model in Beam B2

Beam No. - Floor Lvl	Jacketing (%)	Shear Wall (%)	Bracing (%)
B2-1	46.93 %	-68.86 %	-29.82 %
B2-3	36.02 %	-51.18 %	-24.64 %
B2-7	-1.30 %	70.13 %	-1.30 %

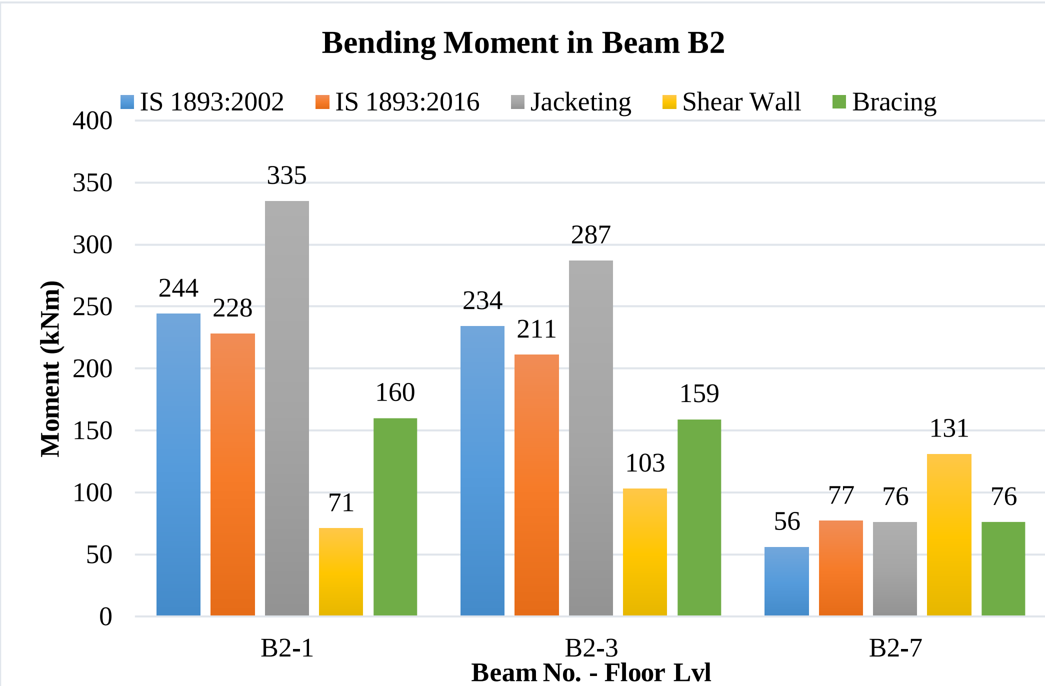


Fig.4.43 Bending Moment in Beam B2

Table 4.42(a) - Bending Moment (kNm) in Beam B4

Beam No. - Floor Lvl	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
B4-1	153	151	156	110	144
B4-3	160	155	154	146	157
B4-7	89	92	89	91	104

Table 4.42(b) - Percentage change in Bending Moment w.r.t. IS1893:2016 Model in Beam B4

Beam No. - Floor Lvl	Jacketing (%)	Shear Wall (%)	Bracing (%)
B4-1	3.31 %	-27.15 %	-4.64 %
B4-3	-0.65 %	-5.81 %	1.29 %
B4-7	-3.26 %	-1.09 %	13.04 %

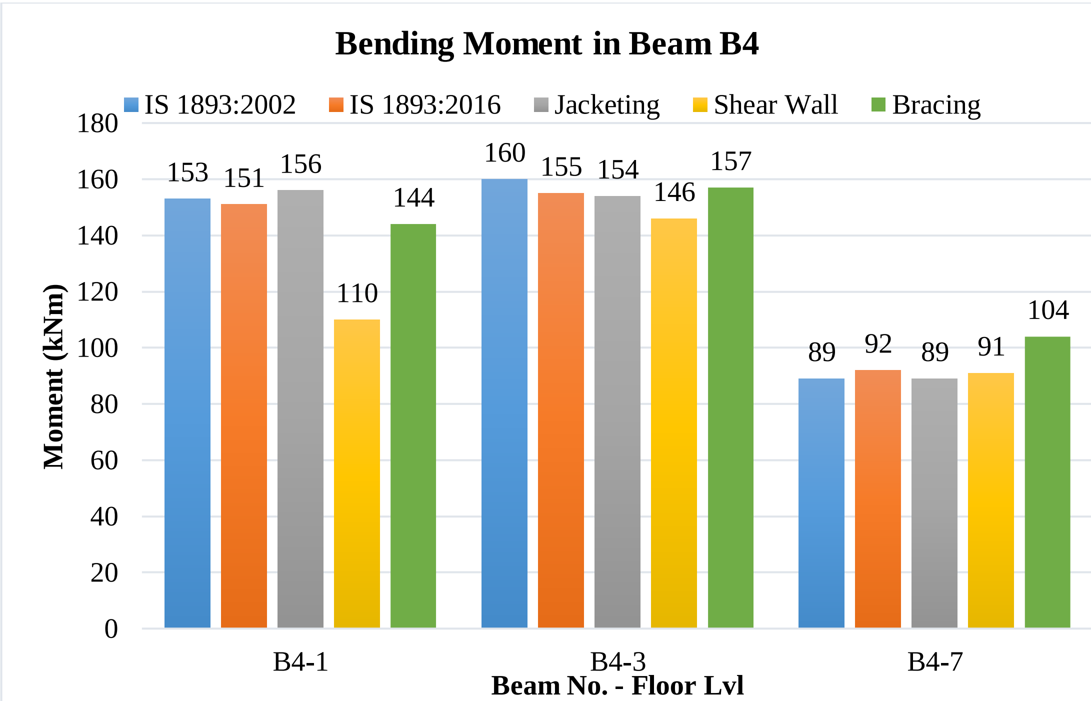


Fig.4.44 Bending Moment in Beam B4

- In Model 3, Bending Moment increased for all the three beams, B2 had the maximum increase of about 45% as its size was increased leading to more force carrying ability and hence more moment in the beam. Similar was the case with Beam B4 with an increase of 5%. Beam B1, even though was not jacketed, showed an increase of 10 % in bending moment.
- In Model 4, there was mostly a decrease in moment of the beams by 30 – 60% due to the fact shear wall attracting most of the forces and the beams experiencing little force and hence less bending moment.
- In Model 5, a similar trend was seen as Model 4, the decrease in moment was from 10-30% as the bracing acts as ties between the columns as well and therefore reducing the forces on beams.

4.7 SHEAR FORCE IN BEAMS

Shear force in beams refers to the internal force that acts parallel to the cross-section of a beam. It arises from the vertical loads applied to the beam and can vary along its length. Shear force is responsible for the transfer of loads between different sections of a beam.

The magnitude and distribution of shear force in a beam depend on various factors, including the applied loads, their positions along the beam, and the beam's support conditions. At any given section of the beam, the shear force is the algebraic sum of the vertical forces acting on one side of the section.

Analyzing the shear force in beams is crucial for structural design. The maximum shear force is used to determine the required beam size, reinforcement, and support conditions. By ensuring that the beam can withstand the anticipated shear forces without failure, one can ensure the structural integrity and safety of the overall system. Shear forces in beams can induce shear stresses along the cross-section, which are perpendicular to the direction of the shear force. These stresses need to be considered when designing the beam's reinforcement or selecting the appropriate beam shape and material.

To resist shear forces effectively, engineers employ various design strategies. These may include using shear reinforcement such as stirrups or bent-up bars, which are placed in the beam's cross-section to enhance its shear capacity. The choice of beam shape, such as I-sections or T-sections, can also influence the beam's ability to resist shear forces. Shear force diagrams help in identifying critical sections where the shear force is maximum or changes sign, which can guide the selection of reinforcement and support conditions. Therefore, shear force in beams is a significant consideration in structural engineering. Understanding and analyzing the shear forces allows to design beams that can effectively transfer loads and resist shear stresses. Proper selection of beam size, reinforcement, and support conditions ensures the structural integrity and safety of the overall system, leading to reliable and durable structures. In this study beams highlighted B1, B2 & B4 shown in fig.4.45 below, their results are discussed in Table 4.43-4.45 and Fig. 4.46-4.48:

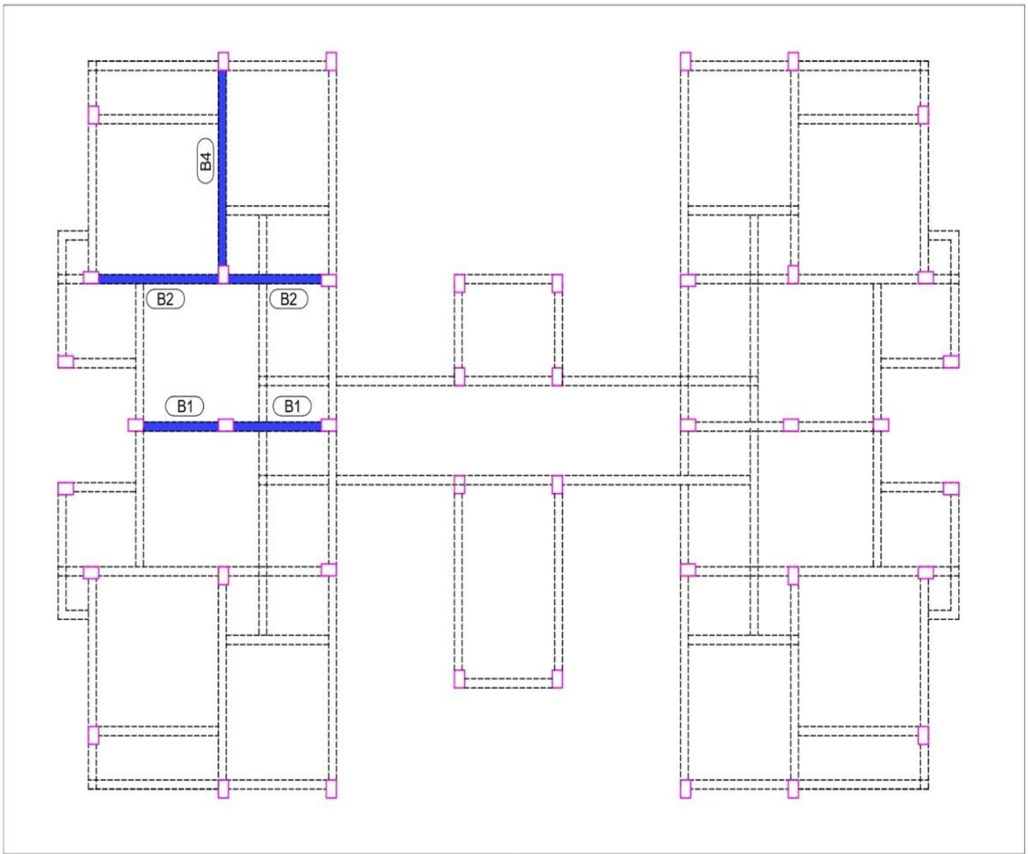


Fig.4.45 Plan highlighting the beams selected for results

Table 4.43(a) - Shear Force (kN) in Beam B1

Beam No. - Floor Lvl	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
B1-1	250	219	237	78	190
B1-3	218	210	218	49	185
B1-7	62	93	94	24	95

Table 4.43(b) - Percentage change in Shear Force w.r.t. IS1893:2016 Model in Beam B1

Beam No. - Floor Lvl	Jacketing (%)	Shear Wall (%)	Bracing (%)
B1-1	7.80 %	-64.30 %	-13.60 %
B1-3	3.88 %	-76.42 %	-11.68 %
B1-7	1.81 %	-74.20 %	3.11 %

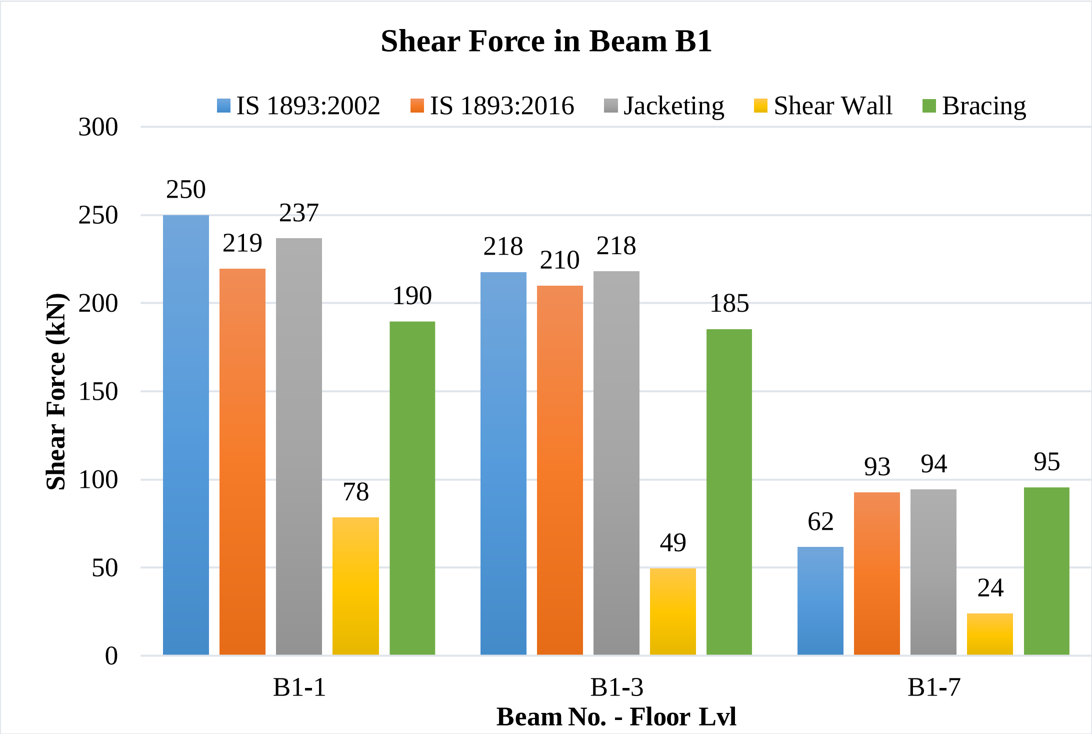


Fig.4.46 Shear Force in Beam B1

Table 4.44(a) - Shear Force (kN) in Beam B2

Beam No. - Floor Lvl	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
B2-1	174	191	254	68	156
B2-3	171	165	208	63	137
B2-7	58	76	78	120	75

Table 4.44(b) - Percentage change in Shear Force w.r.t. IS1893:2016 Model in Beam B2

Beam No. - Floor Lvl	Jacketing (%)	Shear Wall (%)	Bracing (%)
B2-1	33.07 %	-64.68 %	-18.63 %
B2-3	26.07 %	-61.52 %	-16.82 %
B2-7	2.88 %	58.13 %	-1.66 %

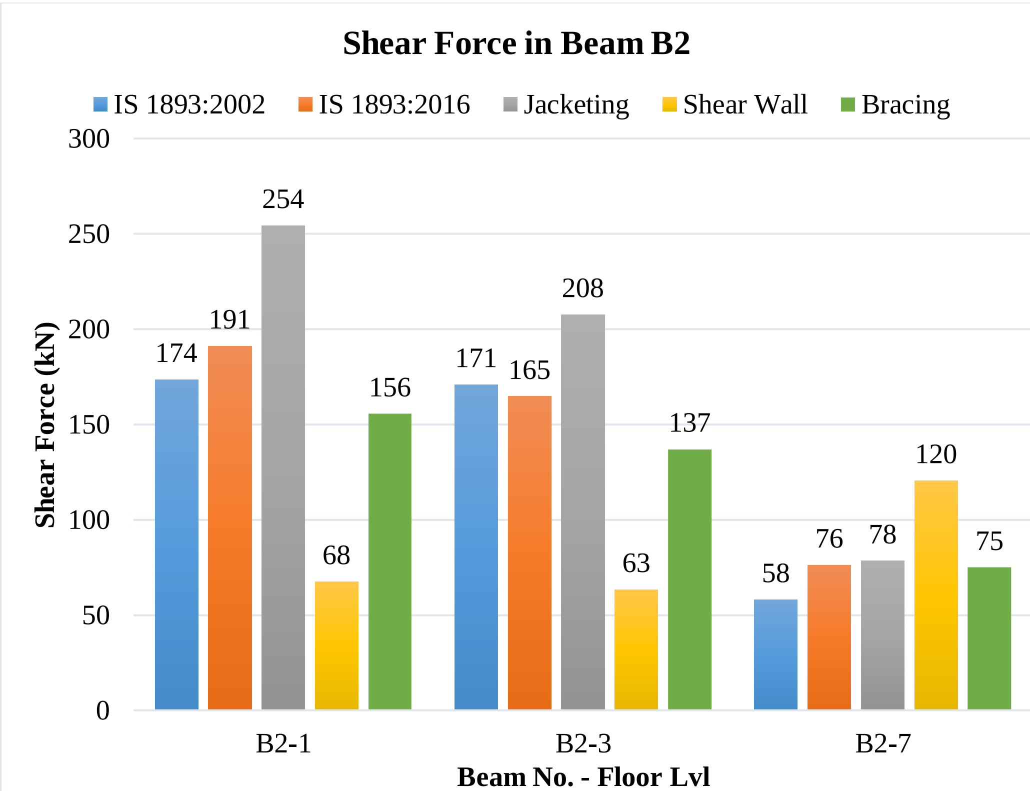


Fig.4.47 Shear Force in Beam B2

Table 4.45(a) - Shear Force (kN) in Beam B4

Beam No. - Floor Lvl	IS 1893:2002	IS 1893:2016	Jacketing	Shear Wall	Bracing
B4-1	163	153	154	107	141
B4-3	162	150	149	135	145
B4-7	97	97	98	83	101

Table 4.45(b) - Percentage change w.r.t. IS1893:2016 Model in Beam B4

Beam No. - Floor Lvl	Jacketing (%)	Shear Wall (%)	Bracing (%)
B4-1	0.99 %	-29.65 %	-7.33 %
B4-3	-0.48 %	-10.08 %	-3.63 %
B4-7	1.36 %	-13.84 %	4.26 %

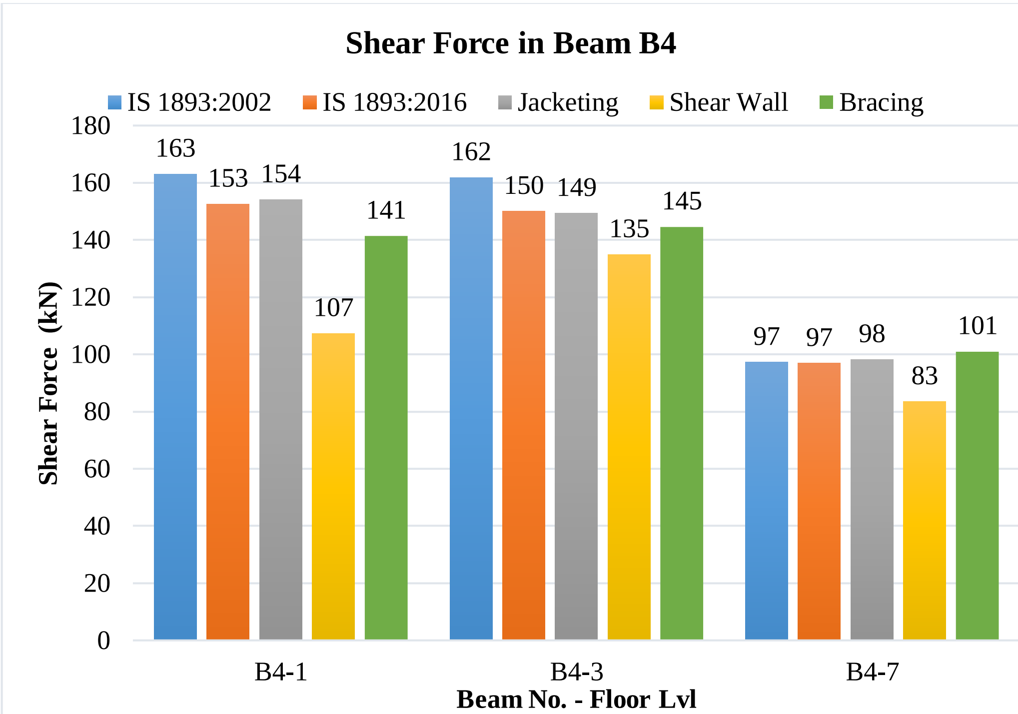


Fig.4.48 Shear Force in Beam B4

- In Model 3, Shear Force is increased for all the three beams, B2 had the maximum increase of about 30% as its size was increased leading to more load carrying capacity and hence more forces acting on it.

- In Model 4, there was mostly a decrease in shear force of the beams by 30 – 60% due to the fact shear wall attracting most of the forces in the structure.
- In Model 5, a similar trend was seen as Model 4, the decrease in shear was from 10-20% as the bracing acts as ties between the columns as well and therefore reducing the forces on beams.

5. CONCLUSIONS

In this study, dynamic analysis of an existing structure has been carried out following the IS Seismic code:2002 and its behaviour is compared after analysing it using latest revision of IS:1893 of 2016 by modelling it in software Staad Pro. It is observed as follows:

- The structural design of the existing structure did not comply with some of the clauses of the latest IS Seismic code and failure of some structural members was seen.
- Therefore, different local and global retrofitting techniques such as Jacketing of distressed members, provision of Shear Wall and Steel Bracing were done and studied.
- Dynamic Base Shear and Multiplying Factor Vb/VB were not calculated in Model-1 as static analysis was performed. In dynamic analysis, it is observed that Design Shear increased in the retrofitted models by 2-5 %. Basically, due to the increase in seismic weight as additional members were added resulting in increase in lateral force resistance of the building and resulted in increase of base shear.
- In Model with jacketing, the members showed a decrease of 10-30 % in lateral displacement due to increase in size the stiffness of the structure increased. The columns which were jacketed attracted more forces and hence bending moment in columns increased by 10-15%. Similarly, there was increase in Bending Moment and Shear Force for jacketed beams.
- In Model with shear wall, the members showed a decrease in lateral displacement of 30-50% as shear wall resists significant lateral forces. The columns showed decrease in bending moment of 30-5% as the columns situated near the wall or in line with wall experienced less forces as shear wall attracts most of the lateral forces. Similary, in beams close proximity to shear wall bending moment and shear force decreased by 30-60% and 25-50% respectively.
- In Model with steel bracing, members showed a decrease in lateral displacement of 40-60%. The columns had a decrease in bending moment of 20-40%. It is due to the fact that bracing acts as connection between columns, it provides resistance both in lateral and vertical direction hence decreasing the lateral displacement and moment in columns. Similarly beams experienced a decrease in Bending Moment and Shear force of 10-30% and 10-20% respectively.

Thus finally it is seen that existing structure designed as per IS seismic code 2002 was found to be deficient as per IS seismic code 2016 and needs retrofitting. The retrofitting techniques can be global or local as per the requirement.

6. FUTURE SCOPE OF RESEARCH

The retrofitting of existing structures is becoming increasingly important in the field of civil engineering due to the need for sustainable and resilient infrastructure. As the world continues to face challenges from natural disasters, aging buildings, and changing climate conditions, research on retrofitting techniques and technologies plays a vital role in enhancing the safety, durability, and performance of existing structures.

- Advancements in Retrofitting Materials and Techniques:

Future research is likely to concentrate on developing advanced retrofitting materials and techniques. Researchers will explore new construction materials, composites, and innovative technologies to improve the seismic performance, and durability of retrofitted structures. Additionally, the investigation of cost-effective and sustainable materials will be crucial in achieving widespread adoption of retrofitting practices.

- Integration of Smart Technologies:

The integration of smart technologies will revolutionize the retrofitting process. Researchers will explore the use of sensors, monitoring systems, and data analytics to assess structural health, predict potential vulnerabilities, and monitor retrofitting effectiveness in real-time. Smart technologies will enable more proactive and adaptive retrofitting practices, ensuring the ongoing safety and resilience of retrofitted structures.

- Retrofitting for Climate Change Adaptation:

With the increasing frequency of extreme weather events and changing climate patterns, retrofitting for climate change adaptation will be a significant area of research. Studies will focus on retrofitting techniques that enhance buildings' resilience against floods, storms, rising sea levels, and other climate-related hazards.

- Retrofitting of Historic and Heritage Structures:

Preservation of historic and heritage structures will be a critical aspect of future retrofitting research. Special attention will be given to developing retrofitting techniques that maintain the historical and architectural integrity of these structures while improving their structural performance and safety.

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1 1. INTRODUCTION 1.1 GENERAL 1.1.1 Framed Structure A RC framed building consists of framework of horizontal and vertical members joined together by rigid connections. The entire structure is a monolithic construction, ensuring rigidity to the connections. The building frame is subjected to gravity loads e.g. dead load, live load and lateral loads due to seismic and wind action. The lateral loads are primarily resisted by the vertical members of the frame like columns, shear walls etc. These lateral loads resisting elements are the most critical members in the frame. The complete framework of beam and slab is integrated with these vertical members of the frame and thus make it behave as a single unit. (Fig. 1.1) Fig. 1.1 Typical arrangement of members in RC framed building (Ref: www.civildigital.com)