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Influence of Positioning of Shear Wall on the Torsional Response of Building

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Abstract:** It would be ideal if all buildings have their lateral-load resisting elements symmetrically arranged and earthquake ground motions would strike in known directions. Due to scarcity of land in big cities, architects often propose irregular buildings in order to utilize maximum available land area and to provide adequate ventilation and light in various building components. However, it is quite often that structural irregularity is the result of a combination of both types. Most buildings have some degree of irregularity in the geometric configuration or the distribution of mass, stiffness, and/or strength. Due to one or more of these asymmetries, thestructure's lateral resistance to the ground motion is usually torsionally unbalanced creating large displacement amplifications and high force concentrations within the resisting elements which can cause severe damages and at times collapse of the structure. Eccentric arrangement of non-structural components, asymmetric yielding, presence of rotational component in ground motions and the variations in the input energy imparted by the ground motions also contribute significantly to the torsional response of buildings. So, this research work demonstrates the importance oflocation of shear wall is should be checked before the sequential failure defined by the Response spectrum analysis method. In Numerical Tool like SAP-2000 which are uses worldwide for pushover analysis method.

I. INTRODUCTION

It would be ideal if all buildings have their lateral-load resisting elements symmetrically arranged, and earthquakewaves would act in known directions. Due to scarcity of land in Metro cities, like Delhi, Mumbai architects many times propose irregular buildings to utilize the maximum available property and to provide adequate ventilation and light in various building components. Most buildings have some degree of irregularity in the geometric configuration or the distribution of mass, stiffness, and strength. Due to one or more of these irregularities, the structure's lateral resistance to the ground motion is usually torsionally unbalanced, creating massive displacement and high force concentrations within the resisting elements, which can cause severe damages and most of the times collapse of the structure. Eccentric arrangement of nonstructural components, unsymmetrical stiffness, asymmetric yielding, presence of a rotational element in ground motions, and the variations in the input energy imparted by the ground motions also contribute significantly to the torsional response of buildings. In India, the failure of the two most famous apartments during the 2001 Bhuj earthquake was noted due to torsional response.

In the last few years, shear walls became an essential part ofmid and high-rise structures. As a component of an earthquake-resistant building, shear walls are located in buildings, reducing lateral displacements under earthquake loads. So, shear-wall frame structures are obtained. Shear wall type buildings are generally regular in elevation and plan also.

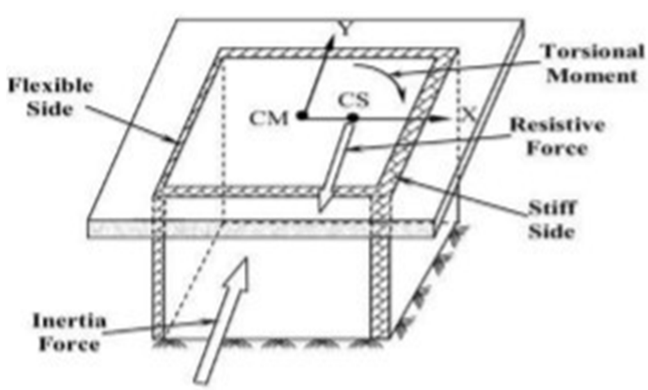

Figure 1 - Generation of the torsional moment inasymmetric structures during seismic excitation(Courtesy internet)

A. Purpose Of Constructing Shear Walls

Shear walls designed to resist lateral loads due to earthquakes and wind. The walls are structurally connected with diaphragms, and other lateral walls at right angles, therefore, give stability to the building structures. Shear wall structural systems are more stable during earthquakes than RCC framed structures. Shear walls also have to resist the uplift forces generated by the pull of the wind. Walls have to resist shear forces that try to push the walls. Shear walls are used to sell all torsional modes and diagonal translational modes to possess a natural time period outside the range of 0.04 to 2 sec by increasing the torsional stiffness of the building. Walls have to resist the lateral force of the wind that tries to push the walls in and pull them away from the building. These walls will attract shear forces and will prevent changing locations and positions of construction and consequently, destruction. Shearwall resists the lateral forces by combined axial-flexure-shear action. Construction of shear wall in talland even short buildings will strengthenthe structure significantly, and either more economical than the bending frames.

B. Need of This Research Work

It will be ideal if all buildings have their lateral-load resisting elements symmetrically arranged, and earthquake ground motions would act in known directions. Due to the scarcity of land in metro cities, architects often propose irregular buildings in order to utilize the maximum available land area and to provide adequate ventilation andlight in various building components. However, it is quite often that structural irregularity is the result of a combination of both types. Most buildings have some degree of irregularity in the geometric configuration or the distribution of mass, stiffness, and strength. Due to one or more of these asymmetries, the structure's lateral resistance to the ground motion is usually torsionally unbalanced, creating significant displacement amplification and high force concentrations within the resisting elements like columns and shear walls which can cause severe damages and at times collapse of the structure. So this research work demonstrates the importance of the location of the shear wall it should be checked before the sequential failure defined by the pushover analysis method. The Numerical Tool like ETAB2017 and SAP-2000, which are uses worldwide for the pushover analysis method

II. LITERATURE REVIEW

Rajlaxmi K. R. Harinarayanan S. (2015) has carried out anon-linear dynamic analysis on mass and stiffness irregular buildings. For that, they have prepared four different models, Regular building, Mass irregular building, Stiffness irregular building, Setback building. They have performed time history analysis using three various records. They have studied the location of plastic hinges formed. In most of cases, hinges are formed at regions of irregularities. This study confirms that the enhancement of member sizes required in the region of irregularities. [3]

Han-Seon Lee Dong-Woo-Ko (2004) have investigated the seismic response of high raised RC bearing wall structure with three types of irregularities at bottom stories. (Korea). For this purpose, they have made three 1: 12 scale, 17 storey RCC models with upper 15 storey have bearing wall system, with lower two stories have framed system (Piloti Type Structures). Model 1- Only MRF, Model 2- Infilled shear walls in central frame, Model 3 – Infilled shear walls in one exterior frame. The test results showed that the existence of shear wall reduces remarkably shear deformation at lower frame but has a negligible effect on reduction of overturning deformation, base shear, and OTM. Upper floors prove to behave almost as rigid bodies Shear deformation in rigid and flexible Models is significantly different. [4]

Gaikwad Ujwala (2017) has performed a Response spectrum analysis on the horizontally unsymmetrical structure. Five different cases are used to analyze the structure, With-out shear wall, Shear walls parallel to X-axis, Shear walls parallel to Y- axis, Concentric shear walls, Shear walls at exterior corners, Shear walls at specified locations. They obtained Torsion, base shear, maximum displacement, and maximum drift results for five different cases. They obtain the optimum benefit in the case of shear walls provided at the exterior corners of buildings (base shear reduced to 28% to 35 %, Torsion reduced by 29% to 35%). They also suggested that a higher thickness of shear walls is uneconomical, and its effect on Torsion and base shear is comparatively less. [5]

Prof. Dr. Adnan Falih Ali (Iraq) 2014 has analyzed the U- shaped six story RCC building with and without shear wallsusing SAP 2000, ETABS, and ANSYS software, under excitation of El-Centro earthquake. The first analysis on MRF showed that Torsion is a dominant mode of vibration, and there is no pure translational mode in the Y direction. They got the result that, left corner vibrates at the higher amplitude and lower frequency than the right corner, and they both move out of phase motion causes the Torsion. The addition of shear walls at a particular location causes pure lateral mode in Y- direction. Now LHS and RHS corner displacement graph coincide with each other.[6]

Dr. Dushyanath and Dr. Babitha Rani (2017) have carried out a response spectrum analysis on G+4 L-shaped IT building. Stiffness is calculated for each column in a corresponding frame will give frame stiffness. Earthquake force is distributed to all structures according to their stiffness. From this, they have observed that distributed force is maximum for some frames, so we can reduce the force by adding shear walls in frames having less lateral force. So, from this, we can choose the appropriate location for shear walls in the less lateral force frame. [7]

Rajan L. Wankhade, (2016) has studied the performance- based analysis on G+9 storey building under earthquake loading. In analysis, various cases are considered with an increase in the percentage of reinforcement in many frame elements at different construction stages. From the analysis it is observed that a combination of change of reinforcement increases the capacity of the structure and also satisfies given acceptance criteria. This paper presents the idea about the Performance-based methods allow designers to come up with a variety of solutions, and the performance- based approach enhances creativity and innovation in the design process. [8]

Hasan R. (2002) presented a simple computer-based pushover analysis technique for performance-based design of building frameworks subject to earthquake loading. Through the use of a plasticity-factor for measurement of the degree of classification, the elastic and geometric stiffness matrices were modified to account for non-linear elastic-plastic behavior under constant gravity loads and incrementally increasing lateral loads. The method accounted for first-order elastic and second-order geometric stiffness properties, and the influence that combined stresses have no plastic behavior.[18]

M. Mouzzoun, A.Taleb O.Moustachi, discussed that plastichinges occur at the ends of beam and base columns, then propagates to upper stories and results in the yielding of members. Under low-intensity plastic hinges formed are in initial stages, so the structure remains stable. Under high- intensity plastic hinges formed in collapse states, making the structure unstable as it lost its rigidity and its original strength. So the pushover analysis is able to evaluate the seismic damage of buildings, to examine the state of the structure under the action of an earthquake, and thus provide information on the damage that can be sustained by a structure and the elements that will be affected in a future earthquake.

Kadid and A. Boumrkik Studied three structuresrepresenting low, medium, and high rise RC frame structures. The performance of reinforced concrete frames was investigated using the pushover analysis. Conclusions from the study are listed below, The behavior of a building is shown by performance points and the distribution of hinges. In the case of adequately designed frames with adequate ductility, most of the hingesdeveloped in the beams and few in the columns but with limited damage. The results obtained in terms of demand, capacity and plastic hinges gave an insight into the real behavior of structures[21]

A. Problem Statement

Modelling of simple G+14, L-shaped building in ETABS 2017 by Influence of Positioning of Shear Wall on The Torsional Response of Building.

B. Objective of the Study

The objectives of the present research work were as follows,

1) To study the variation of base shear, Torsion, time period, and eccentricity between the center of mass and center of stiffness for different positions of shear walls.
2) To apply pushover analysis for locating development of plastic hinges and to study variation in ductility.
3) To suggest an effective structural system for a givenbuilding configuration

C. Scope of Work

The above objectives have the following limitations of scope,

1) Reinforced concrete moment resisting frame with shear walls is considered.
2) Fixity is assumed in all the column Ends.
3) The project is focusing analysis part and that too only for earthquake loads by using numerical investigation technique, not an experimental investigation.
4) The analysis is performed by using numerical tool SAP 2000, ETABS 2017.
5) Other shapes of buildings are not in scope.
6) Research work is carried out as per Indian Codes, i.e., IS 875: 1987, IS 456: 2000, and IS 1893:2016.
7) Hinge properties are considered as per FEMA356.
8) The conclusions will be used to suggest the effectivestructural system for a given building configuration.

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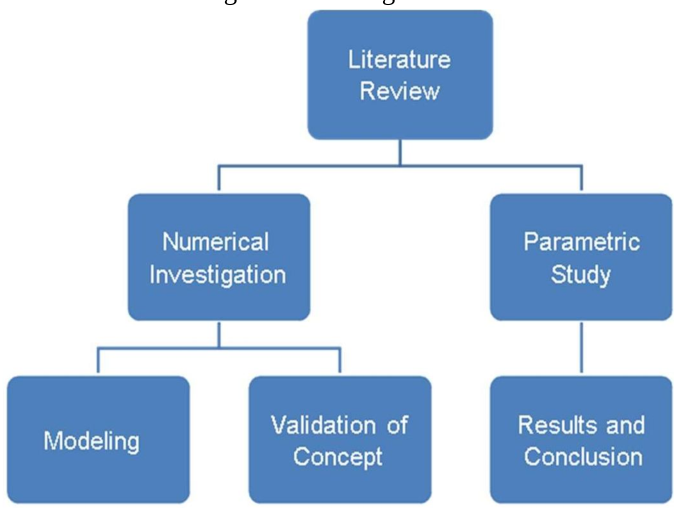
III. METHODOLOGY

1) *Literature Review:* Study of the torsional response of building by response spectrum analysis and non- linear static analysis and its need under this research work. Understanding of the importance of the location of shear walls and its interpretation. Gap analysis is carried out by comparing previous Codal provision and revised one for Torsion and ductile detailing.
2) *Numerical Investigation:* Modeling and Validation Of various concept in the research work by Modeled a 3bay 4 storey structure by using ETABS 2017 and by manual calculation for comparison of base shear. Location and stage of hinges is studied for increasing displacement using SAP2000.
3) *Parametric Study:* Response spectrum analysis is carried out on G+14 storey, L-shaped 5 models, which is under research work using ETABS 2017. Non-linear static analysis is carried out on (G+14) Storey L-shaped 5 models, which is under research work Using SAP-2000. For finding out the performance level of the structure for displacement controlling criteria. Optimum location of shear walls is to be found out.
4) *Result and Conclusion:* Recommendation for shifting Torsion to higher modes ofvibration and achieve good ductility demand.



A. *Numerical Investigation & Analytical Study*

1) *Validation of Sap200*

a) *Part-1-Analatical Study*

An analytical study was carried out using a single frame. The purpose of this was to understand the modeling and pushover analysis (Displacement Control) procedure inSAP2000.

➢ *Description of Frame*

- RCC frame with single-bay and two-storied
- Floor to floor height is 3.5m, and bay width is 4m
- Reinforcement – Fe 415 and Concrete – M20
- Column Size – 400mm x 230mm
- Beam Size – 300mm x 230mm
- Response Spectra- IS:1893 (Part 1)-2002
- Soil strata- Hard Rock
- Zone – V
- Importance Factor- 1
- Modal Combination – Square root of the sum of squares (SRSS)xi)
- Directional Combination - Square root of the sum of squares (SRSS)
- Load Combination- 1.5 (DL+EL) as per IS: 1893-2002
- Supports are fixed at base
- Joints are rigid

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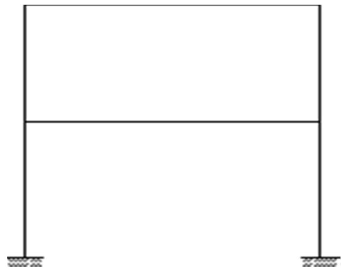

Figure 2: single bay single story frame

- *Modeling of Frame*

Following is the Model of the frame that was developed in SAP2000. The Modelwas then analyzed using a non-linear static analysis method.

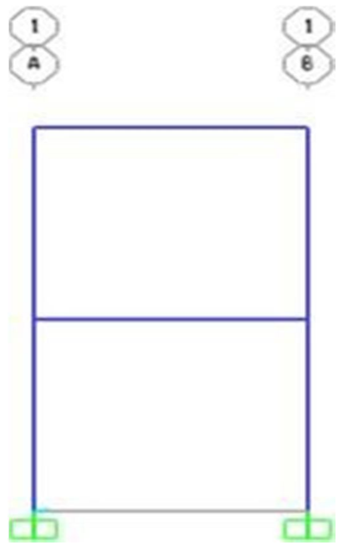

Figure 3: Frame modeled in SAP2000

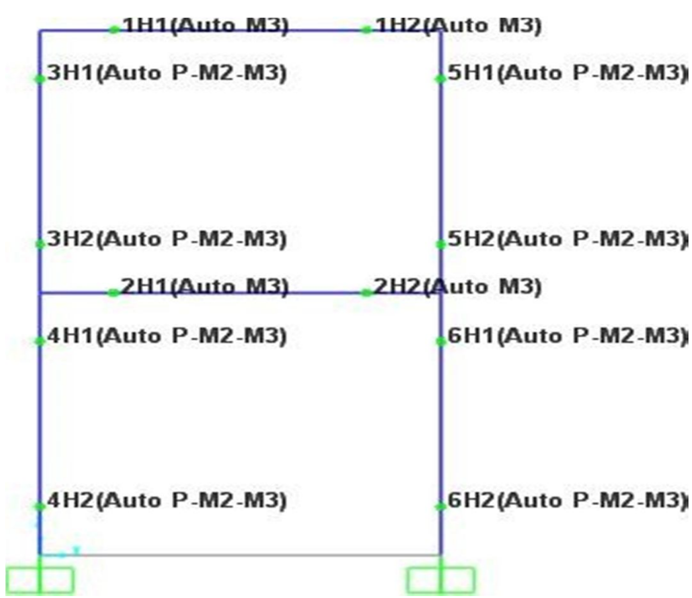

Figure 4: Hinges assigned to the frame

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2) Part-2-Numerical Analysis

During the analysis, the frame was pushed to pre-estimated displacement. At each increment of lateral loading (displacement), corresponding base shear and status of hinges were recorded. With an increment of displacement number of hinges gets formed and transferred from one performance level to another. A&B, B to IO, IO to LS,LS to CP, CP to C, C to D, D to E, beyond E are various performance levels and a number of hinges in these levels helps to determine the performance of structure at particular displacement.

At 0.0013mm (very small displacement), all hinges are within A to B performance level, i.e., serviceability level. This represented when a frame was pushed to small displacement; all hinges were within the elastic range, and no inelastic deformation wasobserved. Further, when the frame was pushed to incrementally increasing displacement and get transferred to subsequent performance level. The figure below shows the capacity curve for the frame. The capacity curve represents the global performance of the frame for the design earthquake.

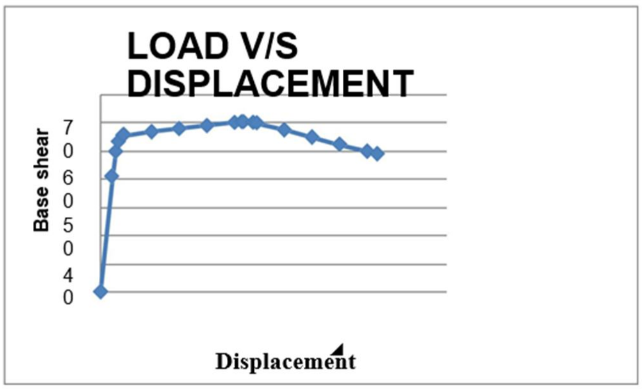

Figure : Capacity curve obtained from pushover analysis for frame

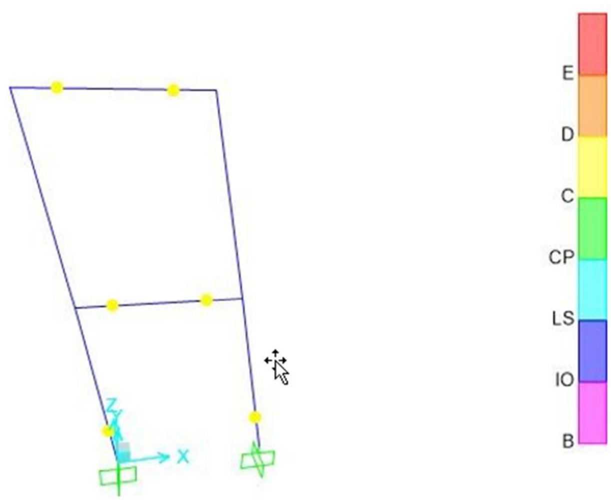

Figure: Status of hinges at a displacement value

a) Conclusion Remark

Analysis Results of the frame have good agreement with FEMA 356 criteria for defining a performance level of the structure by considering the drift criteria. From this Validation, it can be concluded that numerical tool SAP2000 can be appropriately used for pushover analysis.

3) Validation of Etaab 2017

A 3 bay (in both directions), four storey model is created, response spectrum analysis is done, and acomparison of base shear is made from software results and from Manual calculations.

Sections used: -
Column: 230 x 500 mm
Beam: 230 x 450 mm
Slab : 125 mm

Loads: -
Wall load: 7 KN/m
Live load: 2 KN/m^2
Floor Finish: 1.5 KN/m^2

Manual Calculations
i. Self-weight of Beam = 0.23x0.45x25x2.5=6.486 KN
ii. Self-weight of Column = 0.23x0.5x25x2.55=7.33KN
iii. Self-weight of Slab = 0.12x2.5x2.5x25=19.53KN
iv. Dead load on beam = 7x2.5 = 17.5KN
v. Dead load on slab = 1.5x2.5x2.5 =9.375 KN
vi. Total Dead Weight =
(6.486x24x4)+(7.33x16x4)+(19.53x9x3)+(17.5x24x4) +(9.375x9x3) =3550.483
Vii. Total live load = (2x2.5x2.5x9x2) + (0.5x2x2.5x2.5x9x2) =337.5KN
viii. Total weight (W) = D.L + 0.25 L. L = 3626.7024 KN
ix. Zone factor (Z) = 0.16.
x. Importance factor (I) = 1
xi. Response reduction factor = 5
xii. Design acceleration coefficient (Sa/g) = 2.5
xiii.Base shear = (Z/2) *(I/R) *(Sa/g) *W
= (0.16/2) *(1/5) *2.5*3626.7024
= 145.06 KN.

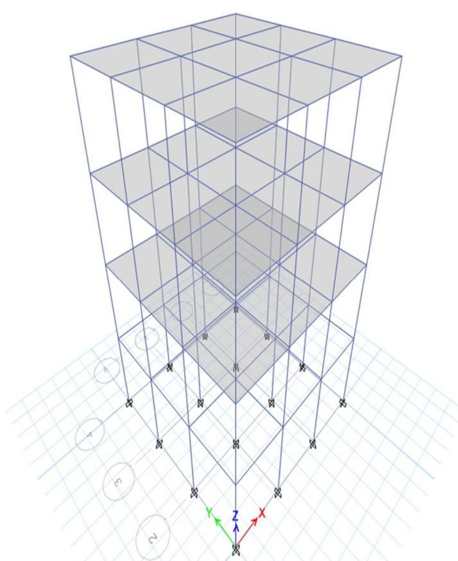

Figure7: Model used for Validation.

Base Reactions

3 of 33 Reload Apply

Load Case/Combo	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m	
Dead	0	0	3542.3274	13283.7278	-13283.7278	0	0
Live	0	0	337.5	1265.625	-1265.625	0	0
EQX	-142.3089	0	0	0	-1422.1237	533.6584	0
EQY	0	-142.3089	0	1422.1237	0	-533.6584	0
RSX Max	134.2387	0	0	0	1203.2511	503.3951	0
RSY Max	0	83.2151	0	729.9306	0	312.0567	0
DCon1	0	0	5313.4911	19925.5918	-19925.5918	0	0
DCon2	0	0	5819.7411	21824.0293	-21824.0293	0	0
DCon3	-170.7707	0	4655.7929	17459.2234	-19165.7719	640.3901	0
DCon4	170.7707	0	4655.7929	17459.2234	-15752.6749	-640.3901	0
DCon5	0	-170.7707	4655.7929	19165.7719	-17459.2234	-640.3901	0

Figure 8: Output from ETABS 2017 software.

4) Conclusion Remark

The manual calculation and the software analysis using ETABS 2017 software were carried out onthe above three-bay, four story model, and the results obtained from both analysis showed that the variation in results for manual calculations and software analysis has a good similarity.

B. Parametric Study

Consideration for parametric study:

As stated in the aim and objectives of these research work to fulfill these requirements,the following are the five cases that are taken into study.

- Building without a shear wall.
- Building with shear walls parallel to X-direction.
- Building with shear walls parallel to Y-direction.
- Building with shear wall at exterior corners.
- Building with shear wall at core. `

C. Brief Description Of The Above Analysed Cases

1) *Case No 1:* In this case, there is no shear wall in the structure. The structure isanalyzed by the response spectrum method using ETAB2017 and by pushover analysis using SAP2000.

The objective of this analysis:

Check the torsional mode of vibration of the structure, base shear of response spectrum case,eccentricity between the center of mass and center of rigidity.

Checking the base shear of pushover case, ductility factor, location and states ofplastic hinges formed.

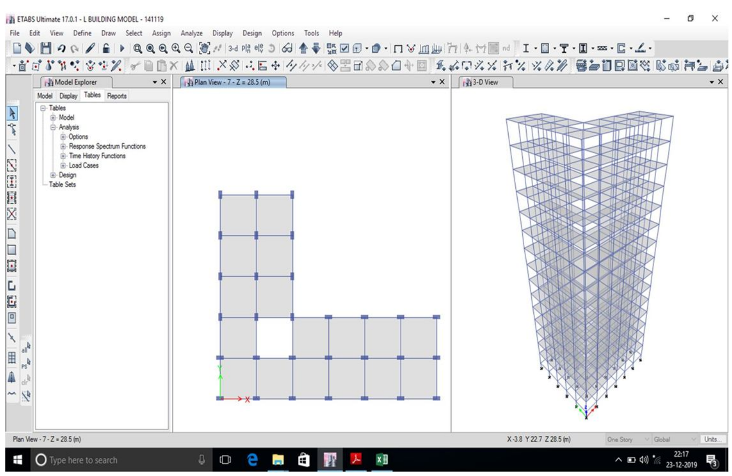

Figure 9 : Case 1- Plan and 3-D view of the building (ETAB MODEL)

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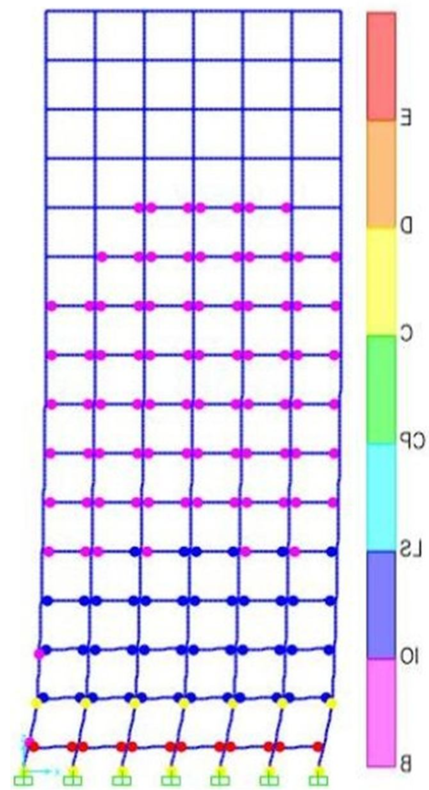

Figure 10 : Case 1- Location and states of plastic hinges formed. (SAP Model)

2) *Case No 2:* In this case, the shear walls are parallel to X-direction. The structure isanalyzed by the response spectrum method using ETAB2017 and by pushover analysis using SAP2000.

The objective of this analysis:

Check the torsional mode of vibration of the structure, base shear of response spectrum case,eccentricity between the center of mass and center of rigidity.

Checking the base shear of pushover case, ductility factor, location, and states ofplastic hinges formed.

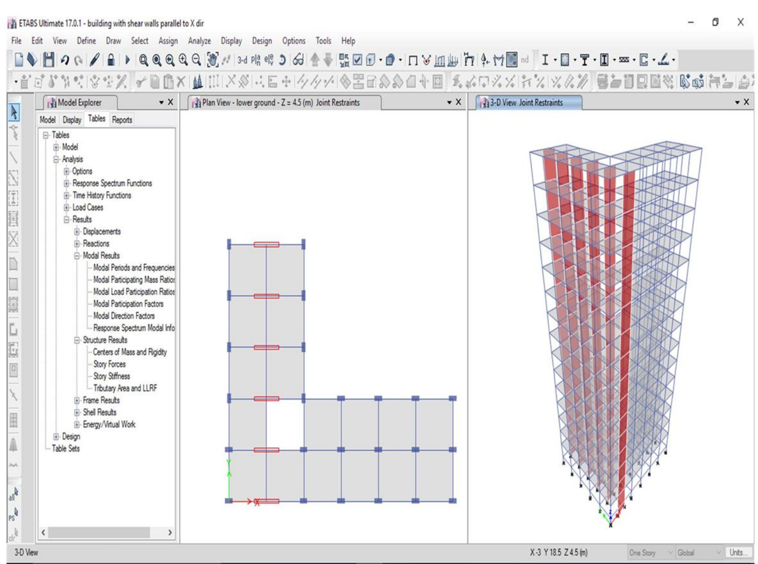

Figure 11 :Case 2- Plan and 3-D view of the building (ETAB Model)

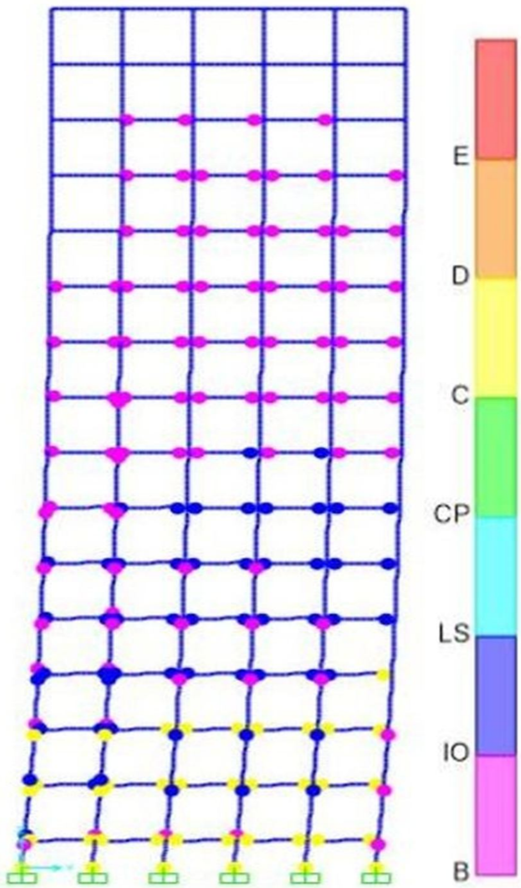

Figure 12 : Case 2- Location and states of plastic hinges formed. (SAP Model)

3) *Case No 3:* In this case, the shear walls are placed parallel to Y-direction. Thestructure is analyzed by the response spectrum method using ETAB2017 and bypushover analysis using SAP2000.

The objective of this analysis:

Check the torsional mode of vibration of the structure, base shear of response spectrum case,eccentricity between the center of mass and center of rigidity.

Checking the base shear of pushover case, ductility factor, location and states of plastichinges formed.

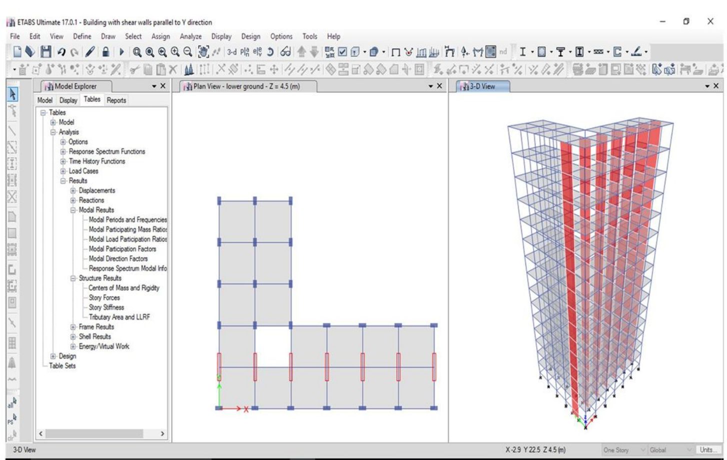

Figure 13 : Case 3- Plan and 3-D view of the building (ETAB MODEL)

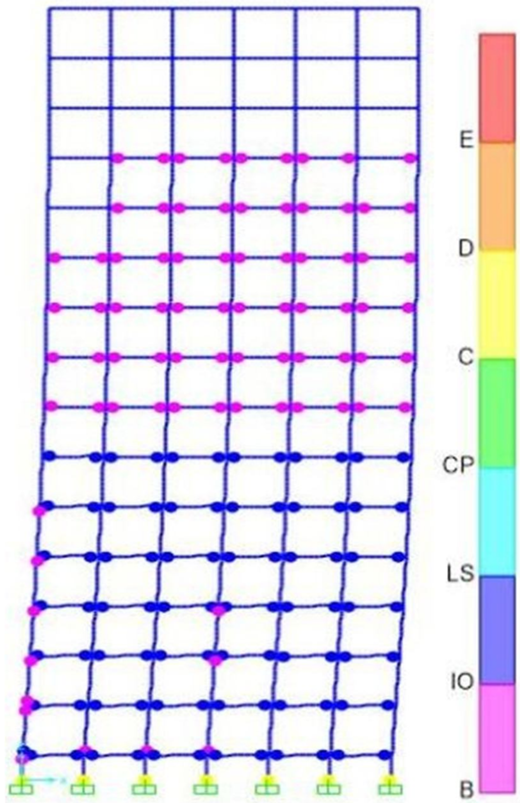

Figure 14 : Case 3- Location and states of plastic hinges formed. (SAP Model)

4) *Case No 4:* In this case, the shear walls are placed at exterior corners. The structure is analyzed by the response spectrum method using ETAB2017 and by pushover analysisusing SAP2000.

The objective of this analysis:

Check the torsional mode of vibration of the structure, base shear of response spectrum case,eccentricity between the center of mass and center of rigidity.

Checking the base shear of pushover case, ductility factor, location, and states of plastichinges formed.

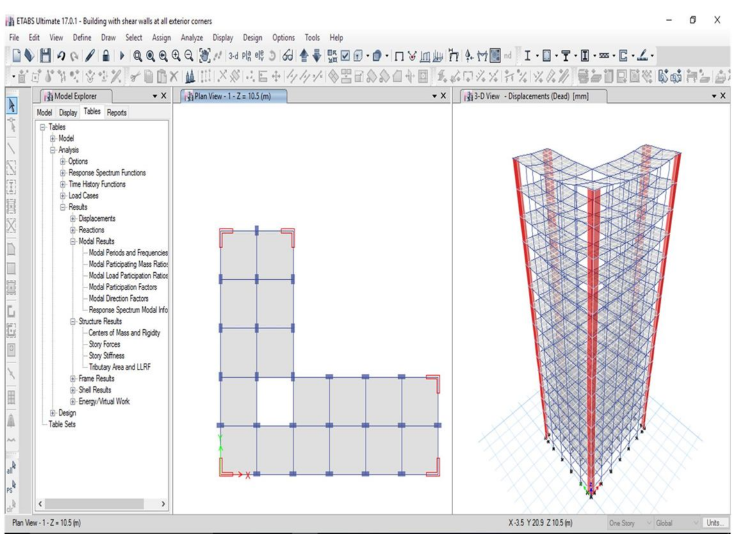

Figure 15 : Case 4- Plan and 3-D view of the building (ETAB MODEL)

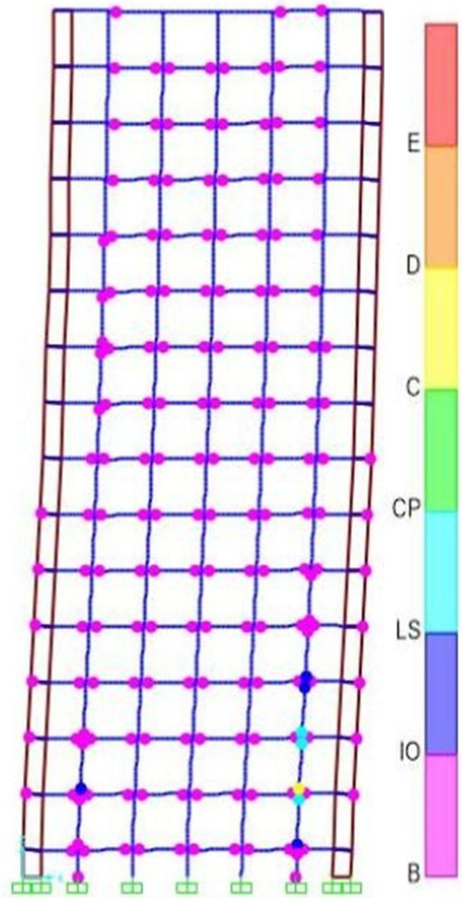

Figure 16 : Case 4- Location and states of plastic hinges formed. (SAP Model)

5) *Case No 5:* In this case, the shear walls are placed at the central core. The structure is analyzed by the response spectrum method using ETAB2017 and by pushover analysisusing SAP2000.

The objective of this analysis:

Check the torsional mode of vibration of the structure, base shear of response spectrum case,eccentricity between the center of mass and center of rigidity.

Checking the base shear of pushover case, ductility factor, location, and states of plastichinges formed.

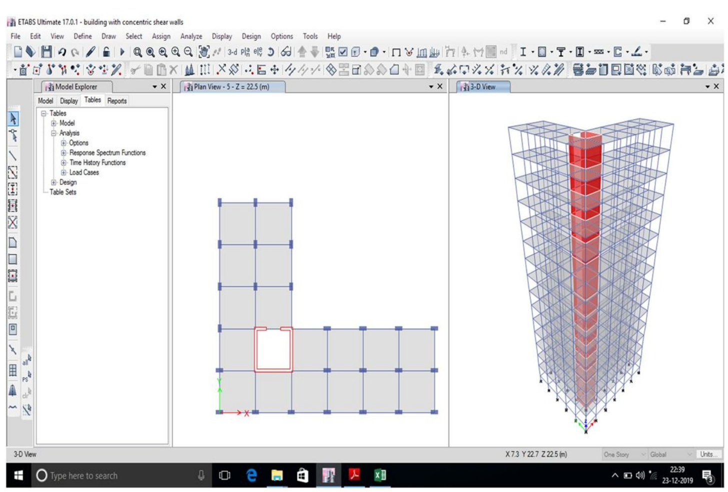

Figure 17 : Case 5- Plan and 3-D view of building (ETAB MODEL)

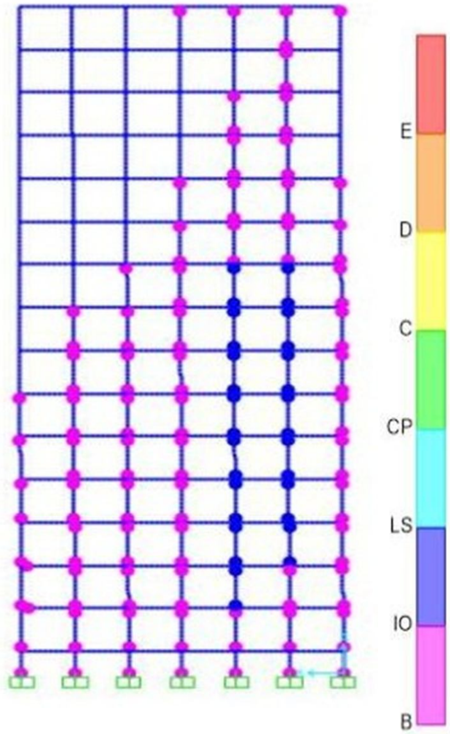

Figure 18: Case 5- Location and states of plastic hinges formed. (SAP Model

D. Observation Of Numerical Output

1) Torsional Mode of Vibration

In parametric study, analysis of structure is carried out for five cases, which are described above, and the modal mass participation ratio is found out using the responsespectrum method to find the torsional mode of vibration.
This gives the idea about the performance of the structure under seismic loading.

Table 1: Torsional mode of vibration

SR. NO.	MODEL	TORSION PERCENTAGE	Column1 MODE
1	BUILDING WITHOUT SW	31	1st
2	BUILDING WITH SW PARALLEL TO X	52	2nd
3	BUILDING WITH SW PARALLEL TO Y	18	2nd
4	BUILDING WITH SW AT EXT. CORNER	4	2nd
5	BUILDING WITH CONC. SW.	38	1st

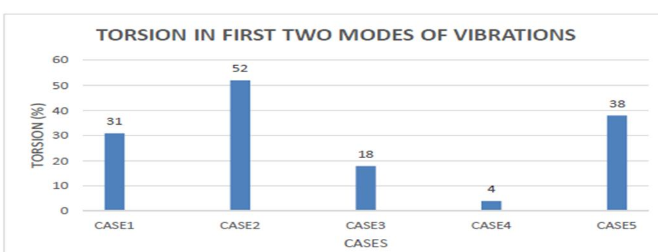

Figure 19 : graphical representation of the Torsional mode of vibration.

- *Commentary:* Above graph represents the torsional mode of vibration for different cases. It represents that for case no. 4, the first two modes are translational, and Torsion is very less in the first two modes.

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2) Eccentricity in X and Y directions:

The eccentricity between the center of mass and center of stiffness in both X and Y directions is calculated for five different cases under consideration. The eccentricity can bedirectly correlated with the torsional moment.

Table 2 : Eccentricity in x and y-direction

SR. NO.	MODEL	ECCENTRICITY	
		X DIR (m)	Y DIR (m)
1	BUILDING WITHOUT SW	1.59	1.43
2	BUILDING WITH SW PARALLEL TO X	1.53	0.83
3	BUILDING WITH SW PARALLEL TO Y	1.11	1.42
4	BUILDING WITH SW AT EXT. CORNERS	1.03	0.88
5	BUILDING WITH CONC. SW	2.088	1.932

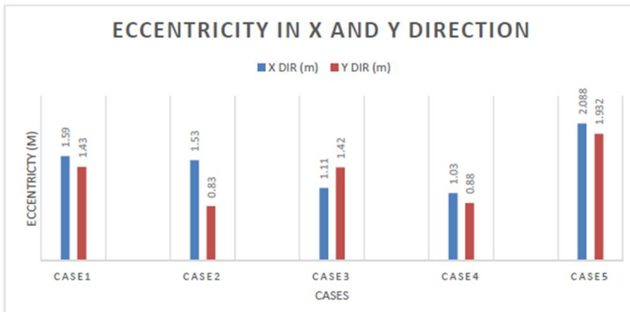

Figure 20 : graphical representation of Eccentricities in X and Y directions.

- *Commentary:* Above graph represents in X and Y directions for five cases taken into consideration. It represents that eccentricity is less in the case of no.4 compared to other cases. It directly affects the torsional mode of vibration.

3) The Time Period of the first Mode

The time period of the first mode is calculated for five different cases under consideration andcompared. The time period can be related to the frequency and how it affects Torsion can be studied.

Table 3: Time period of the first mode

Column1	Column2	Column3
SR. NO.	MODEL	PERIOD OF VIBRATION
		(SEC)
1	BUILDING WITHOUT SW	2.343
2	BUILDING WITH SW PARALLEL TO X	2.216
3	BUILDING WITH SW PARALLEL TO Y	2.074
4	BUILDING WITH SW AT EXT. CORNERS	1.869
5	BUILDING WITH CONC. SW	1.954

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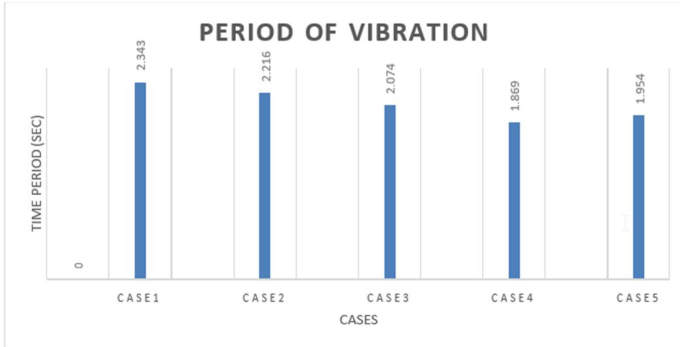

Figure 21 : graphical representation of the Time period of the first mode.

- *Commentary:* The period of vibration of the first mode for five different models are compared. It is found that the highest period is for the first case and lowest for the fourth case.

4) Base shear from pushover analysis:

The base shear in x and y direction is calculated from pushover analysis for five differentcases under considerations. The base shear can be related to the stiffness of the building, which further can be related to the torsional moment in the building.

Table 4: Base shear from pushover case

SR. NO.	MODEL	BASE SHEAR	
		PX	PY
		(KN)	(KN)
1	BUILDING WITHOUT SW	3380.53	3127.66
2	BUILDING WITH SW PARALLEL TO X	6987.17	3344.025
3	BUILDING WITH SW PARALLEL TO Y	3529.023	7226.336
4	BUILDING WITH SW AT EXT. CORNERS	6452.92	6135.12
5	BUILDING WITH CONC. SW	6795.33	8264.98

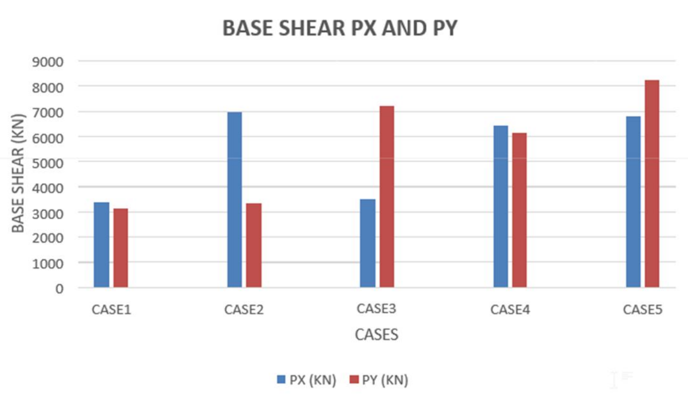

Figure 22: graphical representation of Base shear in x and y direction (from pushover analysis).

- *Commentary:* The base shear from pushover analysis is compared for five cases.From the comparison, it was found that base shear has the highest value for case fifth, and there is a lesser difference in base shear in both directions for the case third.

5) Ductility Factor from Pushover Analysis

Table 5: Ductility Factor in x and y-direction.

SR. NO.	MODEL	DUCTILITY FACTOR	Column1
		X DIRECTION	Y DIRECTION
1	BUILDING WITHOUT SW	2.369989975	2.401753901
2	BUILDING WITH SW PARALLEL TO X	4.707257097	2.4676456
3	BUILDING WITH SW PARALLEL TO Y	2.358925289	5.289856918
4	BUILDING WITH SW AT EXT. CORNERS	4.383777174	4.565229038
5	BUILDING WITH CONC. SW	4.788208684	6.378972879

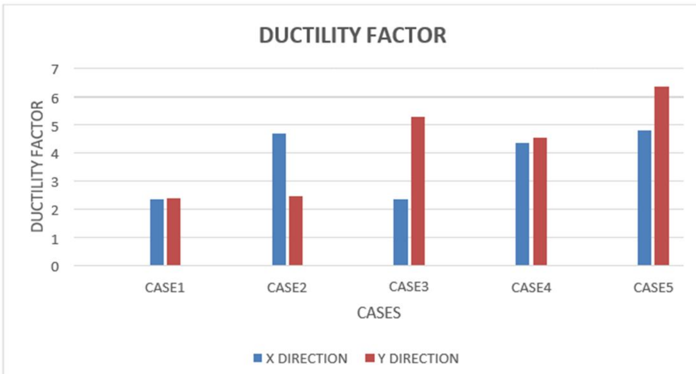

Figure 23: graphical representation of Ductility factors in x and y-direction.

- *Commentary:* The ductility factor in x and y directions are calculated and compared for five different cases. From comparison, it is found that the ductility factors have high valuesfor case fifth. The difference in ductility factor in x and y direction is less for case fourth.

6) Modes of Vibration

MODES OF VIBRATION	CASE 1	CASE 2	CASE 3	CASE 4	CASE 5
MODE 1	TORSIONAL	Y TRANSLATION	X TRANSLATION	Y TRANSLATION	TORSIONAL
MODE 2	DIAGONAL TRANSLATION	TORSIONAL	TORSIONAL	X TRANSLATION	DIAGONAL TRANSLATION
MODE 3	TORSIONAL	X TRANSLATION + TORSIONAL	TORSIONAL	TORSIONAL	TORSIONAL
MODE 4	DIAGONAL TRANSLATION	Y TRANSLATION	X TRANSLATION	Y TRANSLATION	TORSIONAL
MODE 5	TORSIONAL	TORSIONAL	TORSIONAL	X TRANSLATION	DIAGONAL TRANSLATION
MODE 6	DIAGONAL TRANSLATION	X TRANSLATION	Y TRANSLATION	TORSIONAL	TORSIONAL
MODE 7	TORSIONAL	Y TRANSLATION	X TRANSLATION	Y TRANSLATION	TORSIONAL
MODE 8	DIAGONAL TRANSLATION	Y TRANSLATION	X TRANSLATION	X TRANSLATION	TORSIONAL
MODE 9	TORSIONAL	TORSIONAL	TORSIONAL	TORSIONAL	DIAGONAL TRANSLATION
MODE 10	DIAGONAL TRANSLATION	X TRANSLATION	Y TRANSLATION + TORSIONAL	Y TRANSLATION	TORSIONAL

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- *Commentary*
- As the projections are small, no opening-closing mode and dog tail wagging modes are observed.
- For case4, only the first two modes of vibrations are pure translational modes.
- For case1 and case5, the diagonal translational mode is observed, which is dangerous.
- The time period of the fundamental mode is least for case4 and highest for case1.

7) Location and States of Plastic Hinges

The pushover analysis is carried out on five models using displacement control method.The location and states of plastic hinges for target displacement is calculated and compared.

- *Commentary:* From pushover analysis, the location and states of plastic hinges are calculated for different cases. From a comparison of results, it is found that lesser hinges are formed in columns forcase fourth. Hinge states go beyond the collapse prevention level for case1. From the above parametric study, it seems that the time period of case 4 and case 5 are lesser; therefore, the stiffness of these two models is higher. Base shear from pushover analysis and ductility factors are also most upper for these two cases; therefore, it seems that resistance of these cases before the collapseis also highest. But the ductility factors have the approximately same value in x and y direction, and Torsion is in the 3rdmode of vibration for case 4, and in case 5, the difference is quite higher. Therefore, from the above study, we may say that the building with a lesser time period is stiffer one, andagain if these stiffer buildings have the same ductility factors in both plan direction, torsion may shift in 3rd mode. From the above parametric study, it is observed that optimum results are obtained for case no. 4. In caseno.4 shear walls are provided at exterior corners. So, trials are done on the live MHADA project under consideration by applying shear walls at exterior corners to shift Torsion to a higher mode of vibration.

E. B-LIVE Building Project

1) Description Of Structure

This project consists of 14 storied (2B+G+12) MHADA residential buildings. The building is proposed at Talegaon Dabhade, Pune, India. It is proposed to use a conventional RCC structural system for the execution. All structural columns, beams, and roof heights details viz. location and sizes are coordinated with client and contractor in many iterations. The building structure is analyzed and designed using ETAB 2017, RCDC, and SAFE software. The permissible values of the load factors and stresses in the materials will be utilized within the provisions of the Indian standards code of practices.

2) Detals Of Building

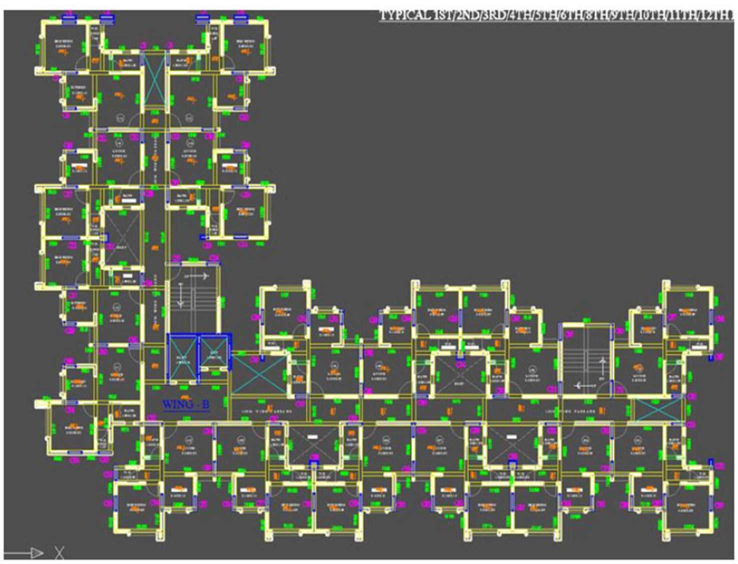

Figure 23–Plan of Structure.

IV. RESULTS & ANALYSIS

1) *Alternative 1:* (G+14) MHADA BUILDING WITH NO SHEAR WALLS.

Response spectrum analysis is carried out and identify the modal participation mass ratios, deformations. The objective of this analysis is to identify the torsional mode of vibration, eccentricity in x and y direction, maximum displacement, area of reinforcement required for columns and shear walls.

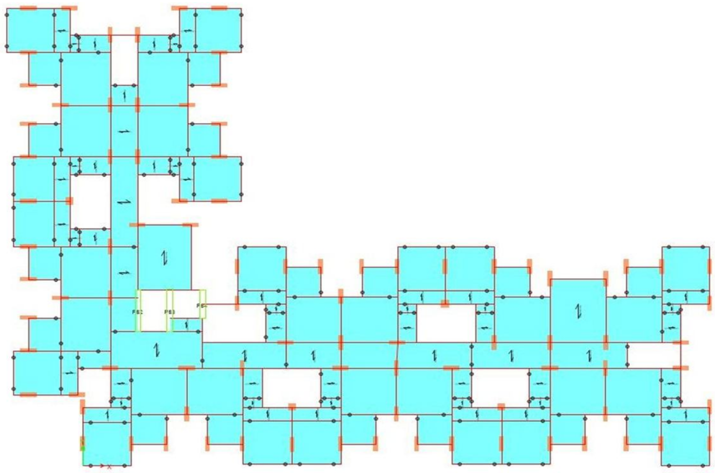

Figure 24 : plan view of the building (Alternative 1).

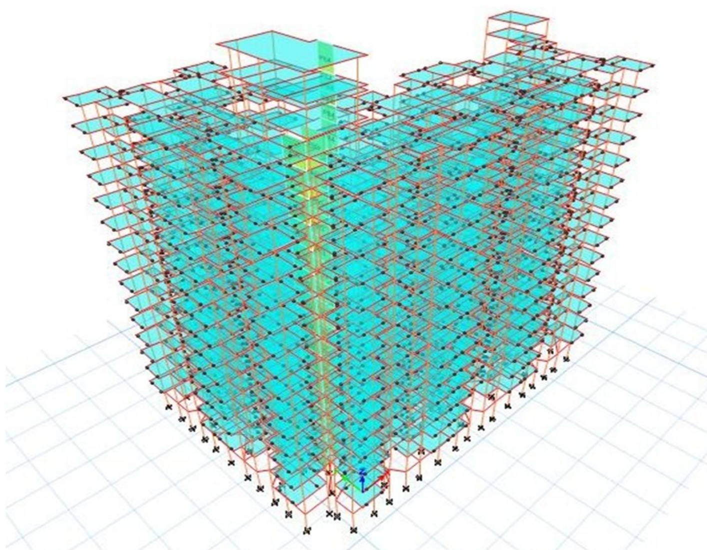

Fig no 25: 3D view of the building (Alternative 1).

Table No 6: Modal participation mass ratios Alternative 1.

Mode	Period	UX	UY.	UZ.	RX.	RY.	RZ.
	sec						
1	1.999	0.2455	0.0778	0	0.0214	0.0519	0.4839
2	1.939	0.5722	0.0498	0	0.0133	0.1357	0.1865
3	1.856	0.0013	0.6646	0	0.1817	0.0006	0.1222
4	0.632	0.0811	0.0017	0	0.0063	0.4477	0.029
5	0.596	0.03	0.0129	0	0.0582	0.1412	0.0816
6	0.572	0.0009	0.1074	0	0.4616	0.0029	0.0148
7	0.347	0.0256	2.55E-05	0	0.0001	0.058	0.0067
8	0.311	0.0053	0.0023	0	0.0054	0.0124	0.0266
9	0.297	0.0002	0.0347	0	0.0771	0.0004	0.0033
10	0.231	0.0117	1.08E-05	0	4.76E-05	0.0539	0.0025
11	0.199	0.002	0.0014	0	0.0054	0.0082	0.0124
12	0.186	0.0001	0.0162	0	0.0622	0.0003	0.002

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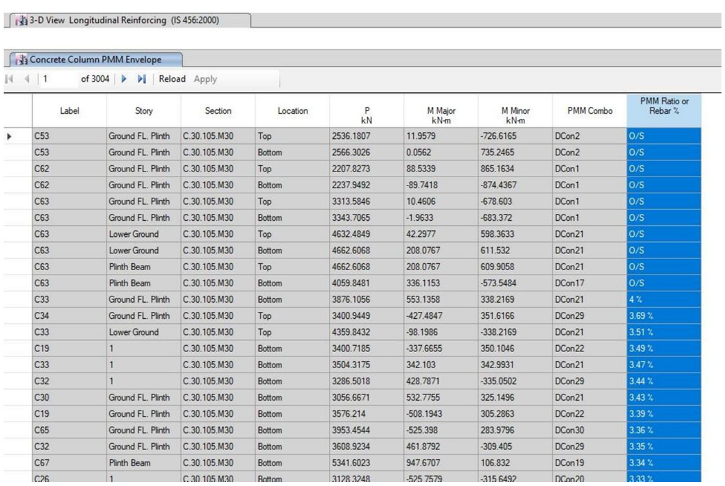
Table No 7: Center of mass and Rigidity (Eccentricity in x and y direction)Alternative 1.

Story	XCM	YCM	XCCM	YCCM	XCR	YCR	ECC.X	ECC. Y
	m	m	m	m	m	m	m	m
Terrace	14.0578	12.1494	14.0578	12.1494	13.3884	12.1234	-0.6694	-0.026
12	14.0056	12.2116	14.0261	12.1873	13.3924	12.2154	-0.6132	0.0038
11	14.1272	12.119	14.0644	12.1614	13.4176	12.3088	-0.7096	0.1898
10	14.1261	12.118	14.0813	12.1495	13.4469	12.4033	-0.6792	0.2853
9	14.1288	12.117	14.0916	12.1425	13.4674	12.5018	-0.6614	0.3848
8	14.1274	12.1165	14.0979	12.1379	13.4779	12.6069	-0.6495	0.4904
7	14.1254	12.1167	14.1021	12.1347	13.4808	12.7211	-0.6446	0.6044
6	14.1169	12.12	14.104	12.1327	13.4725	12.8463	-0.6444	0.7263
5	14.1288	12.1206	14.1069	12.1313	13.4475	12.9831	-0.6813	0.8625
4	14.124	12.1131	14.1087	12.1294	13.4024	13.126	-0.7216	1.0129
3	14.0774	12.1378	14.1057	12.1302	13.3339	13.2584	-0.7435	1.1206
2	14.0977	12.1289	14.105	12.1301	13.2587	13.3836	-0.839	1.2547
1	14.0442	12.1652	14.1002	12.1329	13.1586	13.5266	-0.8856	1.3614
Ground FL. Plinth	13.4034	11.7152	14.0528	12.1045	12.9894	13.8303	-0.414	2.1151
Lower Ground	12.2285	10.2468	13.9608	12.0108	12.5994	14.1893	0.3709	3.9425
Plinth Beam	17.8858	12.5497	13.9754	12.0128	9.6495	13.608	-8.2363	1.0583

Table No 8: Maximum story displacement Alternative 1.

Story	Load Case/Combo	Direction	Maximum	Average	Ratio
			mm	mm	
Terrace	EQX	X	72.2	71.466	1.01
Terrace	RX Max	X	67.52	62.32	1.083
Terrace	EQY	Y	62.687	58.664	1.069
Terrace	RY Max	Y	58.102	51.072	1.138
Terrace	WINDY	Y	34.087	24.69	1.381
Terrace	RX Max	Y	33.392	18.133	1.841
Terrace	RY Max	X	24.288	13.504	1.799
Terrace	WINDX	X	20.041	16.56	1.21
Terrace	SIDL	Y	1.627	0.328	4.959
Terrace	SIDL	X	1.014	0.116	8.77
Terrace	Dead	Y	0.614	0.399	1.539
Terrace	Dead	X	0.2	0.051	3.904
Terrace	Live	X	0.142	0.126	1.128
Terrace	Live	Y	0.132	0.109	1.213
Terrace	TLIVE	Y	0.017	0.012	1.38
Terrace	TLIVE	X	0.016	0.013	1.253



Label	Story	Section	Location	P kN	M Major kN-m	M Minor kN-m	PMM Combo	PMM Ratio or Rebar %
C53	Ground FL. Plinth	C.30.105.M30	Top	2536.1807	11.9579	-726.6165	DCon2	O/S
C53	Ground FL. Plinth	C.30.105.M30	Bottom	2566.3026	0.0562	735.2465	DCon2	O/S
C62	Ground FL. Plinth	C.30.105.M30	Top	2207.8273	88.5339	865.1634	DCon1	O/S
C62	Ground FL. Plinth	C.30.105.M30	Bottom	2237.9492	-89.7418	-874.4367	DCon1	O/S
C63	Ground FL. Plinth	C.30.105.M30	Top	3313.5846	10.4606	-678.603	DCon1	O/S
C63	Ground FL. Plinth	C.30.105.M30	Bottom	3343.7065	-1.9633	-683.372	DCon1	O/S
C63	Lower Ground	C.30.105.M30	Top	4632.4849	42.2977	598.3633	DCon21	O/S
C63	Lower Ground	C.30.105.M30	Bottom	4662.6068	208.0767	611.532	DCon21	O/S
C63	Plinth Beam	C.30.105.M30	Top	4662.6068	208.0767	609.9058	DCon21	O/S
C63	Plinth Beam	C.30.105.M30	Bottom	4059.8481	336.1153	-573.5484	DCon17	O/S
C33	Ground FL. Plinth	C.30.105.M30	Bottom	3876.1056	553.1358	338.2169	DCon21	4 %
C34	Ground FL. Plinth	C.30.105.M30	Top	3400.9449	-427.4847	351.6166	DCon29	3.69 %
C33	Lower Ground	C.30.105.M30	Top	4359.8432	-98.1986	-338.2169	DCon21	3.51 %
C19	1	C.30.105.M30	Bottom	3400.7185	-337.6655	350.1046	DCon22	3.49 %
C33	1	C.30.105.M30	Bottom	3504.3175	342.103	342.9931	DCon21	3.47 %
C32	1	C.30.105.M30	Bottom	3286.5018	428.7871	-335.0502	DCon29	3.44 %
C30	Ground FL. Plinth	C.30.105.M30	Bottom	3056.6671	532.7755	325.1496	DCon21	3.43 %
C19	Ground FL. Plinth	C.30.105.M30	Bottom	3576.214	-508.1943	305.2863	DCon22	3.39 %
C65	Ground FL. Plinth	C.30.105.M30	Bottom	3953.4544	-525.398	283.9796	DCon30	3.36 %
C32	Ground FL. Plinth	C.30.105.M30	Bottom	3608.9234	461.8792	-309.405	DCon29	3.35 %
C67	Plinth Beam	C.30.105.M30	Bottom	5341.6023	947.6707	106.832	DCon19	3.34 %
C26	1	C.30.105.M30	Bottom	3128.3248	-525.7579	-315.6492	DCon20	3.33 %

Fig 26: Maximum reinforcement in column (Alternative 1).

2) *Alternative 2:* (G+14) MHADA BUILDING WITH SHEAR WALLS AT EXTERIOR CORNERS.

Response spectrum analysis is carried out and identify the modal participation mass ratios, deformations. The objective of this analysis is to identify the torsional mode of vibration, eccentricity in x and y direction, maximum displacement, area of reinforcement required for columns and shear walls.

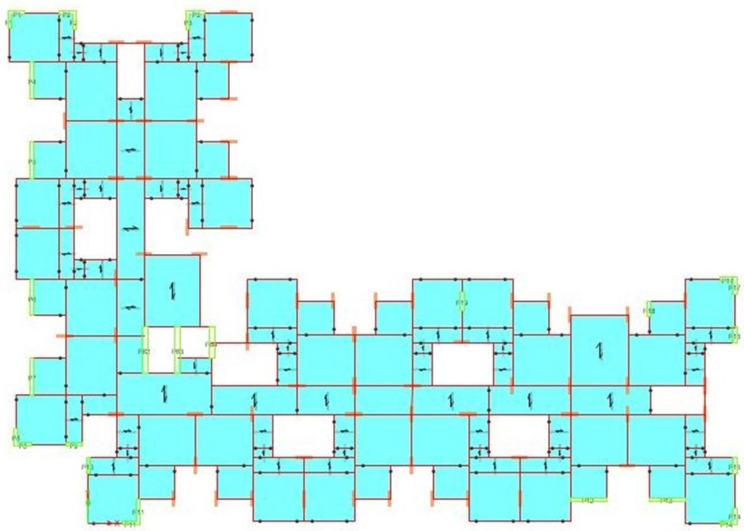


Fig 27: Plan view of the building (Alternative 2).

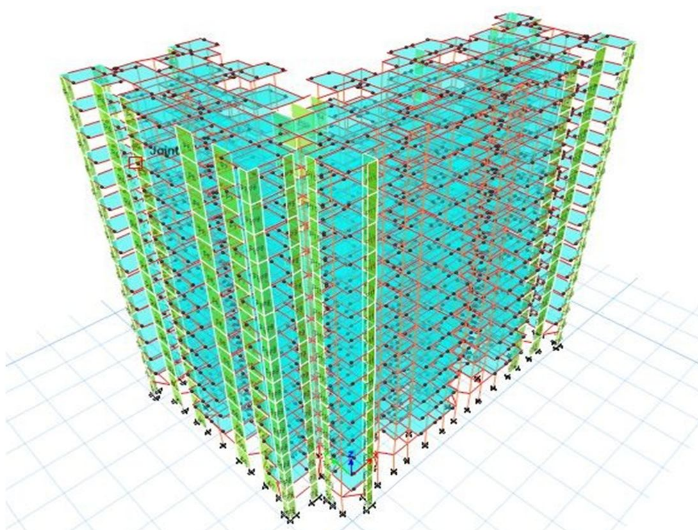


Fig 28: 3D view of the building (Alternative 2).

Table No 10 : Modal participation mass ratios Alternative 2.

Mode	Period sec	UX	UY	UZ.	RX	RY	RZ
1	1.879	0.1118	0.6563	0	0.2017	0.03	0.009
2	1.859	0.626	0.1159	0	0.0361	0.1667	0.0469
3	1.681	0.0568	0.0001	0	0.00001372	0.0175	0.709
4	0.583	0.1132	0.0013	0	0.0048	0.5037	0.0071
5	0.558	0.0014	0.1273	0	0.4752	0.0066	0.0003
6	0.49	0.0073	0.0006	0	0.0014	0.0259	0.1301
7	0.316	0.0364	0.0001	0	0.0002	0.0793	0.0025
8	0.283	0.0003	0.0401	0	0.0875	0.0007	0.0007
9	0.246	0.0018	0.0025	0	0.0047	0.0042	0.0408
10	0.207	0.0166	0.00002917	0	0.0001	0.0664	0.0009
11	0.178	0.0003	0.0169	0	0.0591	0.001	0.0014
12	0.152	0.0006	0.0037	0	0.0117	0.0019	0.018

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Table No 11 : Center of mass and Rigidity (Eccentricity in x and y direction)Alternative 2.

Story	XCM	YCM	XCCM	YCCM	XCR	YCR	ECC X	ECC. Y
	m	m	m	m	m	m	m	m
Terrace	14.1398	12.1124	14.1398	12.1124	14.2553	12.6532	0.1155	0.5408
12	14.0798	12.1589	14.1034	12.1406	14.193	12.7195	0.1132	0.5606
11	14.2013	12.0725	14.1404	12.1148	14.1586	12.8105	-0.0427	0.738
10	14.1925	12.0715	14.1547	12.1029	14.139	12.9083	-0.0535	0.8368
9	14.2027	12.0702	14.1651	12.0959	14.1141	13.0065	-0.0886	0.9363
8	14.1994	12.0696	14.1712	12.0912	14.0771	13.1053	-0.1223	1.0357
7	14.1905	12.07	14.1741	12.088	14.0235	13.2036	-0.167	1.1336
6	14.1816	12.0734	14.1751	12.0861	13.9457	13.2998	-0.2359	1.2264
5	14.1912	12.0736	14.1769	12.0847	13.8312	13.3931	-0.36	1.3195
4	14.1676	12.0605	14.1759	12.0821	13.6605	13.4807	-0.5071	1.4202
3	14.1393	12.088	14.1725	12.0827	13.395	13.5597	-0.7443	1.4717
2	14.1763	12.0732	14.1728	12.0819	12.9691	13.6375	-1.2072	1.5643
1	14.0949	12.0996	14.1667	12.0833	12.3693	13.7861	-1.7256	1.6865
Ground FL. Plinth	13.5208	11.6462	14.1221	12.0531	11.6331	14.0767	-1.8877	2.4305
Lower Ground	12.4575	10.1823	14.0363	11.9567	10.5233	13.4369	-1.9342	3.2546
Plinth Beam	18.0003	12.5548	14.0519	11.959	8.5066	12.9977	-9.4937	0.4429

Table No 12: Maximum storey displacement Alternative 2

Story	Load Case/Comb	Direction	Maximum	Average	Ratio
			mm	mm	
Terrace	EQX	X	68.293	65.717	1.039
Terrace	RX Max	X	62.071	56.514	1.098
Terrace	EQY	Y	59.989	59.596	1.007
Terrace	RY Max	Y	54.87	49.97	1.098
Terrace	RX Max	Y	32.96	18.015	1.83
Terrace	WINDY	Y	31.889	25.301	1.26
Terrace	WINDX	X	18.099	15.483	1.169
Terrace	RY Max	X	9.507	5.928	1.604
Terrace	SIDL	Y	2.275	0.343	6.63
Terrace	SIDL	X	1.79	0.434	4.128
Terrace	Live	X	0.313	0.205	1.528
Terrace	TLIVE	Y	0.03	0.022	1.333
Terrace	TLIVE	X	0.023	0.018	1.286

Concrete Column PMM Envelope

1 of 2112 Reload Apply

Label	Story	Section	Location	P kN	M Major kN-m	M Minor kN-m	PMM Combo	PMM Ratio or Rebar %
C18	1	C.23.1050.M30	Bottom	2923.4965	-240.9016	246.6662	1.5DRYN	5.81 %
C69	1	C.23.1050.M30	Bottom	3035.7487	-319.5176	233.9481	1.5DRXN	5.8 %
C28	1	C.23.1050.M30	Bottom	3003.7139	259.1407	241.5169	1.5DRYN	5.78 %
C48	1	C.23.1050.M30	Bottom	1935.7349	245.2702	-261.5282	1.5DRYN	5.73 %
C66	1	C.23.1050.M30	Bottom	2943.3414	285.5825	-235.2174	1.5DRXN	5.69 %
C65	1	C.23.1050.M30	Bottom	3338.912	-365.4501	-213.7653	1.5DRYN	5.61 %
C17	1	C.23.1050.M30	Bottom	2412.3438	-235.7748	242.9982	1.5DRYN	5.47 %
C48	Terrace	C.23.1050.M30	Top	55.8123	81.4334	291.8957	0.9DRYN	5.46 %
C29	1	C.23.1050.M30	Bottom	2924.0331	391.5325	211.224	1.5DRXN	5.42 %
C73	1	C.23.1050.M30	Bottom	3359.0894	-361.8611	205.9818	1.5DRXN	5.42 %
C32	Terrace	C.23.1050.M30	Top	74.1875	-145.3691	278.3722	1.5DRXN	5.39 %
C36	1	C.23.1050.M30	Bottom	3192.0444	274.6388	-217.0168	1.5DRXN	5.32 %
C40	Terrace	C.23.1050.M30	Top	46.7208	-118.5125	276.0384	0.9DRYN	5.27 %
C44	Terrace	C.23.1050.M30	Top	21.3661	35.8721	-279.3556	0.9DRXN	5.07 %
C36	Terrace	C.23.1050.M30	Top	54.643	146.2157	-256.2468	0.9DRXN	4.92 %
C47	1	C.23.1050.M30	Bottom	2779.2651	-236.0426	215.6847	1.5DRXN	4.92 %
C66	Terrace	C.23.1050.M30	Top	53.3583	-69.6327	264.4893	1.5DRXN	4.83 %
C79	1	C.23.1050.M30	Bottom	2932.574	-361.016	192.7264	1.5DRXN	4.8 %
C41	Terrace	C.23.1050.M30	Top	75.6739	-95.4999	-253.3062	0.9DRYN	4.63 %
C26	Terrace	C.23.1050.M30	Top	9.698	219.6418	-229.9285	1.5DRXN	4.55 %
C33	Terrace	C.23.1050.M30	Top	52.6909	81.8134	-248.9679	0.9DRYN	4.48 %

Fig 29 : Maximum reinforcement in column (Alternative 2).

Shear Wall Pier Summary - IS 456-2000

1 of 928 Reload Apply

Story	Pier Label	Station	Design Type	Edge Rebar	End Rebar	Rebar Spacing mm	Required Reinf %	Current Reinf %	Pier Leg
Plinth Beam	P19	Bottom	Uniform	10	10	250	5.38	0.34	Bottom Leg 1
Plinth Beam	P19	Top	Uniform	10	10	250	5.09	0.34	Top Leg 1
Lower Ground	P19	Bottom	Uniform	10	10	250	5.07	0.34	Bottom Leg 1
Lower Ground	P19	Top	Uniform	10	10	250	5.04	0.34	Top Leg 1
Plinth Beam	P14	Bottom	Uniform	14	14	250	5	0.64	Bottom Leg 1
Plinth Beam	P14	Bottom	Uniform	14	14	250	5	0.64	Bottom Leg 2
1	P9	Bottom	Uniform	10	10	250	4.68	0.34	Bottom Leg 1
Plinth Beam	P1	Bottom	Uniform	14	14	250	4.62	0.64	Bottom Leg 1
Plinth Beam	P1	Bottom	Uniform	14	14	250	4.62	0.64	Bottom Leg 2
Plinth Beam	P15	Bottom	Uniform	10	10	250	4.54	0.34	Bottom Leg 1
1	P9	Top	Uniform	10	10	250	4.5	0.34	Top Leg 1
Plinth Beam	P13	Bottom	Uniform	10	10	250	4.32	0.3	Bottom Leg 1
Plinth Beam	P2	Bottom	Uniform	14	14	250	4.22	0.67	Bottom Leg 1
Plinth Beam	P2	Bottom	Uniform	14	14	250	4.22	0.67	Bottom Leg 2
Plinth Beam	P8	Bottom	Uniform	14	14	250	4.19	0.66	Bottom Leg 1
Plinth Beam	P8	Bottom	Uniform	14	14	250	4.19	0.66	Bottom Leg 2
Plinth Beam	P12	Bottom	Uniform	10	10	250	3.8	0.3	Bottom Leg 1
Plinth Beam	P9	Bottom	Uniform	10	10	250	3.67	0.34	Bottom Leg 1
Ground FL. Plinth	P9	Top	Uniform	10	10	250	3.62	0.34	Top Leg 1
1	P8	Bottom	Uniform	14	14	250	3.41	0.66	Bottom Leg 1

Fig 30: Maximum reinforcement in shear walls (Alternative 2).

3) *Alternative 3:* (G+14) MHADA BUILDING WITH SHEAR WALLS AT EXTERIOR CORNERS AND EXPANSION JOINT. Response spectrum analysis is carried out and identify the modal participation mass ratios, deformations. The objective of this analysis is to identify the torsional mode of vibration, eccentricity in x and y direction, maximum displacement, area of reinforcement required for columns and shear walls.

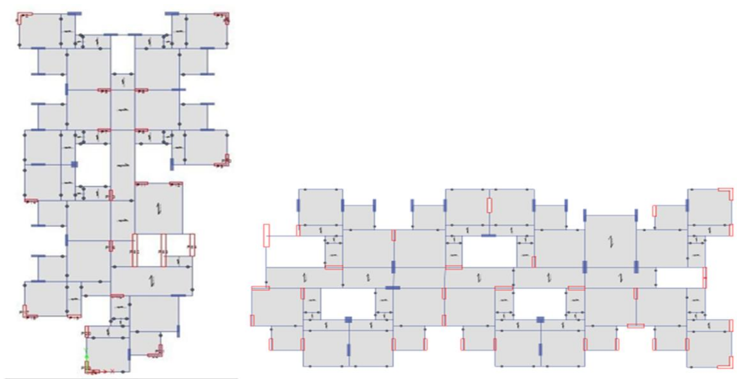

Fig 31: Plan view of the building (Alternative 3).

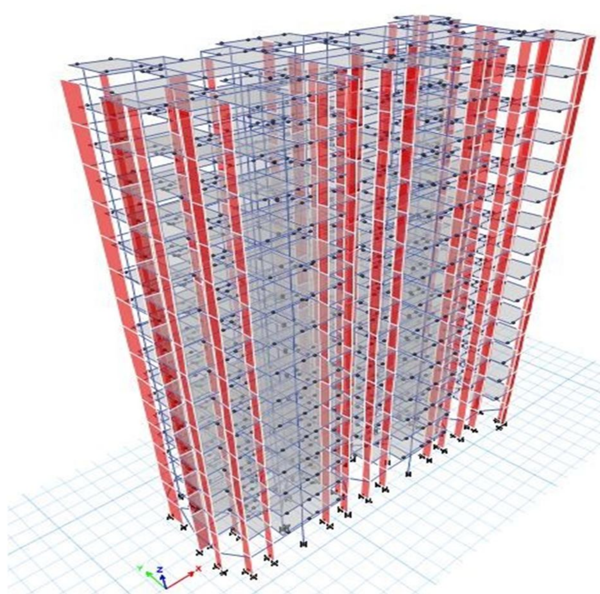

Fig 32: 3D view of X- building (Alternative 3).

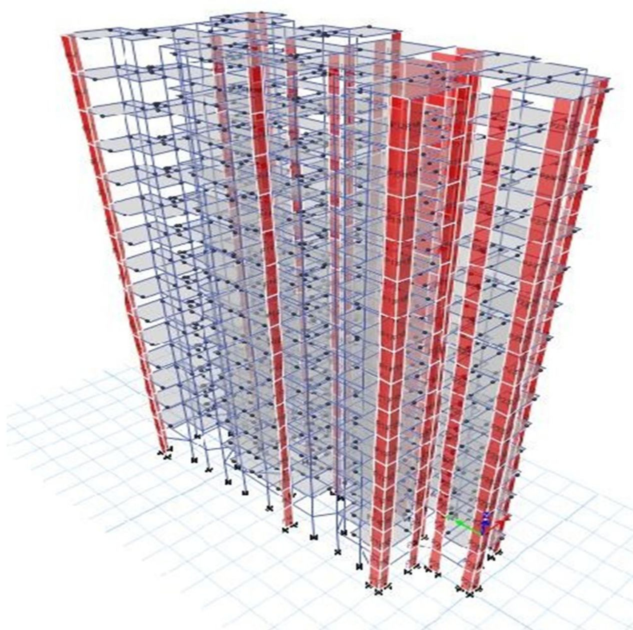

Fig 33: 3D view of Y- building (Alternative 3).

Table No 13: Modal participation mass ratios X building Alternative 3.

Mode	Period	UX	UY	UZ.	RX	RY	RZ
	sec						
1	1.997	0.818	0.0015	0	0.0004	0.1856	0.0001
2	1.645	0.0018	0.709	0	0.2086	0.0004	0.0689
3	1.569	0.00001009	0.069	0	0.0216	0.000003682	0.6961
4	0.647	0.1048	0.0002	0	0.0007	0.5819	0.0001
5	0.497	0.0002	0.1257	0	0.4713	0.0008	0.0126
6	0.463	0.000006348	0.0122	0	0.0436	0.000002617	0.1296
7	0.364	0.0335	0.00003749	0	0.0001	0.0685	0.0003
8	0.256	0.00002261	0.033	0	0.0797	0.0001	0.0033
9	0.244	0.0151	0	0	0.000005166	0.0682	0.0004
10	0.235	0.000003747	0.0034	0	0.0082	0.0001	0.0366
11	0.176	0.0078	0.000001958	0	0.000007039	0.022	0.00003652
12	0.164	0.000005568	0.0148	0	0.0568	0.00002358	0.0013

Table No 14 : Center of mass and Rigidity (Eccentricity in x and y direction) X building Alternative 3.

Story	XCM	YCM	XCCM	YCCM	XCR	YCR	ECC. X	E.C.C. Y
	m	m	m	m	m	m	m	m
Terrace	23.9905	7.4063	23.9905	7.4063	24.1096	7.5698	0.1191	0.1635
12	24.0339	7.4114	24.0166	7.4094	24.1198	7.5701	0.0859	0.1587
11	23.9529	7.4186	23.9924	7.4129	24.1343	7.5736	0.1814	0.155
10	23.9461	7.4183	23.9797	7.4144	24.1502	7.5762	0.2041	0.1579
9	23.9499	7.4174	23.9732	7.4151	24.1635	7.5752	0.2136	0.1578
8	23.9499	7.4174	23.9691	7.4155	24.1753	7.5691	0.2254	0.1517
7	23.9499	7.4174	23.9662	7.4158	24.188	7.5561	0.2381	0.1387
6	23.9508	7.4155	23.9642	7.4157	24.2029	7.5339	0.2521	0.1184
5	23.9495	7.4248	23.9625	7.4168	24.221	7.4989	0.2715	0.0741
4	23.9238	7.4163	23.9585	7.4167	24.2489	7.445	0.3251	0.0287
3	23.9272	7.4201	23.9555	7.4171	24.302	7.3613	0.3748	-0.0588
2	23.959	7.4111	23.9558	7.4166	24.3533	7.2451	0.3943	-0.166
1	23.8275	7.4278	23.9457	7.4174	24.4226	7.1352	0.5951	-0.2926
Ground FL. Plinth	23.7971	6.9338	23.9357	7.3848	24.5377	7.1306	0.7406	0.1968
Lower Ground	23.8313	4.9872	23.9307	7.2688	24.7671	6.9303	0.9358	1.9431
Plinth Beam	31.8372	10.646	23.9579	7.2805	31.3917	10.2259	-0.4455	-0.4201

Table No 15 : Maximum story displacement X building Alternative 3

Story	Load Case/Combo	Direction	Maximum	Average	Ratio
			mm	mm	
Terrace	EQX	X	73.514	73.282	1.003
Terrace	RX Max	X	59.447	58.794	1.011
Terrace	RY Max	Y	47.69	41.129	1.16
Terrace	EQY	Y	47.433	45.841	1.035
Terrace	WINDY	Y	28.618	23.795	1.203
Terrace	WINDX	X	9.278	9.207	1.008
Terrace	RY Max	X	5.934	4.148	1.431
Terrace	SIDL	Y	3.374	1.671	2.019
Terrace	Dead	Y	2.484	0.864	2.873
Terrace	SIDL	X	0.775	0.252	3.074
Terrace	Dead	X	0.678	0.181	3.747
Terrace	Live	Y	0.509	0.001	406.361
Terrace	Live	X	0.162	0.007	24.836
Terrace	TLIVE	Y	0.042	0.023	1.797
Terrace	TLIVE	X	0.008	0.003	3.147

Concrete Column PMM Envelope

1 of 608 Reload Apply

Label	Story	Section	Location	P kN	M Major kN-m	M Minor kN-m	PMM Combo	PMM Ratio or Rebar %
C37	2	C.30.105.M30	Bottom	2894.3859	281.7392	-313.7026	1.5DRYN	2.73 %
C37	3	C.30.105.M30	Bottom	2559.8129	337.177	-313.8793	1.5DRYN	2.7 %
C52	Terrace	C.23.1050.M30	Top	95.9839	-106.9957	154.026	1.5DRYN	2.63 %
C37	1	C.30.105.M30	Top	3075.4747	-164.5507	313.7026	1.5DRYN	2.6 %
C51	Terrace	C.23.1050.M30	Top	97.1716	-92.4089	-154.026	1.5DRYN	2.59 %
C37	2	C.30.105.M30	Top	2867.8077	-177.805	313.8793	1.5DRYN	2.54 %
C40	Terrace	C.30.105.M30	Top	76.398	-95.3868	303.7994	0.9DRYN	2.49 %
C37	4	C.30.105.M30	Bottom	2390.0865	-305.9213	304.7708	1.5DRYN	2.48 %
C29	1	C.30.105.M30	Bottom	3147.274	543.9278	221.0899	1.5DRXN	2.3 %
C37	3	C.30.105.M30	Top	2533.2348	-166.5275	304.7708	1.5DRYN	2.26 %
C51	1	C.23.1050.M30	Bottom	2632.8675	111.8169	-133.666	1.5DRYN	2.24 %
C41	Terrace	C.30.105.M30	Top	87.2672	-84.3612	-277.7669	0.9DRYN	2.2 %
C37	5	C.30.105.M30	Bottom	2161.9088	-242.9262	298.0786	1.5DRYN	2.18 %
C52	1	C.23.1050.M30	Bottom	2420.8389	105.4546	-135.6169	1.5DRYN	2.12 %
C37	4	C.30.105.M30	Top	2361.7365	-150.5572	298.0786	1.5DRYN	2.07 %
C41	3	C.30.105.M30	Bottom	2541.9132	-149.6896	291.3242	1.5DRYN	2.07 %
C41	2	C.30.105.M30	Bottom	2755.5166	-151.5861	282.4574	1.5DRYN	2.06 %
C41	1	C.30.105.M30	Top	3222.0831	-10.08	282.4574	1.5DRXN	2.05 %
C37	6	C.30.105.M30	Bottom	1022.7722	-236.9331	284.7947	0.9DRYN	2.03 %
C41	2	C.30.105.M30	Top	2727.1666	-80.413	-291.3242	1.5DRYN	2.01 %
C61	Lower Ground	C.30.105.M30	Bottom	3029.8666	256.1587	-251.2471	1.5DRYN	2.01 %

Fig 34: Maximum reinforcement in column X building (Alternative 3).

Shear Wall Pier Summary - IS 456-2000

1 of 1120 Reload Apply

Story	Pier Label	Station	Design Type	Edge Rebar	End Rebar	Rebar Spacing mm	Required Reinf %	Current Reinf %	Pier Leg
Plinth Beam	P15	Bottom	Uniform	10	10	250	3.77	0.26	Bottom Leg 1
Plinth Beam	P16	Bottom	Uniform	10	10	250	3.7	0.26	Bottom Leg 1
Plinth Beam	P23	Bottom	Uniform	10	10	250	3.67	0.26	Bottom Leg 1
Plinth Beam	P8	Bottom	Uniform	10	10	250	3.57	0.26	Bottom Leg 1
Plinth Beam	P9	Bottom	Uniform	10	10	250	3.4	0.26	Bottom Leg 1
Plinth Beam	P12	Bottom	Uniform	10	10	250	3.3	0.26	Bottom Leg 1
Plinth Beam	P36	Bottom	Uniform	10	10	250	2.95	0.28	Bottom Leg 1
Plinth Beam	P3	Bottom	Uniform	10	10	250	2.75	0.26	Bottom Leg 1
1	P14	Bottom	Uniform	10	10	250	2.74	0.37	Bottom Leg 1
1	P34	Bottom	Uniform	10	10	250	2.47	0.37	Bottom Leg 1
1	P13	Top	Uniform	10	10	250	2.43	0.37	Top Leg 1
1	P34	Top	Uniform	10	10	250	2.43	0.37	Top Leg 1
1	P13	Bottom	Uniform	10	10	250	2.42	0.37	Bottom Leg 1
1	P10	Bottom	Uniform	10	10	250	2.41	0.37	Bottom Leg 1
1	P14	Top	Uniform	10	10	250	2.41	0.37	Top Leg 1
Plinth Beam	P3	Top	Uniform	10	10	250	2.38	0.26	Top Leg 1
1	P10	Top	Uniform	10	10	250	2.38	0.37	Top Leg 1
Lower Ground	P3	Bottom	Uniform	10	10	250	2.34	0.26	Bottom Leg 1
Plinth Beam	P5	Bottom	Uniform	10	10	250	2.31	0.28	Bottom Leg 1
1	P31	Bottom	Uniform	10	10	250	2.31	0.37	Bottom Leg 1

Fig 35: Maximum reinforcement in shear walls X building (Alternative 3).

Table No 16: Modal participation mass ratios Y building Alternative 3.

Mode	Period	UX	UY	UZ.	RX	RY	RZ.
	sec						
1	1.824	0.0001	0.7655	0	0.2409	0.00001433	0.0028
2	1.519	0.6815	0.0007	0	0.0002	0.2148	0.0781
3	1.417	0.0889	0.003	0	0.0008	0.0266	0.6803
4	0.55	0.0002	0.1254	0	0.4668	0.0006	0.0023
5	0.526	0	0.00001845	0	0.0001	0	0.000003706
6	0.463	0.1345	0.0005	0	0.0017	0.4635	0.0141
7	0.431	0.0081	0.0009	0	0.0039	0.0371	0.1261
8	0.28	0.0003	0.0442	0	0.0922	0.0008	0.0022
9	0.243	0.0368	0.0007	0	0.0015	0.0868	0.0055
10	0.226	0.0033	0.0007	0	0.0013	0.0067	0.0368
11	0.174	0.0004	0.0201	0	0.0668	0.0014	0.002
12	0.159	0.0158	0.001	0	0.0034	0.0586	0.0009

Table No 17: Center of mass and Rigidity (Eccentricity in x and y direction) Y building Alternative 3.

Story	XCM	YCM	XCCM	YCCM	XCR	YCR	ECC X	ECC. Y
	m	m	m	m	m	m	m	m
Terrace	2.2608	17.5512	2.2608	17.5512	2.8565	18.1095	0.5957	0.5583
12	2.3177	17.5624	2.2954	17.558	2.855	18.0478	0.5373	0.4854
11	2.3656	17.4575	2.3218	17.5202	2.8503	17.981	0.4847	0.5235
10	2.372	17.4468	2.3355	17.5001	2.8427	17.9114	0.4707	0.4646
9	2.3666	17.4557	2.3422	17.4906	2.8329	17.838	0.4663	0.3823
8	2.3669	17.4502	2.3466	17.4834	2.8228	17.7659	0.4559	0.3157
7	2.372	17.4392	2.3504	17.4768	2.816	17.6928	0.444	0.2536
6	2.372	17.4392	2.3532	17.4719	2.8115	17.6111	0.4395	0.1719
5	2.372	17.4392	2.3554	17.4681	2.8069	17.5195	0.4349	0.0803
4	2.3821	17.4082	2.3582	17.4618	2.8048	17.4206	0.4227	0.0124
3	2.3672	17.4295	2.359	17.4588	2.8093	17.3316	0.4421	-0.0979
2	2.3683	17.4281	2.3598	17.4562	2.8253	17.2697	0.457	-0.1584
1	2.3433	17.4452	2.3585	17.4553	2.8763	17.4211	0.533	-0.0241
Ground FL. Plinth	1.8796	16.6745	2.3242	17.3993	3.0207	17.9371	1.1411	1.2626
Lower Ground	0.7208	15.1386	2.234	17.2722	3.0843	17.9159	2.3635	2.7773
Plinth Beam	4.9626	14.4302	2.2455	17.2602	4.6196	16.0435	-0.343	1.6133

Table No 18: Maximum storey displacement Y building Alternative 3

Story	Load Case/Combo	Direction	Maximum	Average	Ratio
			mm	mm	
Terrace	EQY	Y	57.565	56.334	1.022
Terrace	RX Max	X	55.439	45.147	1.228
Terrace	EQX	X	51.18	47.004	1.089
Terrace	RY Max	Y	46.578	44.502	1.047
Terrace	WINDX	X	29.633	22.876	1.295
Terrace	WINDY	Y	16.252	16.13	1.008
Terrace	RX Max	Y	12.406	6.795	1.826
Terrace	RY Max	X	10.775	5.833	1.847
Terrace	SIDL	Y	3.686	3.248	1.135
Terrace	Dead	Y	2.923	2.609	1.121
Terrace	SIDL	X	2.229	1.305	1.708
Terrace	Dead	X	1.232	0.57	2.16
Terrace	Live	Y	0.808	0.693	1.166
Terrace	Live	X	0.499	0.257	1.94
Terrace	TLIVE	X	0.047	0.04	1.185
Terrace	TLIVE	Y	0.014	0.01	1.34

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Concrete Column PMM Envelope

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Label	Story	Section	Location	P kN	M Major kN-m	M Minor kN-m	PMM Combo	PMM Ratio or Rebar %
C81	Terrace	C.23.1050.M30	Top	102.5919	-83.864	186.479	1.5DRXN	3.16 %
C81	2	C.23.1050.M30	Bottom	2763.5257	211.4334	-146.6952	1.5DRXN	2.95 %
C81	3	C.23.1050.M30	Bottom	2534.6647	193.0785	-154.1578	1.5DRXN	2.93 %
C81	4	C.23.1050.M30	Bottom	2324.7559	184.6368	-156.8574	1.5DRXN	2.88 %
C81	2	C.23.1050.M30	Top	2741.7907	-122.6851	154.1578	1.5DRXN	2.86 %
C81	1	C.23.1050.M30	Top	2994.3534	-118.3865	146.6952	1.5DRXN	2.83 %
C81	3	C.23.1050.M30	Top	2512.9297	-121.5641	156.8574	1.5DRXN	2.78 %
C78	1	C.23.1050.M30	Bottom	2350.8722	267.156	133.5633	1.5DRYN	2.58 %
C86	1	C.23.1050.M30	Bottom	2588.7211	-288.4295	-126.1193	1.5DRXN	2.58 %
C81	5	C.23.1050.M30	Bottom	2080.2845	178.7906	-142.7067	1.5DRXN	2.41 %
C81	4	C.23.1050.M30	Top	2303.0209	-115.8544	142.7067	1.5DRXN	2.27 %
C87	1	C.23.1050.M30	Bottom	2586.6311	248.9107	-118.5936	1.5DRXN	2.26 %
C27	1	C.23.1050.M30	Bottom	2715.1679	193.1916	-119.5221	1.5DRXN	2.2 %
C17	1	C.23.1050.M30	Bottom	2.7715	-32.0181	137.9826	0.9DRXN	2.19 %
C78	Terrace	C.23.1050.M30	Top	121.602	-195.8794	-118.0866	1.5DRYN	2.04 %
C81	6	C.23.1050.M30	Bottom	1798.5733	170.4174	-135.0793	1.5DRXN	2.04 %
C78	3	C.23.1050.M30	Bottom	2001.6774	237.4993	123.2414	1.5DRYN	2.02 %
C78	4	C.23.1050.M30	Bottom	1849.2351	239.9798	124.4522	1.5DRYN	2.01 %
C1	Terrace	C.23.1050.M30	Top	55.2919	152.9251	-118.0866	1.5DRXN	1.98 %
C78	2	C.23.1050.M30	Bottom	2192.4294	238.6726	118.6126	1.5DRYN	1.96 %

Fig 36: Maximum reinforcement in column Y building (Alternative 3).

Shear Wall Pier Summary - IS 456-2000

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Story	Pier Label	Station	Design Type	Edge Rebar	End Rebar	Rebar Spacing mm	Required Reinf %	Current Reinf %	Pier Leg
1	P18	Bottom	Uniform	10	10	250	3.55	0.37	Bottom Leg 1
1	P19	Bottom	Uniform	10	10	250	3.33	0.34	Bottom Leg 1
1	P18	Top	Uniform	10	10	250	3.32	0.37	Top Leg 1
1	P19	Top	Uniform	10	10	250	3.05	0.34	Top Leg 1
Plinth Beam	P12	Bottom	Uniform	10	10	250	2.95	0.28	Bottom Leg 1
1	P5	Bottom	Uniform	10	10	250	2.69	0.37	Bottom Leg 1
1	P13	Bottom	Uniform	10	10	250	2.65	0.37	Bottom Leg 1
1	P7	Bottom	Uniform	10	10	250	2.56	0.37	Bottom Leg 1
3	P13	Bottom	Uniform	10	10	250	2.55	0.37	Bottom Leg 1
3	P13	Top	Uniform	10	10	250	2.48	0.37	Top Leg 1
2	P13	Top	Uniform	10	10	250	2.47	0.37	Top Leg 1
1	P5	Top	Uniform	10	10	250	2.45	0.37	Top Leg 1
1	P7	Top	Uniform	10	10	250	2.4	0.37	Top Leg 1
2	P13	Bottom	Uniform	10	10	250	2.38	0.37	Bottom Leg 1
1	P13	Top	Uniform	10	10	250	2.37	0.37	Top Leg 1
1	P6	Bottom	Uniform	10	10	250	2.32	0.37	Bottom Leg 1
Plinth Beam	P14	Bottom	Uniform	10	10	250	2.3	0.28	Bottom Leg 1
1	P6	Top	Uniform	10	10	250	2.23	0.37	Top Leg 1
Plinth Beam	P7	Bottom	Uniform	10	10	250	2.2	0.28	Bottom Leg 1

Fig 37 : Maximum reinforcement in Shear Y building (Alternative 3).

V. COMPARISON OF RESULTS

A. *Torsional Mode Of Vibration*

Table 19: Comparison of the torsional mode of vibration.

Sr.No.	ALTERNATIVES	Model	Torsion		
			1st	2nd	3rd
1	ALTERNATIVE 1	Original Model (full building, no sw)	33.52%	35.06%	10.53%
2	ALTERNATIVE 2	Full Model with sw @ exterior corners	0.90%	4%	70%
3A	ALTERNATIVE 3A	A) X-Building with sw	0.01%	6.89%	69.60%
3B	ALTERNATIVE 3B	B)Y-Building with sw	0.28%	7.81%	68.03%

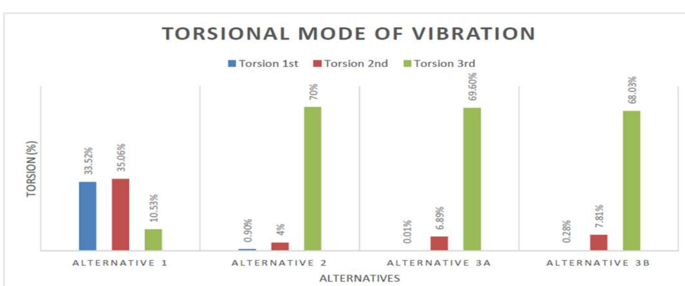

Fig 38: Graphical representation of a Comparison of the torsional mode of vibration.

- *Commentary:* For alternative1, Torsion is in first and second mode of vibration. For alternative 2 and Alternative 3 torsion is in third mode of vibration and first two modes are translation inX and Y direction.

B. Eccentricity In X And Y Direction

Table No 20 : Comparison of eccentricity in x and y-direction.

SR.NO.	ALTERNATIVES	Model	ECC. X	E.C.C. Y
			m	m
1	ALTERNATIVE 1	Original Model (full building, no sw)	0.669	0.026
2	ALTERNATIVE 2	Full Model with sw @ exterior corners	0.1155	0.5408
3A	ALTERNATIVE 3A	A) X-Building with sw	0.1191	0.1635
3B	ALTERNATIVE 3B	B)Y-Building with sw	0.5957	0.5583

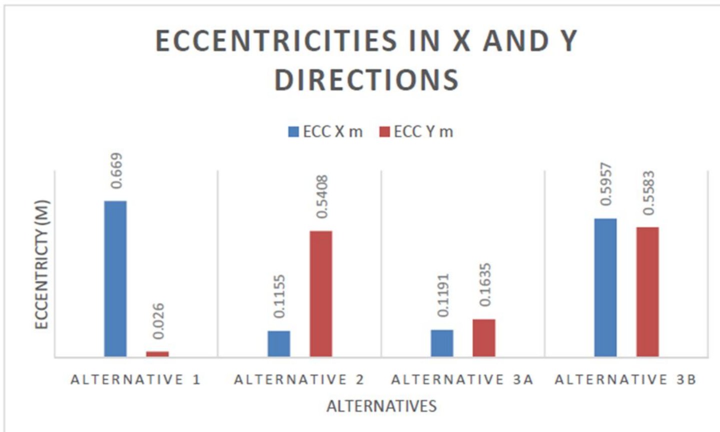

Fig 39 : Graphical representation of Comparison of eccentricity in x and y-direction.

- *Commentary:* For alternative 1, eccentricity in the X direction is high and lesser in the Y direction. For alternative2 Eccentricity in the Y direction is more than in X direction. For alternative 3A eccentricity in X Y-direction is very less, and for alternative case 3 eccentricity in X and Y direction is moderate And have almost the same value For

C. Maximum Top Storey Displacement

Table: Comparison of maximum top storey displacement.

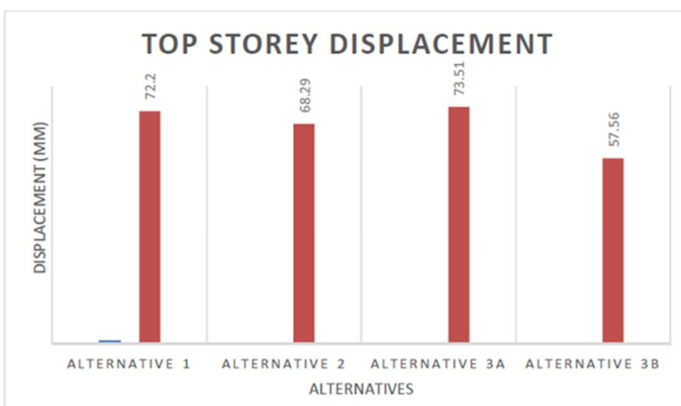

Fig 40 : Graphical representation of Comparison of maximum top story displacement.

- *Commentary:* The maximum top storey displacement is almost the same for all the cases.

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D. Maximum Area Of Reinforcement In Columns And Shear Walls

Table No 21: Comparison of maximum area of reinforcement for columns and shear walls.

SR.NO.	ALTERNATIVES	MODELS	Area of reinforcement
1	ALTERNATIVE 1	Original Model (full building, no sw)	6.2
2	ALTERNATIVE 2	Full Model with sw @ exterior corners	5.81
3	ALTERNATIVE 3A	A) X-Building with sw	3.7
4	ALTERNATIVE 3B	B)Y-Building with sw	3.55

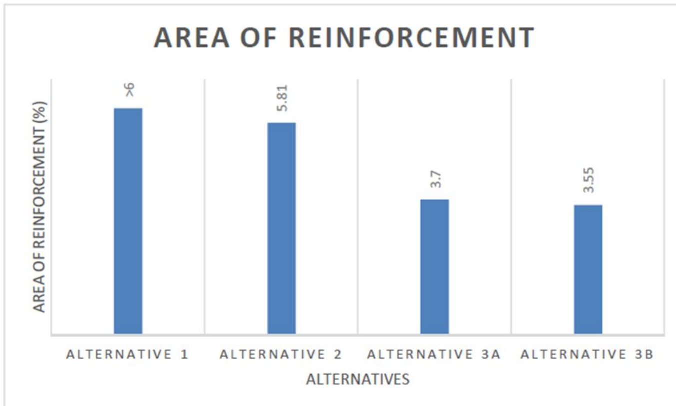

Fig41: Comparison of the maximum area of reinforcement for columns and shear walls.

- *Commentary:* Maximum area of reinforcement required for columns and shear walls is highest forAlternative 1 and Alternative 2 and it less for Alternative 3.

E. Modes of Vibration

MODES OF VIBRATION	ALTERNATIVE 1	ALTERNATIVE 2	ALTERNATIVE 3	.
			X BUILDING	Y BUILDING
MODE 1	TORSIONAL	DIAGONAL TRANSLATION	X TRANSLATION	Y TRANSLATION
MODE 2	TORSIONAL	DIAGONAL TRANSLATION	Y TRANSLATION	X TRANSLATION
MODE 3	Y TRANSLATION + TORSIONAL	TORSIONAL	TORSIONAL	TORSIONAL
MODE 4	X TRANSLATION + TORSIONAL	X TRANSLATION	X TRANSLATION	Y TRANSLATION
MODE 5	TORSIONAL	Y TRANSLATION	Y TRANSLATION	NO VIBRATION
MODE 6	Y TRANSLATION + TORSIONAL	TORDIONAL	TORSIONAL	X TRANSLATION + TORSION
MODE 7	X TRANSLATION + TORSIONAL	X TRANSLATION	X TRANSLATION	TORSIONAL
MODE 8	TORSIONAL	Y TRANSLATION	Y TRANSLATION	Y TRANSLATION
MODE 9	Y TRANSLATION	TORSIONAL	X TRANSLATION	X TRANSLATION
MODE 10	X TRANSLATION	X TRANSLATION	TORSIONAL	TORSIONAL

- *Commentary:* The primary modes of vibrations for alternative 1 are torsional. For alternative 2 theInitial modes of vibrations are diagonal translational. For alternative 3 first two Modes are pure translational in nature.

VI. CONCLUSION

The torsional response of five different cases is studied under a parametric study. Thefollowing are the conclusions of the study.

1) The study compares the torsional mode of vibration for five cases, and these objectivesconclude that Torsion is in the third mode of vibration for only case 4 (L-shaped shearwalls at exterior corners).
2) This study concludes that the period of vibration and eccentricity between the center ofmass and center of rigidity is minimum for case no. 4 this concluded that there is uniformstiffness distribution for case 4.
3) This study concludes that the base shear and ductility factor obtained from pushoveranalysis are maximum for case 5, followed by case 4.
4) This study concluded that the location of plastic hinges obtained from pushover analysis is less in columns and shear walls for case no. 4. This indicates strong columnweak beams philosophy achieved in case 4.
5) This study concludes that for a live project, torsion is in the third mode of vibration for alternatives 2 and 3.
6) This study concludes that eccentricity between the center of mass and rigidity and top storey displacement is almost the same for all the alternatives considered.
7) This study concludes that area of reinforcement for alternative 3 is quite less and feasiblefrom an economical point of view. So alternative 3 is suggested as the most viable and optimum solution for the live project of MHADA building under consideration.
8) This study concludes that for alternative 1 and alternative 2 the initial modes are torsionaland diagonal translational in nature, both are very dangerous in nature. For alternative 3, only the first two modes are pure translational in X and Y direction, and Torsion is shiftedto the third mode of vibration. So, our industry guide, Mr. C. V Patil, sir, suggested that alternative 3 is an excellent structural configuration for MHADA building under consideration.

VII. SCOPE FOR FURTHER WORK

1) In the present study analysis of 14-storey building has been performed using ETABS.The same exercise can be carried out for more tall buildings.
2) The effect of the location of the shear walls can also be studied by shifting these wallssymmetrically towards the centre.
3) Thickness of shear walls throughout the height of building is constant. Analysis can be performed considering different thickness in building height.

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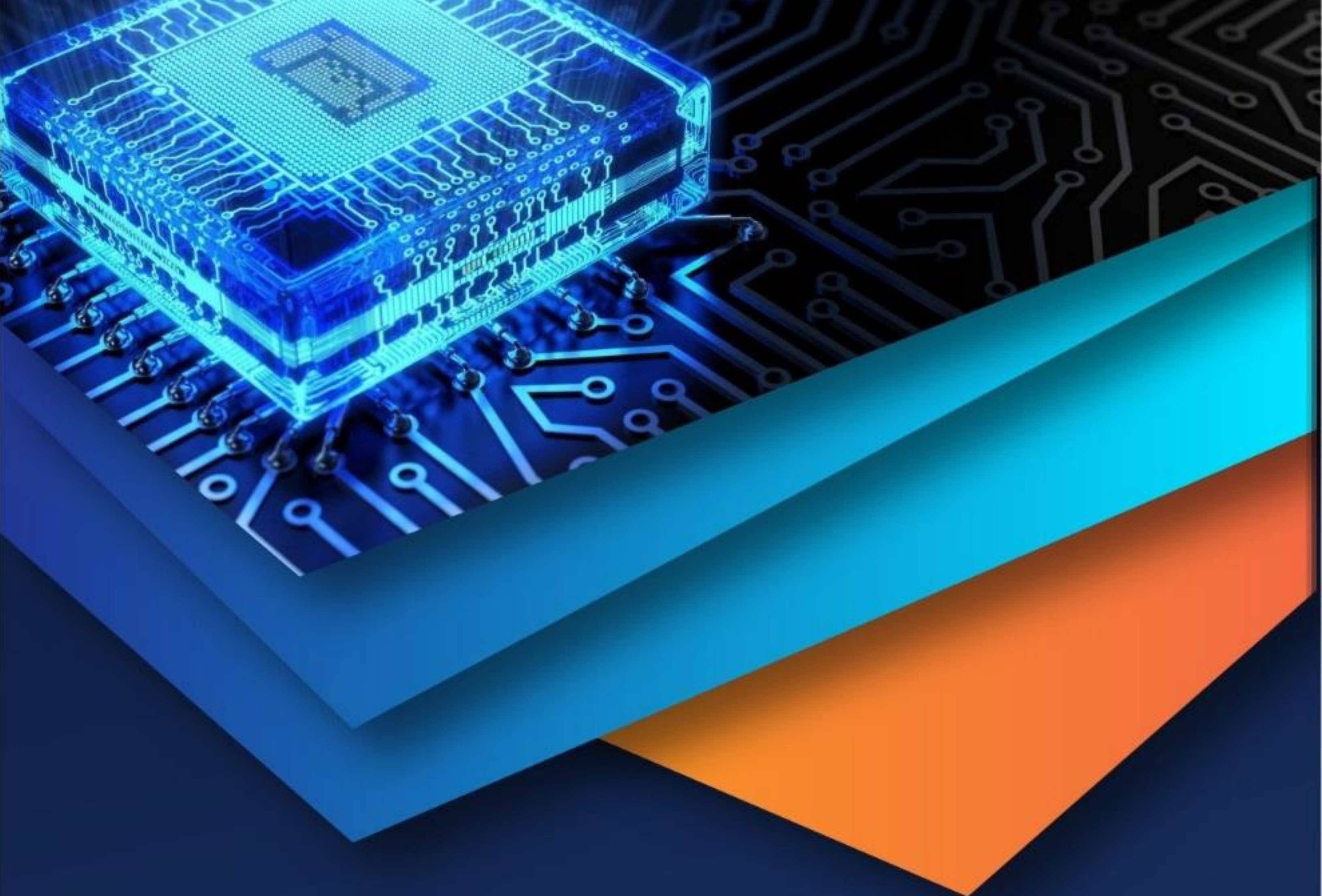

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