STATIC AND DYNAMIC ANALYSIS OF SHEAR WALL SUBJECTED TO LATERAL LOADS

Imam MansurAhemad[1], Patva Vivek[2]
[1] P.G student Civil Engineering(structure), Sakanchand Patel engineering college, Visnagar, Gujarat, India
[2] Assistant professor Civil Engineering(structure), Sakanchand Patel engineering college, Visnagar, Gujarat, India

Abstract- *Due to increase in population spacing in India is needed, especially in urban areas. Also due to increase in the transportation and safety measure the FSI (Floor Spacing Index) in Indian cities is increasing considerably. Structural engineers in the seismic regions across the world often face the pressure to design high rise buildings with stiffness irregularities, even though they know these buildings are vulnerable under seismic loading. Today's tall buildings are becoming more and more slender, leading to the possibility of more sway in comparison with earlier high rise buildings. improving the structural systems of tall buildings can control their dynamic response. With more appropriate structural forms such as shear walls and tube structures and improved material properties. The general design concept of the contemporary bearing wall building system depends upon the combined structural action of the floor and roof systems with the walls. The floor system carries vertical loads and, acting as a diaphragm, lateral loads to the walls for transfer to the foundation. Lateral forces of wind and earthquake are usually resisted by shear walls which are parallel to the direction of lateral load. These shear walls, by their shearing resistance and resistance to overturning, transfer the lateral loads to the foundation. In the present study a 21 story high rise building, with podium up to 3rd floor level is considered. After podium level (3rd floor level), there is no sudden change in plan because if there is any sudden change it may result in the stiffness/torsional irregularities of building if a small seismic forces or any other less magnitude horizontal force strike the structure. The optimization techniques which are used in this project are firstly considered the size of shear wall is same throughout the building and then analysis is done from the result the failed shear wall dimensions are increased to resist the whole structure, in this way the optimization was done for number of time till the whole structure comes to stable to resist the forces .In this present project shear wall design and optimization is done by using the software Etabs and the shear walls are arranged in such a way to resist the lateral forces in zone III region throughout the structure according to Indian codes.*

Keywords- Story Drifts, shear wall, Story Stiffness, base shear , Etabs2017

I. INTRODUCTION

In the 21st century, there has been the tremendous growth in the infrastructure development in the developing countries, especially India, in terms of construction of buildings, bridges and industries etc. This infrastructure development is mainly due to the growing population and to fulfill their demands. Since the land is limited, there is a huge scarcity of land in urban cities. To overcome this problem tall and slender multi-storied buildings are constructed. There is a high possibility that such structures are subjected to huge lateral loads. These lateral loads are generated either due to wind blowing against the building or due to inertia forces induced by ground shaking (excitation) which tends to snap the building in shear and push it over in bending. In the framed buildings, the vertical loads are resisted by frames only, however, the lateral resistance is provided by the infill wall panels. For the framed buildings taller than 10-stories, frame action obtained by the interaction of slabs and columns is not adequate to give required lateral stiffness and hence the framed structures become an uneconomical solution for tall buildings. The lateral forces due to wind and earthquake are generally resisted by the use of shear wall system, which is one of the most efficient methods of maintaining the lateral stability of tall buildings. In practice, shear walls are provided in most of the commercial and residential buildings up to 30th storey beyond which tubular structures are recommended. Shear walls may be provided in one plane or in both planes. The typical shear wall system with shear walls located in both the planes and subjected to lateral loads is shown in Fig. 1.1(a). In such cases, the columns are primarily designed to resist gravity loads.

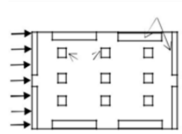


Fig. 1.1(a).

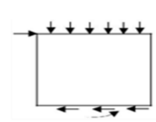


Fig. 1.1(b).

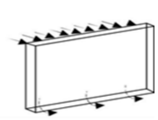


Fig. 1.1(c).

The shear walls are expected to resist large lateral loads (due to earthquake or wind) that may strike "in-plane" [Fig. 1.1(b)] and "out-of-plane" [Fig. 1.1(c)] to the wall. The in-plane shear resistance of the shear wall can be estimated by subjecting the wall to the lateral loads as shown in Fig. 1.1(b). On the other hand, the flexural capacity can be estimated by subjecting the shear wall to the out-of-plane lateral loads as shown in Fig. 1.1(c). During extreme earthquake ground motions, the response of a structure is dependent upon the amount of seismic energy fed in and how this energy is consumed. Since the elastic capacity of the structure is limited by the material strength, survival generally relies on the ductility of the structure to dissipate energy. At higher loads, inelastic deformation arises which are permanent and imply some damage. The damages generally vary from minor cracks to major deterioration of structure, which may be beyond repair. It has been learnt from past experiences that the shear wall buildings exhibit excellent performance during the severe ground motion due to stiff behavior at service loads and ductile behavior at higher loads thus preventing the major damage to the RC buildings (Fintel, 1977). The behavior of shear wall can be ascertained well by observing the deflected shape. The deflected shape of the tall shear wall is dominated by flexure and that of short shear walls by shear as shown in Fig. 1.2.

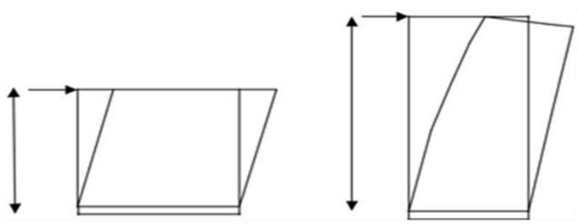


FIG. 1.2.

II. MODEL INTERPRETATION

TOWER

The proposed building consists of Ground floor + 6 Podium floors + 1 Service floor + 1 Fire check + 31 habitable floors apart from the terrace floor. No floor provision is accounted in the design. The size of the structure at typical floor level is around 38.3 m X 35.6 m. The podium floors are for car parking and services. There is a service floor above 6th podium floor. The upper floors are for residential use only, while parts of 1st, 8th, 15th, 22nd ,29th floors are assigned as the refuge area. There is a fire check floor above 12th floor.

The proposed building consists of Ground floor + 6 Podium floors + 1 Service floor + 1 Fire check + 32 habitable floors apart from the terrace floor. No floor provision is accounted in the design. The size of the structure at typical floor level is 54 m X 45.6 m. The podium floors are for car parking and services. There is aservice floor above 6th podium floor. The upper floors are for 7/16 residential use only, while parts of 1st, 8th, 15th, 22nd ,29th floorsare assigned as the refuge area .There is a fire check floor above 12th floor.RCC beam and slab system is adopted as the general framing system for all the towers at typical floor level, whereas flat slab system is adopted in the podium portion. Shear walls are included at places wherever possible to resist the lateral loads effectively.

CONSTRUCTION MATERIAL

Concrete shall comply with IS 456-2000. Unless noted otherwise concrete is to be normal-weight, with a typical dry density of 25 KN/m3. The following list shows the 28-day cube strength (minimum) to be achieved by the concrete for some main structural members.

- Columns 50 to 30 N/mm2
- Slabs and Beams 40 to 30 N/mm2
- Foundation 40 N/mm2
- Retaining wall 40 N/mm2
- P.C.C. 20 N/mm2

LOADING

The building is analyzed for the following basic load cases
- Dead Load
- Live Load
- Seismic Load
- Wind Load

DEAD LOAD

The dead load comprises of self-weight of the structure and loading due to finishes, flooring etc. which are permanent in nature. The dead load consisting of self-weight, partitions, ceiling, flooring, facade etc. are applied as either area loads to slabs or line loads to beams. The following parameters are considered as per IS 875 (Part I) – 1987
- Finishes including services load = 1.5 Kn/m2
- Facade load = as per Actual.
- Density of Concrete = 25 Kn/m3
- Density of Light-weight Blocks = 8 Kn/m3
- Density of Plain Concrete = 24 Kn/m3
- Density of Steel = 78 Kn/m3
- Density of Aluminum = 28 Kn/m3
- Density of glass = 25 Kn/m3
- Density of soil filling = 20 Kn/m3
- Density of Brick Bat = 21 Kn/m3
- Partition walls = As per actual

All filling material in toilets, dry balconies, flower beds and terrace water proofing to be of Brick Bat. The earth pressure loading will be taken from the soil consultant's report for the design of the soil retention elements.

LIVE (IMPOSED) LOAD

Live Loads considered on floor slabs are as per
IS 875 (Part 2) – 1987

- Residential area = 2.0 Kn/m2
- Corridors, Lift Lobbies = 3.0 Kn/m2
- Car Parking Area = 2.5 Kn/m2
- Refuge Area = 5.0 Kn/m2
- Wash Room (Toilet areas) = 2.0 Kn/m2
- Staircases = 3.0 Kn/m2
- Lift machine room area = 10.0 Kn/m2
- General Terrace area = 1.5 Kn/m2
- Terrace without access = 0.75 Kn/m2
- A.H.U, server room, chiller & other = As per actual

SEISMIC LOAD

IS 1893 – 2002 is used for calculating seismic load and as the basis for seismic load combinations. Following parameters are considered.
Seismic Zone = 3 (Refer table 2 of IS 1893 – 2016)
Z = 0.16
Response Reduction Factor (Refer table 2 of IS 1893 – 2016)
R = 4.0 (For Tower A, A1) R = 4.5 (For Tower A)
R = 3.0 (For Podium Portion)
Soil Type = 1
Damping = 5%
Importance factor = 1.0
Ta = 0.09h/√d (For Tower Portion)
Ta = 0.075h0.75 (For Podium Portion)
h= height of structure
d = Base dimension along the bldg.

WIND LOAD

IS 875 Part 3 -2015 is used for finding out the wind pressure.
Basic wind speed Vb = 44m/sec
Design Wind Speed Vz = Vb x k1 x k2 x k3
Where K1 is Risk Factor
K2 is a Terrain, Height and Structure size Factor
K3 is a Topography Factor
K1 = 1 Refer Table - 1 IS
Design Pressure (Pz) = 0.6 VZ2
Since the building external faces are 875 - 1987 - Part – 3
K2 = Refer table - 2 of IS 875 - 1987 - Part – 3
(Category = 3 Class C Structure)
K3 = 1.0 Refer Clause -5.3.3.1 IS 875 -1987 - Part – 3

LOAD COMBINATION

1. Primary Load cases:
2. Dead Load (D. L.)
3. Live Load (L. L.)
4. Eqx (X- dir. earthquake)
5. Eqy (Y- dir. earthquake)
6. W90 (90 degrees - dir. Wind)
7. W210 (210 degree- dir. Wind)
8 W330 (330 degree-dir. Wind)

ii) Basic Load combinations:
1. 1.5(D. L. + L. L.)
2. 1.2(D. L. + L. L.+- Eqx,y)
3. 1.5(D. L. + Eq-x,y)
4. (0.9D. L. +- 1.5Eqx,y)
5. (0.9D. L. +- 1.5Specx,y)
6. 1.5(D. L. + Eq-Specx,y)
7. 1.2(D. L. + L. L.+- Specx,y)

III. ANALYSIS

Equivalent Static Analysis.

All design against earthquake effects must consider the dynamic nature of the load. However, for simple regular structures, analysis by equivalent linear static methods is often sufficient. This is permitted in most codes of practice for regular, low- to medium-rise buildings and begins with an estimate of peak earthquake load calculated as a function of the parameters given in the code. Equivalent static analysis can therefore work well for low to medium-rise buildings without significant coupled lateral–torsional modes, in which only the first mode in each direction is of significance. Tall buildings (over, say, 75 m), where second and higher modes can be important, or buildings with torsional effects, are much less suitable for the method, and require more complex methods to be used in these circumstances.

Dynamic Analysis

Structural design of buildings for seismic loading is primarily concerned with structural safety during major earthquakes, but serviceability and the potential for economic loss are also of concern. Seismic loading requires an understanding of the structural behavior under large inelastic deformations. Behavior under this loading is fundamentally different from wind or gravity loading, requiring much more detailed analysis to assure acceptable seismic performance beyond the elastic range. Some structural damage can be expected when the building experiences design ground motions because almost all building codes allow inelastic energy dissipation in structural systems.

Analysis of Time Period

As per IS 1893:2016 The approximate fundamental natural period of vibration (T), in seconds, of a moment-resisting frame building without brick infill panels may be estimated by the empirical expression:

$$T_a = \frac{0.075h^{0.75}}{\sqrt{A_w}} \geq \frac{0.09h}{\sqrt{d}}$$

SHEAR WALL	B	D	AWix	AWiy
C1	0.35	2.4	0.03470493	0.041535622
C1A	0.3	1.95	0.024058813	0.027845771
C2	0.23	2.52	0.023683617	0.028948582
C3	0.23	2.52	0.023683617	0.028948582
C4	0.23	2.52	0.023683617	0.028948582
C5	0.23	2.4	0.022555826	0.027294837
C6	0.35	2.4	0.03470493	0.041535622
C6A	0.3	1.95	0.024058813	0.027845771
C7	0.3	2.15	0.026526384	0.031228581
C8	0.3	2.15	0.026526384	0.031228581
C9	0.3	4	0.049351412	0.067561394
C10	0.3	2.45	0.03022774	0.03649615

SUMMETION OF AWix & AWiy	1.813387518 2.430387881

Tx	1.85
Ty	1.60

Analysis of Base Shear

Base shear is the maximum expected lateral force that will occur due to seismic ground acceleration at the base of the structure [1]. The base shear, or earthquake force, is given by the symbol "VB". The weight of the building is given as the symbol "W".

$$A_h = \frac{\left(\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)}{\left(\frac{R}{I}\right)}$$

Z	Sa/g	R	I	Ah	Vb=Ah*W
0.16	0.540	5	1	0.216	71554
0.24	0.625	5	1	0.25	82817

Analysis of Gust Factor

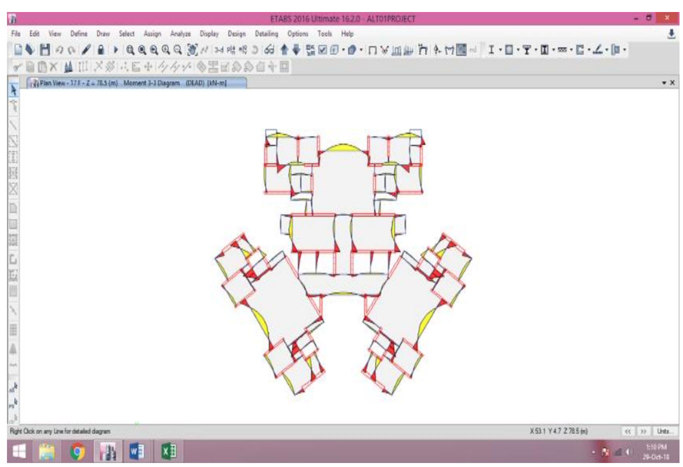
Tall and slender structures are flexible and exhibit a dynamic response to wind. Tall structures vibrate in wind due to the turbulence inherent in the wind as well as that generated by the structure itself due to separation of the flow. Thus there is a mean and a fluctuating response to the wind. Besides, the dynamic forces act not only in the direction of wind flow but also in a direction nearly perpendicular to the flow (lift forces), so that tall structures also exhibit an across-wind response.

FLOOR	H	Hi	K2	Pz	Bsx	Bsy	Hs	Sx	Sy	Nx	Ny	GX	GY
TRFL	3.6	107.3	0.79	0.74	0.9	0.8	2	0.13	0.11	1.32	1.32	7.48	7.31
24 F	3.6	103.7	0.79	0.73	0.9	0.8	1	0.13	0.11	1.32	1.32	7.49	7.32
23 F	3.6	100.1	0.78	0.72	0.9	0.8	1	0.13	0.11	1.31	1.31	7.49	7.32
22 F	3.6	96.5	0.78	0.71	0.9	0.8	1	0.12	0.11	1.31	1.31	7.49	7.32
21 F	3.6	92.9	0.77	0.70	0.8	0.8	1	0.12	0.11	1.31	1.31	7.48	7.32
20 F	3.6	89.3	0.77	0.69	0.8	0.8	1	0.12	0.11	1.30	1.30	7.47	7.32
19 F	3.6	85.7	0.76	0.68	0.8	0.8	1	0.12	0.11	1.30	1.30	7.46	7.31
18 F	3.6	82.1	0.76	0.67	0.8	0.8	1	0.12	0.10	1.26	1.29	7.44	7.30
17 F	3.6	78.5	0.75	0.66	0.8	0.8	1	0.12	0.10	1.22	1.29	7.42	7.29
16 F	3.6	74.9	0.75	0.65	0.8	0.8	1	0.12	0.10	1.28	1.28	7.40	7.27
15 F	3.6	71.3	0.74	0.64	0.8	0.8	1	0.12	0.10	1.28	1.28	7.38	7.26
14 F	3.6	67.7	0.73	0.63	0.8	0.8	1	0.12	0.10	1.27	1.27	7.36	7.24
13 F	3.6	64.1	0.73	0.62	0.8	0.8	1	0.11	0.10	1.27	1.27	7.34	7.22
12 F	3.6	60.5	0.72	0.61	0.8	0.8	1	0.11	0.10	1.26	1.20	7.32	7.21
11 F	3.6	56.9	0.71	0.59	0.8	0.7	1	0.11	0.10	1.26	1.20	7.30	7.19
10 F	3.6	53.3	0.70	0.58	0.8	0.7	1	0.11	0.09	1.25	1.24	7.27	7.17
9 F	3.6	49.7	0.70	0.57	0.7	0.7	1	0.11	0.09	1.24	1.24	7.25	7.15
8 F	3.6	46.1	0.69	0.55	0.7	0.7	1	0.11	0.09	1.24	1.24	7.23	7.13
7 F	3.6	42.5	0.68	0.53	0.7	0.7	1	0.10	0.09	1.23	1.23	7.21	7.12
6 F	3.6	38.9	0.66	0.52	0.7	0.7	1	0.10	0.09	1.22	1.22	7.19	7.10
5 F	3.6	35.3	0.65	0.50	0.7	0.7	1	0.10	0.08	1.22	1.22	7.17	7.08
4 F	3.6	31.7	0.64	0.48	0.7	0.7	1	0.10	0.08	1.21	1.21	7.14	7.06
3 F	3.6	28.1	0.62	0.45	0.7	0.7	1	0.09	0.08	1.20	1.20	7.12	7.04
2 F	3.6	24.5	0.61	0.43	0.7	0.6	1	0.09	0.08	1.10	1.19	7.10	7.02
1 F	3.6	20.9	0.59	0.40	0.6	0.6	1	0.09	0.07	1.12	1.19	7.08	7.01
Pod 3	3.6	17.3	0.56	0.37	0.6	0.6	1	0.08	0.07	1.15	1.18	7.06	6.99
Pod 2	3.6	13.7	0.53	0.33	0.6	0.6	1	0.07	0.06	1.18	1.18	7.04	6.96
Pod 1	3.6	10.1	0.49	0.28	0.6	0.6	1	0.07	0.06	1.17	1.17	7.01	6.94
0	6.5	6.5	0.44	0.22	0.5	0.5	1	0.06	0.05	1.18	1.18	6.82	6.74

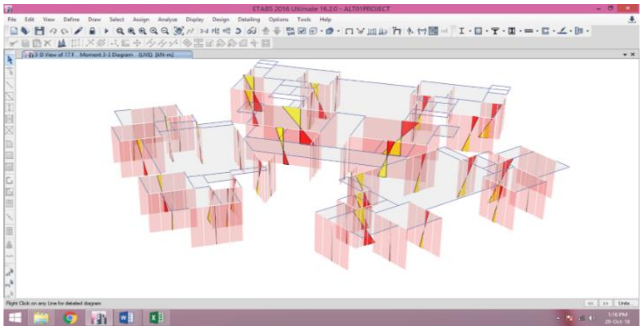
Model Frequency Analysis

Modal	2	2.816	0.355	2.2311	4.9777
Modal	3	2.545	0.393	2.4687	6.0946
Modal	4	0.942	1.061	6.669	44.476
Modal	5	0.828	1.208	7.5887	57.589
Modal	6	0.698	1.432	8.998	80.963
Modal	7	0.493	2.028	12.745	162.44
Modal	8	0.431	2.322	14.591	212.88
Modal	9	0.336	2.975	18.694	349.48
Modal	10	0.326	3.069	19.281	371.75
Modal	11	0.279	3.587	22.539	507.99
Modal	12	0.224	4.457	28.006	784.33

Beam Moment and Reaction

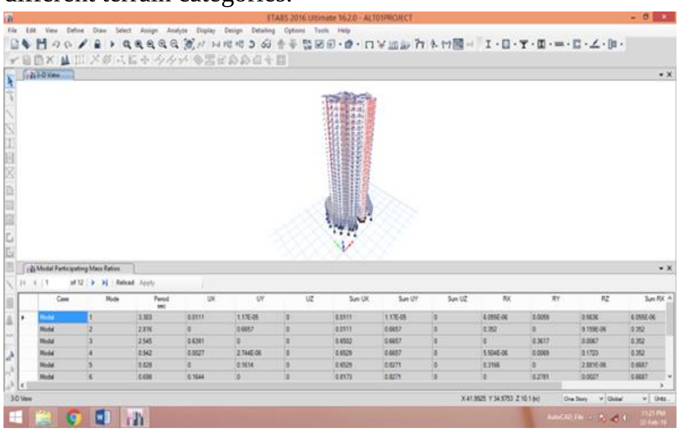


Wall Reaction and Moment

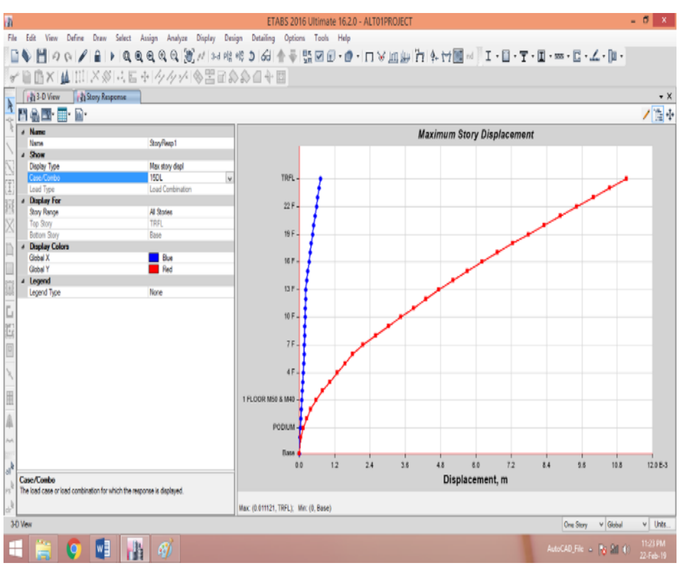


Structure Torsion Check

The procedure followed for computing wind load is same as laid down in IS 875(Part 3)-1987. However, hourly mean wind speeds used are based on statistical analysis of hourly mean wind speed data available in literature instead of using conversion table given in the code for converting 3-second winds to hourly mean wind speeds at various heights in different terrain categories.



Story Drift in Structure



MODEL FACTOR AS PER IS 1893

NAME	PERIOD	ACCELERATION	DAMPING	Z	SOIL TYPE
IS 1893	0	0.16	5	0.16	I
IS 1893	0.1	0.4	5	0.16	I
IS 1893	0.4	0.4	5	0.16	I
IS 1893	0.6	0.266667	5	0.16	I
IS 1893	0.8	0.2	5	0.16	I
IS 1893	1	0.16	5	0.16	I
IS 1893	1.2	0.133333	5	0.16	I
IS 1893	1.4	0.114286	5	0.16	I
IS 1893	1.6	0.1	5	0.16	I
IS 1893	1.8	0.088889	5	0.16	I
IS 1893	2	0.08	5	0.16	I
IS 1893	2.5	0.064	5	0.16	I
IS 1893	3	0.053333	5	0.16	I
IS 1893	3.5	0.045714	5	0.16	I
IS 1893	4	0.04	5	0.16	I
IS 1893	4.5	0.04	5	0.16	I
IS 1893	5	0.04	5	0.16	I
IS 1893	5.5	0.04	5	0.16	I
IS 1893	6	0.04	5	0.16	I
IS 1893	6.5	0.04	5	0.16	I
IS 1893	7	0.04	5	0.16	I
IS 1893	7.5	0.04	5	0.16	I
IS 1893	8	0.04	5	0.16	I
IS 1893	8.5	0.04	5	0.16	I
IS 1893	9	0.04	5	0.16	I
IS 1893	9.5	0.04	5	0.16	I
IS 1893	10	0.04	5	0.16	I

I. CONCLUSION

On the basis of exhaustive numerical studies carried out to identify the limiting opening size and desirable locations of the openings at different floors of the multistoried shear wall, the following conclusions have been drawn.

•For openings up to 14%, the load carrying capacity and ultimate displacement response were not found to be severely affected by openings. However, for openings beyond 14%, the load carrying capacity of slender as well as squat shear wall gets affected due to the presence of openings

•In general, strengthening of shear wall around openings was found beneficial in improving the load carrying capacity and ductility of the shear wall. However, for shear wall up to 14% opening, the responses of both slender as well as squat shear walls were not overly dependent on the strengthening around the openings. On the other hand, beyond 14% opening, the performance of shear wall was found to be strongly influenced due to strengthening around the openings.

•The shear wall with 18% opening was not considered very safe in case of squat shear wall. However, slender shear wall exhibited better performance than squat shear wall for the same opening size.

•The squat shear wall with 21% opening suffered from severe degradation in the load carrying capacity of more than 50% and hence such large openings are to be strictly avoided.

The performance of slender shear wall in the presence of 21% opening was slightly better than that of squat shear wall. In spite of improved performance of slender shear wall over squat shear wall, 21% opening is not considered safe. Hence the shear wall with 14% opening is identified as safe opening size.

•The opening orientation in shear wall significantly affected the performance of shear wall. In case of rectangular openings, it is beneficial to provide the shorter side of the opening parallel to the loading direction in order to minimize the degradation in the load carrying capacity and ductility.

•The shear wall with door cum window opening as well as with two door opening resulted in satisfactory time history displacement response for both 5% and 2% damping. Moreover, for both opening combinations the reduction in the load carrying capacity was found to be less as compared to solid shear wall.

•For shear walls with two/three windows aligned horizontally at the same level, the load carrying capacity and ultimate displacement were found to be severely affected and resulted in the kind of storey mechanism. Moreover, the maximum displacement response was found to be very high under severe dynamic ground motion. It is suggested to avoid such openings to avoid detrimental effect on shear wall. The degradation was more severe for squat shear walls than for slender shear walls especially for 2% damping. Even the strengthening was not found to be positively influencing the behavior of shear wall.

•The aspect ratio of opening plays the crucial role on the structural response of shear wall. The degradation in the strength and displacement was found to be minimal for shear wall with four windows placed symmetrically. The strengthening around the openings as well as damping was not impacting the behavior of the shear wall significantly. terrain conditions (roughness), and are also affected by transitional flow regimes (specifically, changes in terrain and the distance from the upstream terrain change to the measuring device).

REFERENCES

[1] Lefas, I.D., Kotsovos, M.D., and Ambraseys, N.N. (1990). "Behavior of reinforced concrete structural walls: Strength, deformation characteristics, and failure mechanism.".

[2] IS 1893 (2016). Criteria for earthquake resistant design of structures, BIS, New Delhi

[3] IS 13920 (1993). Code of practice for ductile detailing of reinforced concrete structures.

[4] IS 1786 Foundation Movement

[5] Dr H.J SHAH Design of Rcc Structure

[6] S Ramamurthy Structure Design

[7] Apurba Mondal, Siddhartha Ghosh, and GR Reddy. Performance-based evaluation of the response reduction factor for ductile rc frames. Engineering structures, 56:1808{1819, 2013.

[8] CVR Murty. IITK-BMTPC Earthquake Tips: Learning Earthquake Design and Construction. National Information Centre of Earthquake Engineering, Indian Institute of Technology Kanpur, 2005.

[9] Bungale S Taranath. Reinforced concrete design of tall buildings. CRC Press, 2009.

[10] CVR Murty, Rupen Goswami, AR Vijayanarayanan, and Vipul V Mehta. Some concepts in earthquake behaviour of buildings. Gujarat State Disaster Management Authority, Government of Gujarat, 2012.

[11] CVR Murty and Sudhir K Jain. Bene cial in uence of masonry in ll walls on seismic performance of rc frame buildings. In 12th World Conference on Earthquake Engineering (Auckland, New Zealand, January 30), 2000.

[12] Robin Davis, Praseetha Krishnan, Devdas Menon, and A Meher Prasad. E ect of in ll sti ness on seismic performance of multi-storey rc framed buildings in india. In 13 th World Conference on Earthquake Engineering. Vancouver, BC, Canada, 2004.

[13] Hemant B Kaushik, Durgesh C Rai, and Sudhir K Jain. Stress-strain characteristics of clay brick masonry under uniaxial compression. Journal of materials in Civil Engineering, 19(9):728{739, 2007.

[14] Santiago Pujol, Amadeo Benavent-Climent, Mario E Rodriguez, and J Paul Smith-Pardo. Masonry in ll walls: an e ective alternative for seismic strengthening of low-rise reinforced concrete building structures. In 14th World Conference on Earthquake Engineering (Beijing, China, Unknown Month October 12), 2008.

[15] GV Mulgund and AB Kulkarni. Seismic assessment of rc frame buildings with brick masonry in lls. International Journal of advanced engineering sciences and technologies, 2(2):140{147, 2011.

[16] J Prakashvel, C UmaRani, K Muthumani

[17] N Sivakumar, S Karthik, S Thangaraj, S Saravanan, and CK Shidhardhan. Seismic vulnerability of open ground oor columns in multi storey buildings. International Journal of Scienti c Engineering and Research (IJSER), 2013.

[18] MS Lopes. Experimental shear-dominated response of rc walls: Part i: Objectives, methodology and results. Engineering Structures, 23(3):229{239, 2001.

[19] Rahul Rana, Limin Jin, and Atila Zekioglu. Pushover analysis of a 19 story concrete shear wall building. In 13th World Conference on Earthquake Engineering Vancouver,

BC, Canada August 1-6, 2004 Paper No, volume 133, 2004.

[20] Han-Seon Lee and Dong-Woo Ko. Seismic response characteristics of high-rise rc wall buildings having di erent irregularities in lower stories. Engineering structures, 29(11):3149{3167, 2007.

[21] O Esmaili, S Epackachi, M Samadzad, and SR Mirghaderi. Study of structural rc shear wall system in a 56-story rc tall building. In The 14th world conference earthquake engineering, 2008.

[22] YM Fahjan, J Kubin, and MT Tan. Nonlinear analysis methods for reinforced concrete buildings with shear walls. 14th econference in Earthquake Engineering, 2010.

[23] H Gonzales and Francisco Lopez-Almansa. Seismic performance of buildings with thin rc bearing walls. Engineering Structures, 34:244{258, 2012.

[24] Luca Martinelli, Paolo Martinelli, and Maria Gabriella Mulas. Performance of ber beam{column elements in the seismic analysis of a lightly reinforced shear wall. Engineering Structures, 49:345{359, 2013.

[25] C Todut, D Dan, and V Stoian. Theoretical and experimental study on precast reinforced concrete wall panels subjected to shear force. Engineering Structures, 80:323{338, 2014.

[26] Xinzheng Lu, Linlin Xie, Hong Guan, Yuli Huang, and Xiao Lu. A shear wall element for nonlinear seismic analysis of super-tall buildings using opensees. Finite Elements in Analysis and Design, 98:14{25, 2015.

[27] John B Mander, Michael JN Priestley, and R Park. Theoretical stress-strain model for con ned concrete. Journal of structural engineering, 114(8):1804{1826, 1988.

[28] J Enrique Mart nez-Rueda and AS Elnashai. Con ned concrete model under cyclic load. Materials and Structures, 30(3):139{147, 1997.

[29] M Menegotto. Pinto,(1973), pe, method of analysis for cyclically loaded reinforced concrete plane frames including changes in geometry and non-elastic