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Effect of Planar Solid Shear Wall - Frame Arrangement on the Deformation Behaviour of Multi-Storey Frames

Aginam, C. H., Chidolue, C. A[2]. And Ubani, O. U[3].
[1,2,3] *Department of Civil Engineering, Nnamdi Azikiwe University, Awka*
PMB 5025, Awka Anambra State

Abstract: *In this paper, four different solid shear wall-frame arrangements of equivalent stiffnesses were symmetrically placed and considered on a 15- storey (45m) rigid reinforced concrete framed office building; to withstand a wind induced net surface pressure of 0.25 KN/m^2. The three dimensional modelling and analysis of the different shear wall - frame arrangements were carried out on Staad Pro V8i software, and the deformation of the structure in the x, y, z global coordinates (y – upwards) were compared when the wind is coming in the z direction of the structure. All the shear walls were modelled to resist lateral load with respect to the z coordinate axis only. This is peculiar to cases where the shear walls are not applied in the core areas of structures like elevators and stairwells. The results show that the best arrangement in terms of minimal storey displacement was the one where the shear walls were positioned away from the centroidal axis of the frame, parallel to the direction of the wind.*
Keywords: *Displacement, finite element modelling, shear wall, Staad Pro, Tall building.*

I. Introduction

Shear walls are important structural components of the modern high rise apartments and other tall buildings. Basically, the major functions of shear walls are in three folds:

i. To resist lateral forces such as those induced by wind and seismic forces such as earthquakes and blasts
ii. To support vertical loading
iii. To resist uplift forces

Lateral forces that are induced in high-rise buildings always tend to snap up the building in shear and push it over in bending (Wahid, 2007). To resist the horizontal loads induced on it, shear walls should be able to provide the necessary lateral strength to prevent the structure from excessive side sway (Rasikan and Rajendran, 2013). Often times, a better way of resisting such lateral forces is by providing a suitable arrangement of shear walls linked to structural frames. However, instability in tall buildings can occur due to a number of reasons such as slenderness, excessive axial loads, creep, shrinkage, deflection, temperature changes, movement of supports etc. Most of these are often ignored in first order analysis of structures but this may lead to much more lateral deflection than initially anticipated. The increased deflection can lead to additional moment in axially loaded columns as a result of the p-delta effect (McGinley and Choo, 1990) and thus increase the probability of buckling failure.

In the resistance of lateral forces, planar solid or coupled shear wall have been widely used. Shear walls are normally located around elevators and stairwell areas in high-rise office buildings. But often times, rigid frames may be combined with reinforced concrete shear walls to create shear wall-frame interaction systems (Ali et al, 2007).

Anshuman et al (2011) discussed the solution of shear wall problems in multi-storey frames by considering a 15 storey building. By providing shear wall alongside the frames, the top deflection was reduced to tolerable value. Also it was observed that the internal stresses in the frame were reduced by the provision of shear walls and the non-linear analysis was found to be within the elastic limit. Rahangdale and Satone, (2013) also discovered that bending moment and shearing forces on columns of tall buildings can be reduced by the provision of shear walls. Based on Khan's classification (1969) as reported by Ali et al (2007), buildings up to 20 storeys can be efficiently analysed as rigid frames. This notion was also supported by McGinley and Choo (1990). But Rasikan et al (2013) compared the performance of 20 storey framed building with and without shear walls and found out that top storey displacement for building with shear wall was 14.6% less than that without shear wall while it was 20.18% for 15 storey building.

Since most design of concrete structures are carried out under elastic state where internal forces are proportional to the deformations they produce, and also where the rules of superposition apply, the study of optimum placement of planar solid shear walls outside the core areas of buildings for minimal top displacement in shear wall – rigid frame arrangement is the main aim of this paper.

II. Materials And Modelling Method

Concise Wind Load Analysis

According to BS 6399:2-1997 (clause 2.1.2.1),

$q_s = 0.613V_e^2$ -- (1)

where q_s =Dynamic pressure (N/m^2), and V_e = Effective wind speed (m/s)

Taking effective wind speed $(V_e) = 19m/s$

Hence $q_s = 0.613x19^2 = 221.293 N/m^2 = 0.221KN/m^2$

Net surface pressure (P) $= P_e - P_i$ (Reynolds et al 2008)------------------------------ (2)

Where $P_e = external\ pressure = q_s C_{pe}$, and

$P_i = \text{int}ernal\ pressure = q_s C_{pi}$

$C_{pe}\ and\ C_{pi}$ stand for the external and internal pressure coefficients respectively.

From Table 5 of BS 6399-2:1997, D/H = 12/45 = 0.267. Since D/H < 1, $C_{pe} = +0.85$

From Table 16 of BS 6399-2:1997, $C_{pi} = -0.3$

Hence net surface pressure (P) $= 0.221(0.85 + 0.3)KN/m^2 = 0.25KN/m^2$

Length of the building normal to wind direction = 15m (see figure 2)

Hence the equivalent uniformly distributed load on surface (w) = 0.25 x 15 = 3.75 KN/m^2

With this, the equivalent concentrated load (F_i) at each floor level

$$F_i = 0.5w(h_{up} + h_{below}) \text{ -- (3)}$$

$h_{up} =$ Height of storey above floor level

$h_{below} =$ Height of storey below floor level

These concentrated loads are now applied as nodal loads at each floor level in each case of the building model. The preliminary data of the building is shown in the table below.

Table 1: Preliminary data of the office block

Height of each storey	3m
Number of storey	Fifteen (G+14)
Plan area of building	(12 x 15) m
Modulus of elasticity of concrete (using STAAD's default)	21.718 KN/mm^2
Shear wall thickness	200mm
All beam sizes	(400 x 600) mm
Column sizes	(400 x 400) mm
Wind net surface pressure	0.25 KN/m^2
Equivalent uniformly distributed load	3.75 KN/m^2

In the modelling of the structure, all nodes were considered to be rigid and all joints are capable of resisting moment in order to represent the monolithic construction that is predominant in the construction of reinforced concrete buildings. In terms of the support conditions, the frames and shear walls were all assigned fixed support which exists by default in Staad Pro so as to emulate the rigid connection between columns and footings, and also to emulate the vertical cantilever behaviour which is associated with planar shear walls. The beams were assigned a cross-section of (600 x 400) mm of concrete material while the columns were assigned cross sections of (400 x 400) mm. The shear walls were modelled using the surface mesh command in Staad for finite element analysis of the shear wall. Staad allows two main methods of meshing which are the polygonal and quadrilateral meshing methods. The polygonal method of meshing has been utilized with a division of 10 nodal points and a bias value of 1 in all axes of consideration for each storey (see figure 1).

In this work, the thickness of the shear walls are 200mm all through and the length of each shear wall in the direction of the wind (z-direction) is 2.5m which makes the stiffness of all the rectangular shear walls the same throughout the structure. Since Staad does not have an inbuilt BS6399:2-1997 wind loading, the calculation for the wind load has been carried out manually as shown above, and the values of the concentrated loads have been fed into the software program using a single load case. Since the building has a constant storey height of 3m all through the different floor levels;

At the first and last storey levels, $F_1 = F_{15} = 0.5 \times 3.75 \times (3 + 0) = 5.625$ KN
At the other intermediate floor levels, $F_i = 0.5 \times 3.75 \times (3+3) = 11.25$ KN

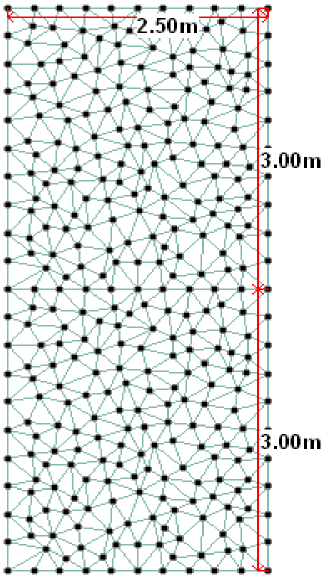


Figure 1: Meshing pattern for shear wall adopted in Staad

Using a single load case, these loads have been applied to the different arrangement models neglecting the self weight of the structure. This same modelling and loading procedure were used to analyse for the deflections in the structure when there are no shear walls and for when shear walls of different arrangements are present. The 'statics check' command in Staad was used for the analysis.

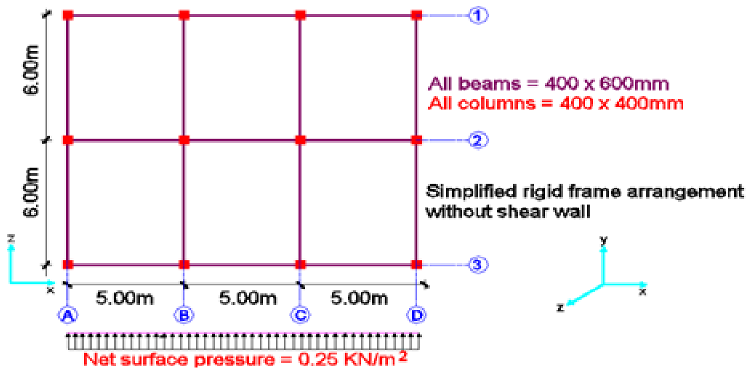
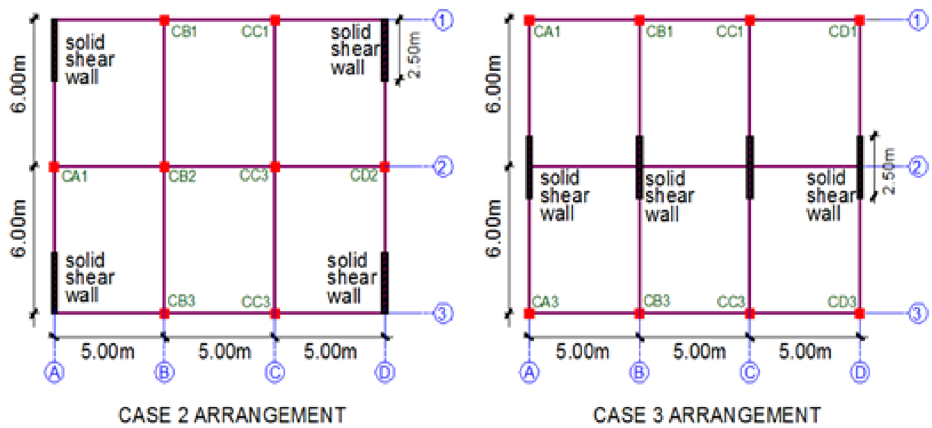


Figure 2: Arrangement Of Structure Without Shear Wall (CASE 1)



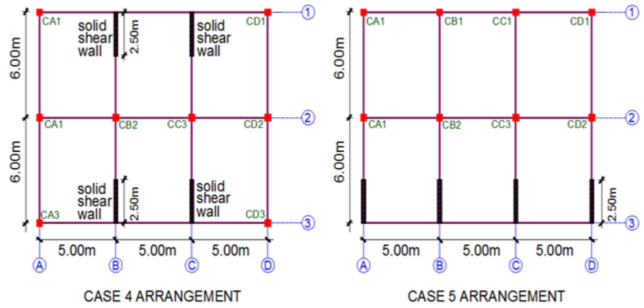


Figure 3: Different cases of shear wall arrangement

III. Results And Discussions

The load cases considered in Staad Pro V8i Software and the resulting maximum lateral deflections occurring due to the loads are shown in Table 2 below;

Table 2: Load Cases and Calculated Deflections

Arrangement Case	Load Case Considered	Calculated Deflection (mm)
Case 1	1.0Wk (wind load only)	48.200
Case 2	1.0Wk (wind load only)	38.376
Case 3	1.0Wk (wind load only)	43.992
Case 4	1.0Wk (wind load only)	38.136
Case 5	1.0Wk (wind load only)	38.604

The full results for deformation of the structure under the various cases are shown in Table 3 and a detailed analyses show that the frame without shear walls (case 1) has a deformed shape that can be represented using a 2nd degree (quadratic) polynomial, while the other cases containing shear walls can be represented using 3rd degree (cubic) polynomials. Table 4 shows polynomial regression equations relating the storey height (independent variable) and the lateral displacements (dependent variable). The different points of inflexion (POI) for the different cases have also been shown in the table.

Table 3a: Deflection results for CASE 1

STOREY LEVEL	DEFLECTION X (mm)	Y (mm)	Z (mm)
15	0.000	1.368	48.200
14	0.000	1.367	47.303
13	0.000	1.364	46.058
12	0.000	1.356	44.443
11	0.000	1.341	42.460
10	0.000	1.316	40.114
9	0.000	1.278	37.410
8	0.000	1.227	34.354
7	0.000	1.158	30.955
6	0.000	1.070	27.220
5	0.000	0.960	23.162
4	0.000	0.825	18.791
3	0.000	0.664	14.120
2	0.000	0.474	9.171
1	0.000	0.254	4.033
0(GL)	0.000	0.000	0.000

Table 3b: Deflection results for CASE 2

STOREY LEVEL	DEFLECTION X (mm)	Y (mm)	Z (mm)
15	0.136	1.264	38.376
14	0.136	1.272	36.532
13	0.137	1.319	34.614
12	0.136	1.387	32.538
11	0.134	1.468	30.255
10	0.131	1.553	27.735
9	0.127	1.633	24.961
8	0.121	1.701	21.961
7	0.113	1.745	18.746
6	0.103	1.754	15.379
5	0.090	1.713	11.947
4	0.075	1.601	8.571
3	0.075	1.403	5.447
2	0.038	1.097	2.769
1	0.017	0.647	0.831
0(GL)	0.000	0.000	0.000

Table 3d: Deflection results for case 4

STOREY LEVEL	DEFLECTION X (mm)	Y (mm)	Z (mm)
15	0.000	1.278	43.992
14	0.000	1.274	42.028
13	0.000	1.263	39.867
12	0.000	1.244	37.540
11	0.000	1.216	34.928
10	0.000	1.178	32.032
9	0.000	1.128	28.848
8	0.000	1.065	25.394
7	0.000	0.987	21.709
6	0.000	0.894	17.857
5	0.000	0.784	13.931
4	0.000	0.656	10.062
3	0.000	0.512	6.436
2	0.000	0.353	3.305
1	0.000	0.180	1.015
0(GL)	0.000	0.000	0.000

Table 3d: Deflection results for case 4

STOREY LEVEL	DEFLECTION X (mm)	Y (mm)	Z (mm)
15	0.003	1.377	38.136
14	0.001	1.376	36.986
13	0.002	1.371	35.418
12	0.003	1.360	33.418
11	0.003	1.340	31.292
10	0.003	1.309	28.819
9	0.003	1.265	26.107
8	0.004	1.204	23.186
7	0.004	1.126	20.095
6	0.004	1.029	16.107
5	0.005	0.910	13.650
4	0.005	0.769	10.456
3	0.005	0.607	7.400
2	0.004	0.424	4.551
1	0.001	0.220	1.934
0(GL)	0.000	0.000	0.000

Table 3e: Deflection results for Case 5

STOREY LEVEL	DEFLECTION X (mm)	Y (mm)	Z (mm)
15	0.000	1.201	38.604
14	0.000	1.215	36.941
13	0.000	1.276	35.170
12	0.000	1.360	33.203
11	0.000	1.454	30.996
10	0.000	1.550	28.524
9	0.000	1.640	25.783
8	0.000	1.716	22.785
7	0.000	1.769	19.562
6	0.000	1.789	16.165
5	0.000	1.795	12.675
4	0.000	1.667	9.202
3	0.000	1.486	5.923
2	0.000	1.183	3.058
1	0.000	0.711	0.937
0(GL)	0.000	0.000	0.000

Table 4: Regression equations for storey displacements of various cases

Arrangement case	Displacement (mm)	R^2	POI (metres)
Case 1 (No Shear Wall)	$Y = -0.0174X^2 + 1.887X - 1.0555$	0.999	-
Case 2	$Y = -0.000454X^3 + 0.0279X^2 + 0.5179X - 0.684$	0.999	20.50
Case 3	$Y = -0.00052X^3 + 0.0317X^2 + 0.6121X - 0.764$	0.999	20.23
Case 4	$Y = -0.000379X^3 + 0.020678X^2 + 0.685X - 0.1518$	0.999	18.18
Case 5	$Y = -0.000455X^3 + 0.02658X^2 + 0.588X - 0.7283$	0.999	19.45

For the purpose of clearer understanding and discussion of results, the internal stresses that are induced in the various shear wall arrangements under the wind loading, are represented in Table 5 below;

Table 5: Maximum internal stresses induced in the various arrangement cases

Internal Stresses	Arrangement Case 1	Arrangement Case 2	Arrangement Case 3	Arrangement Case 4	Arrangement Case 5
Maximum Axial Force (KN)	292.767	242.626	209.012	255.245	224.65
Maximum beam moment (KN.m)	121.590	79.397	93.244	80.512	85.410
Maximum beam shear force (KN)	36.360	24.202	19.765	24.450	24.736
Maximum column moment (KN.m)	103.935	78.397	47.506	79.621	71.404
Maximum column shear force (KN.m)	68.489	52.285	31.295	52.937	47.381
Maximum Torsion (KN.m)	0.000	4.405	0.000	4.445	0.000
Maximum lateral displacement (mm)	48.200	38.376	43.992	38.136	38.604

Since the building is supported by units placed at symmetrical distances from the centre of gravity of the structure, and with equal stiffness, the centre of rotation (CR) of the structure is located at the centre of gravity of the structure. A correlation analysis carried out on the above result showed that there is a relatively strong positive relationship between the beam moment and the maximum displacement of the structure with a correlation coefficient of 0.955. This discovery can have a direct influence on the observation made in the results. For instance, a little consideration on arrangement case 3 shows that more economy can be generated in the design against wind loading in terms of lesser column axial load, column moments, beam shear force and column shear force, while the design engineer has larger beam moment and lateral deflection to deal with. The correlation result is shown in Table 5 below;

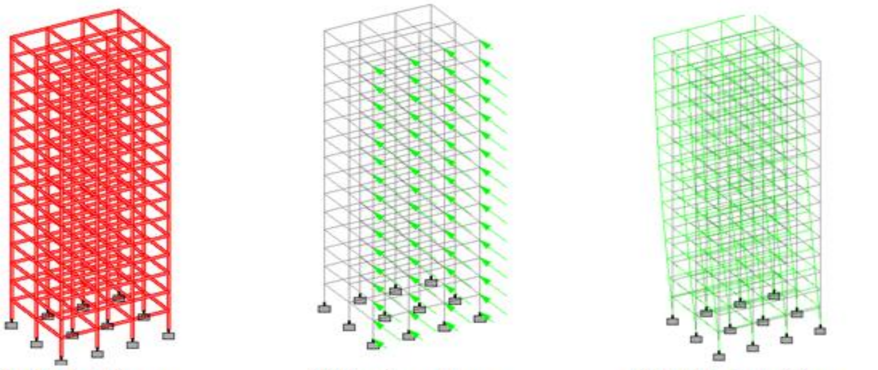
Table 5: Correlation result for the internal stresses and displacement

	MAF	MBM	MBSF	MCM	MCSF	MT	Y
MAF	1						
MBM	0.634828	1					
MBSF	0.924508	0.806409	1				
MCM	0.96042	0.533118	0.92543	1			
MCSF	0.957024	0.515446	0.917477	0.999776	1		
MT	0.116996	-0.63333	-0.23233	0.128307	0.146168	1	
Y	0.456187	0.955445	0.614859	0.302904	0.283732	-0.65218	1

MAF – Maximum Axial Force, MBM – Maximum Beam Moment, MBSF – Maximum Beam Shear Force, MCM – Maximum Column Moment, MCSF – Maximum column shear force, MT – Torsional Moment, Y – Lateral displacement

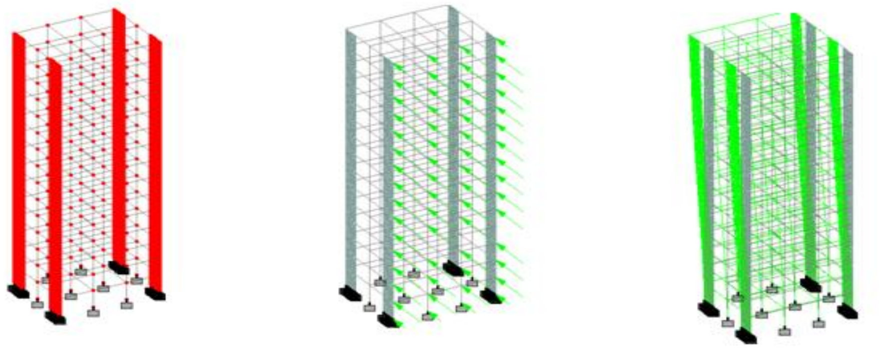
It is a well known fact that buildings which have stabilizing components of different stiffnesses can be quite difficult to calculate by hand computations and force distributions in the entire structure cannot be well established without the use of computer packages that can handle finite element analysis. As the shear walls are not coupled in any form, and positioned at distances from each other, the interaction between the four walls is greatly influenced by the stiffnesses of the floor beams and columns. An interesting observation is made in the arrangement cases 2 and 4. In the plan of the structure, the shear walls are positioned in a square and rectangular fashion respectively in the plan of the building, and it is in these arrangements that we observed torsional moments of 4.405 KNm and 4.445 KNm respectively, in the floor beams. It is also discovered that these two arrangements had the least lateral deformation of 38.376 mm and 38.136 mm respectively. Correlation analysis also showed that there is a weak negative relationship between lateral displacement and torsion in such cases. The presence of torsion in the analysis results show that it is in these cases that the shear walls interacted. All other parallel arrangements had all the shear walls resisting wind loading as individual units without any interaction with each other.

On looking at case 3 arrangement, relatively high value of displacement (43.992 mm) was observed despite the presence of shear walls. The parallel arrangement of the shear walls in the axis coinciding with centre of gravity and centre of rotation of the structure is the explanation for the lesser axial forces in the columns of the structure for such load case. The shear walls will attract higher forces due to their higher stiffness but will likely perform in an awful manner in resisting deformation. Shear walls sway in a predominantly flexural (bending) mode, while frames sway in a shear mode. If we treat the structure as a unit, and knowing full well the variation of bending and shear stresses in a structure, placing the shear walls along the neutral axis will not influence the displacement of the frame adequately since its mode of deformation is flexural, and in the distribution of bending stresses, the neutral axis lies in the centroid hence making it less effective. On the other hand, a similar parallel arrangement was adopted in case 5 and the lateral displacement was lesser with a value of 38.6mm because the shear walls were placed in the tension face of the structure, and the shear walls were able to assist in resisting lateral deformation, at the expense of the columns of the frame being subjected to bigger axial loads.



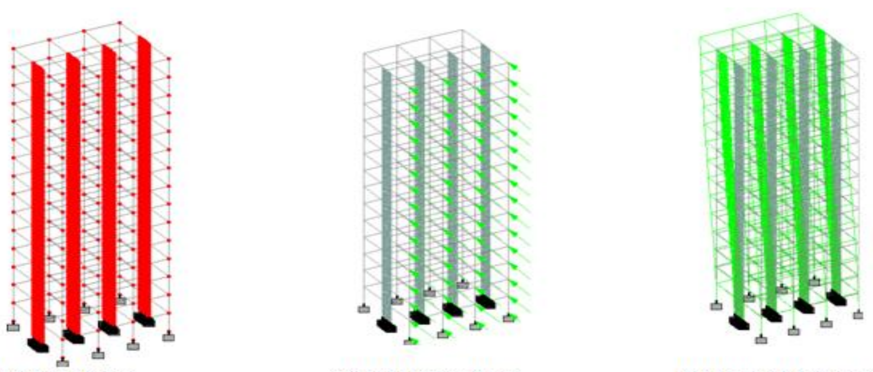
(4a) Model of frame (4b) Loading pattern (4c) 3D Deflected shape

Figure 4: Modelling, loading and deflected shape of arrangement CASE 1



(5a) Model of frame (5b) Loading pattern (5c) 3D Deflected shape

Figure 5: Modelling, loading and deflected shape of arrangement Case 2



(6a) Model of frame (6b) Loading pattern (6c) 3D Deflected shape

Figure 6: Modelling, loading and deflected shape of arrangement Case 3

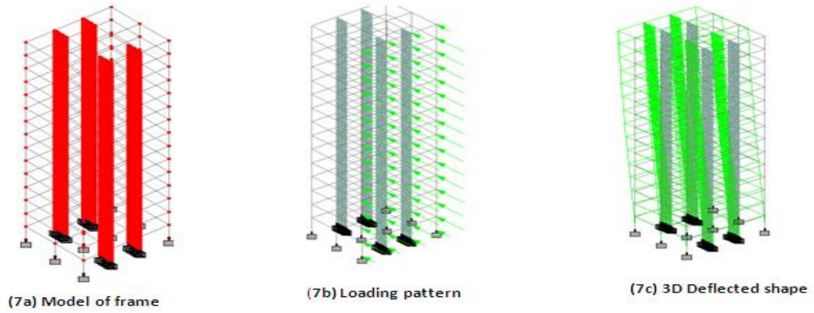


Figure 7: Modelling and deflected shape of arrangement CASE 4

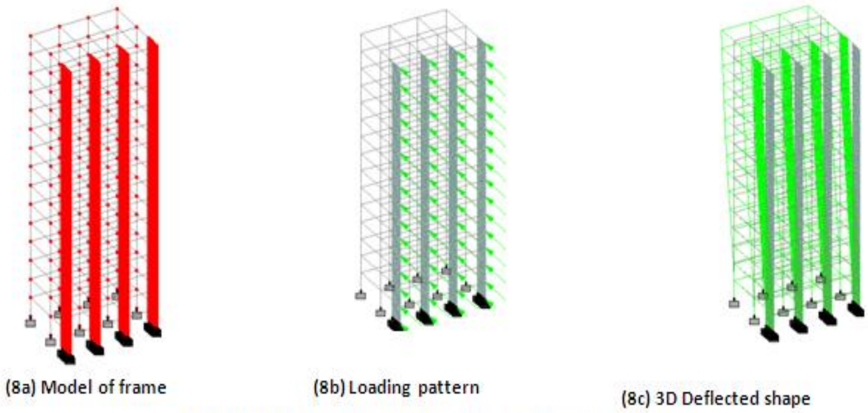


Figure 8: Modelling and deflected shape of arrangement CASE 5

In general, the variation of lateral displacement in the multi-storey frame is shown in Figure 9 below, and the percentage difference is shown in Table 6.

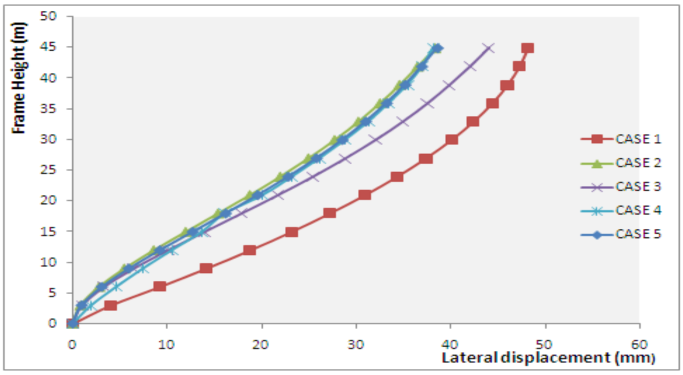


Figure 9: Variation of Lateral displacement with Height of Frame.

Table 6: Variation of maximum displacement across the different cases.

Shear Wall Arrangement Cases	**Maximum Lateral Displacment (mm)**	**Percentage Difference**
Case 1 (No Shear Wall)	48.200	-
Case 2	38.372	20.39 %
Case 3	43.992	8.73%
Case 4	38.136	20.87%
Case 5	38.605	19.91%

IV. Conclusion

From the above results, it can be seen that the deflection of the structure in the lateral (z) direction is maximum at the last storey level. The major difference in displacement when shear walls are not provided (Case 1) and when shear walls are provided (Case 3) is 8.73%. This relatively close margin is mainly due to the positioning of the shear walls at the centre of the structure or close to the lateral neutral axis in Case 3. Under the effect of lateral load, the structure behaves like a vertical cantilever, with the windward side in tension, and leeward side in compression. Thus, for a solid shear wall- frame combination, placing the shear walls close to the centroidal axis will have the least positive effect in terms of storey displacement as can be seen from the analysis results (see Case 1 and Case 3), since displacement is reduced only by about 8.73%. However, placing the shear walls at or close to the face of the windward or leeward side reduced the displacements by a range of 19.91% - 20.87% depending on the adopted symmetrical arrangement. A little consideration of the above results shows that Cases 2, 4, and 5 yielded approximately the same amount of displacement at the last storey level. Hence, the optimization of displacement in tall buildings can be said to be dependent on the position of the shear walls in the frame amongst other factors.

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