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CONFIGURATION OF A MULTISTOREY BUILDING SUBJECTED TO LATERAL FORCES

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Abstract

A study has been carried out to determine the optimum configuration of a multistorey building by changing shear walls location. Four different cases of shear wall position for a 25 storey building have been analyzed as a space frame system using a standard package ETAB subjected to lateral and gravity loading in accordance with UBC provisions.

It is found that columns and beams forces are found to increase on grids opposite to the changing position of shear wall away from the centroid of the building. Twisting moments in members are observed to be having increasing trend with enhancement in the eccentricity between geometrical centroid of the building and shear wall position. Stresses in shear wall elements have more pronounced effect in elements parallel to displaced direction of shear wall as compared to those in perpendicular direction.

The lateral displacements of the building is uniform for a zero eccentricity case. On the contrary, the drift is more on grids on one side than that of the others in case of eccentric shear wall position. It is concluded that the shear wall should be placed at a point by coinciding center of gravity and centroid of the building.

Keywords: Shear wall; lateral loading; eccentricity; drift; forces; stresses

1. Introduction

Reinforced concrete walls, which include lift wells or shear walls, are the usual requirements of Multi Storey Buildings. Design by coinciding centroid and mass center of the building is the ideal for a Structure. However, on many occasions the design has to be based on the off center position of the lift and stair case walls with respect to the center of mass. The design in these cases results into an excessive stresses in most of the structural members, unwanted torsional moments and sways.

A 2-D plane frame, which is probably the simplest assembly to be modeled, has both its column and beam members represented by line elements, [1-4]. Shear deformations of the members are normally neglected except for beams with a span to depth ratio of less than

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almost 5. The results of the analysis include the vertical and horizontal displacements, and the out of plane rotations of the nodes, together with the members axial forces, shear forces and bending moments. Out of plane displacements are assumed to be zero. In two-dimensional analysis, only typical plane frames are selected and it is assumed that the analysis of one of the plane frames, generally, would also represent other frames of the structure in one direction. The same rule is followed in other direction, [1-2&5].

With the availability of high-speed digital computers and advancement of numerical techniques, a rigorous three-dimensional analysis of a multi storey building may be performed.

Three-dimensional analysis is relatively more realistic and however, it is cost prohibitive. It gives significantly more exact results than those by two-dimensional analysis. Nevertheless, three-dimensional analysis is the only solution in case of an unsymmetrical loading or geometry of the structure.

Shell-type behavior means that both in-plane membrane stiffness and out-of-plane plate bending stiffnesses are provided for a thin plate element [6]. Membrane elements have properties defined in a plane. There are two translational freedoms at each node. Membrane-type behavior means that only in-plane membrane stiffness is provided for the section. Plate-type behavior means that only out-of-plane bending stiffness is required.

The minimization of the total potential energy in case of membrane action leads to the force displacement relationship as given, [6]:

$$f^{e\rho} = K^{e\rho} a^{\rho}$$

for node 'i'
$$a_i^{\rho} = \begin{Bmatrix} u_i \\ v_i \end{Bmatrix} \quad f_i^{\rho} = \begin{Bmatrix} U_i \\ V_i \end{Bmatrix} \tag{1}$$

Similarly, when bending is considered, the state of strains is given uniquely by the nodal displacement in the z direction (w) and the two rotations θ_x and θ_y. This results in the force-displacement relationship as provided in [19]:

$$f^{eb} = K^{eb} a^{b}$$

for node 'i'
$$a_i^{b} = \begin{Bmatrix} w_i \\ \theta_{xi} \\ \theta_{yi} \end{Bmatrix} \quad f_i^{\rho} = \begin{Bmatrix} W_i \\ M_{xi} \\ M_{yi} \end{Bmatrix} \tag{2}$$

$$a_i = \begin{Bmatrix} u_i \\ v_i \\ w_i \\ \theta_{xi} \\ \theta_{yi} \\ \theta_{zi} \end{Bmatrix} \tag{3}$$

and the corresponding 'forces' as

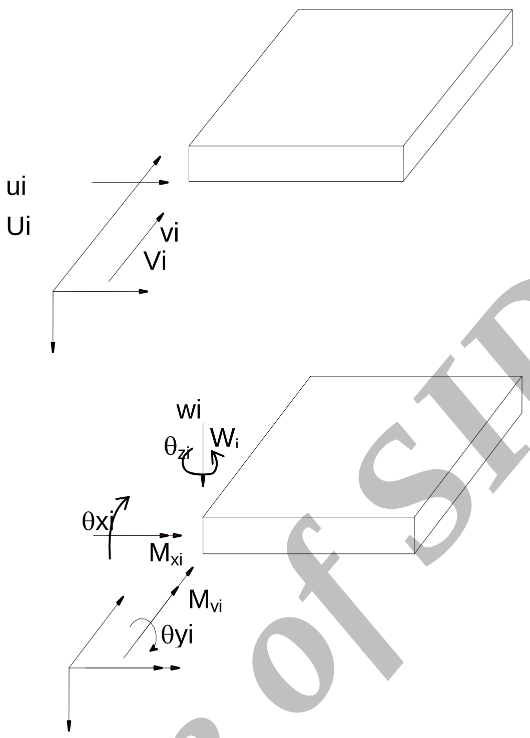

Figure 1. A flat plate element subjected to 'in plane' and bending actions [6]

Before combining these stiffnesses, it is important to note two facts. The first, that the displacements prescribed for 'in plane' do not affect the bending deformations and vice versa. The second, that rotation θ_z does not enter as a parameter into deflections or deformations in either mode [7]. It is convenient, for reasons, which will be apparent when assembly is considered, to take this rotation into account, which is associated with a fictitious couple M_z. The fact that it does not enter into the minimization procedure can be accounted for simply by inserting an appropriate number of zeros into the stiffness matrix with the exception of leading diagonal. On the leading diagonal, one unit corresponding to θ_{zi} is placed. For node 'i', as shown in Figure 1, the likely displacements are as follows:

$$f_i^e = \begin{Bmatrix} U_i \\ V_i \\ W_i \\ M_{xi} \\ M_{yi} \\ M_{zi} \end{Bmatrix} \tag{4}$$

Force displacement relationships can be written as

$$f^e = K^e a$$

The stiffness matrix is now made up from the following submatrices including the coefficients corresponding to the fictitious rotation θ_z for node 'i' as given, [6]:

$$K_i^e = \left[\begin{array}{cc|ccc|c} \multicolumn{2}{c|}{} & 0 & 0 & 0 & 0 \\ \multicolumn{2}{c|}{K_{rs}^p} & 0 & 0 & 0 & 0 \\ \hline 0 & 0 & & & & 0 \\ 0 & 0 & & K_{rs}^b & & 0 \\ 0 & 0 & & & & 0 \\ \hline 0 & 0 & 0 & 0 & 0 & 0 \end{array}\right] \qquad (5)$$

It should be noted that the displacements for a typical node "i" is as follows:

$$a_i = \begin{Bmatrix} a_i^p \\ a_i^b \\ \theta_{Zi} \end{Bmatrix} \qquad (6)$$

The above formulation is valid for any shape of an element and, in particular, for the two important cases illustrated in Figure 1. The element stiffness sub-matrices for in plane and bending actions for each of the node are 2×2 and 3×3, respectively. Further, to solve the eccentricity problem, between skeletal and continuum elements in y and z directions, can be dealt using a technique p[rovided in Ref. [6].

Out of various available structural analysis programs, ETABS V 8.4.6 program (based on finite element method) was used for the purpose of analysis [7].

A 25 storey RCC office buildings was selected with different positions of shear walls. This building was analyzed as 3-D for the specified combinations of gravity and earthquake loads, [4,8,10]. Then a comparison was made between the different cases of variable positions of shear walls.

This study was carried out under the following assumptions:

- Seismic zone '2A' was assumed for the calculation of earthquake loading using equivalent static method (pseudo static method).
- Effect of pattern loading was ignored in the analysis.

2. Structural Data

Building consists of 5 bays of 21 ft. (6.24m) in short direction and 7 bays of 25 ft. (7.62m) in long direction, so from preliminary design the sizes of various structural members were estimated as follows:

Column Sizes:
34″ × 34″ (850 × 850mm) From Base to Storey level 13
30″ × 30″ (750 ×750mm) From Storey level 13 to 16
26″ × 26″ (650 × 650mm) From Storey level 16 to 19
22″ × 22″ (550×550mm) From Storey level 19 to 22
18″ × 18″ (450 ×450mm) From Storey level 22 to 25

However, columns around the periphery were kept of the same size i.e. 24″×24″ (600 x 600mm) to avoid the local eccentricity.

Beam Size:
All beams are of uniform size of 16″ × 24″ (400 × 600mm) having 7″ (175mm) thick slab for all the spans.

Shear Wall Thickness:
24″ (600mm) Thick From Base to Storey level 2
21″ (525mm) Thick From Storey level 2 to 4
18″ (450mm) Thick From Storey level 4 to 6
15″ (375mm) Thick From Storey level 6 to 8
12″ (300mm) Thick From Storey level 8 to 10
9″ (225mm) Thick From Storey level 10 to 25

Storey height is kept as 11 ft. (3.35mm) for all the floors. Grade 60 (430 MPa) hot rolled deformed steel is recommended to be used. Concrete having 3000 psi (21 MPa) cylinder strength for walls, beams and slabs is to be employed. Whereas, columns are to be made of concrete having 4,000 psi (28 MPa) cylinder strength.

3. Loadings

3.1 Gravity loading

Gravity loading consists of dead and live loading. Dead loading can be predicted reasonably accurately from the designed member sizes and material densities. Dead load due to structural self weights and superimposed dead loads were as follows:

Slab self weight = 87.5 psf (4.20 kN/m^2)
Superimposed dead load for typical floors = 40 psf(1.92 kN/m^2)
Superimposed dead load for roof =60 psf (2.86 kN/m^2)

Live loading magnitude was estimated based on ANSI for office loading, [9]. The probability of not all parts of a floor supported by a beam, and of not all floors supported by a column, being subjected to the full live loading simultaneously, is provided by reductions in the beam loading and in the column loading, respectively. Typical live loads are as follows:

Live load at typical floors =50 psf (2.62 kN/m^2)
Live load at roof =30 psf (1.31 kN/m^2)

3.2 Lateral loading

Lateral loading consists of wind loading and earthquake loading. Wind loading is usually estimated by a manual procedure but in ETABS package program; it has been estimated automatically by the application of wind pressure to the vertical face of the building according to the UBC code. Modern static methods of determining a wind loading account for the region of the country where the building is to be located, the exposure of the particular location, the effect of gusting, and the importance of the building in a post-wind storm situation.

Earthquake loading has been calculated by the program and it has been applied to the mass center of the building.

Since the building under consideration was in Zone –2A with standard occupancy so the total base shear was computed as follows:

Case: EQX and EQY
Period Calculation: Program Calculated
Top Storey: STOREY 25
Bottom Storey: BASE
R = 9
I = 1
(Building Height): Hn = 276 ft. (84.15m)
Soil Profile Type = SD
Z = 0.15
Ca = Seismic Coefficient, as set forth in Table 16-Q (UBC-97) = 0.22
Cv = Seismic Coefficient, as set forth in Table 16-R (UBC-97) = 0.32

The total design base shear in a given direction shall be determined from the following formula:

$$V = (Cv|W)/(RT) \tag{1}$$

The total design base shear should not exceed the following:

$$V \le 2.5 Ca|W/R \tag{2}$$

The total design base shear shall not be less than the following:

$$V \le 0.11 Ca\,|W \tag{3}$$

If $T \le 0.7$ sec, then $F_t = 0$ If $T > 0.7$ sec, then $F_t = 0.07\ T\ V \le 0.25$ V Base shear is converted into lateral forces over the top of each storey by a simple technique.

3.3 Strength requirements

The required strength ‘U’ of the structural members to resist dead load (D.L), live load

(L.L), wind load (W.L), and equivalent earthquake load (E.L) should be the greatest value computed from analyses subjected to the following combination of loads according to ACI 318-99 [8]:

$$U = 1.4\,D.L + 1.7\,L.L$$
$$U = 1.05\,D.L + 1.275\,L.L + 1.275\,W.L$$
$$U = 0.9\,D.L + 1.3\,W.L$$
$$U = 1.05\,D.L + 1.275\,L.L + 1.4025\,E.L$$
$$U = 0.9\,D.L + 1.43\,E.L$$

4. Building Under Consideration

The building under consideration was a twenty-five storied office building, as shown in Figure2 with shear wall at the center of gravity. Studies have been made for the displacement of the shear walls away from mass center for given loading. For this three dimensional analysis of the given building was performed on ETABS for gravity as well as for lateral loadings.

Gravity loads are vertical downward loads i.e. both dead and live loads, whereas lateral loads are the wind load and earthquake loads computed by the program ETABS.

The given building is 105 ft.×175 ft. (32 × 53.35m) as shown in the Figure 2. The building has 1378 joints, 3050 line elements and 525 plate elements. The maximum eccentricity of shear wall is equal to 75ft (22.8m) in X- direction is indicated in Figure 3.

5. Results

Results obtained from the analyses are recorded in tabular form for the following four cases of the building separately for comparison of beams, column and shear walls critical forces and displacements:

Case 1 When shear wall is placed at center of building
Case 2 When shear wall is displaced 25 ft. (7.62m) from the centroid in X-direction
Case 3 When shear wall is shifted 50 ft. (15.24m) from the centroid in X-direction
Case 4 When shear wall is located at 75 ft. (22.86m) from the centroid in X-direction

5.1 Beam moments

Comparison of results of negative bending moments at faces of columns due to gravity and lateral loading in comparison to zero eccentricity case of shear wall location has been presented in Table 1 and leads to the following important points:

1. At extreme grid 'H' bending moment is found to increase with the increase in eccentricity in case of lower levels of the building. On the contrary, for higher levels of the building, the opposite is true.
2. The difference in moment at grid varies between 72% to 198% for 1st storey for case 2

to case 4 of the shear wall location. Whereas, this is found to decrease for these cases between 21% to 56% for storey #25.

At other extreme side i.e. grid 'A', bending moments is, generally, found to decline with the increase of eccentricity of shear wall location on the opposite side of grid A.

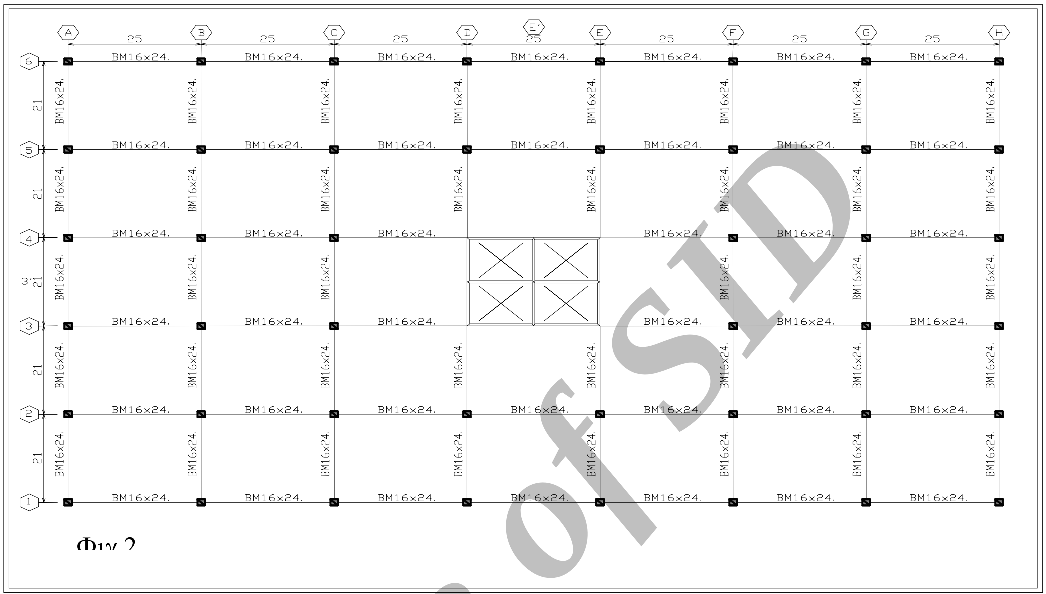

Figure 2. Plan of building (shear wall at center of mass)
Note:1 ft = 0.305 m and 1 in = 25 mm

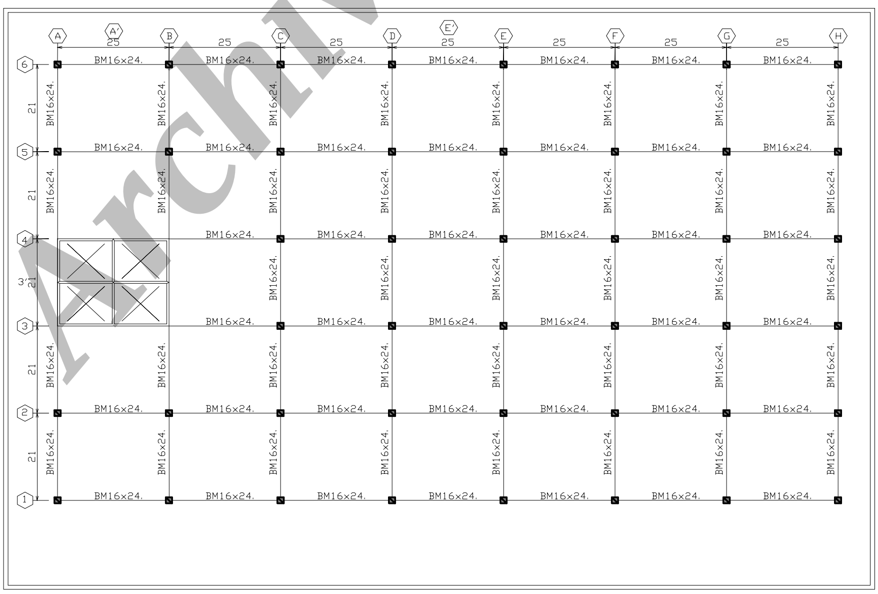

Figure 3. Plan of building (shear wall dispalced 75 ft (22.8 m) IN X-DI rection)
Note: 1 ft = 0.305m and 1 in = 25mm

5.2 Beam torsion

Comparison of results due to gravity and lateral loading in comparison to zero eccentricity case with other eccentric shear wall location show that torsion in general has been found to be increased with the increasing eccentricity of the shear wall location.

Further, the maximum twisting moment is produced for 25^{th} storey at grid '2'. This is approximately 21 k-ft. (28.5 kN-m) among all the eccentric cases. The torsion is negligibly small for concentric case of the buildi

Table 1. The comparison of beam moments for various eccentric positions of shear wall

Grid Line	**Shear Wall Placed at C.G.**		**Shear Wall 25 ft. (7.62m) Eccentric From Centroid**				**Shear Wall 50 ft. (15.24m) Eccentric From Centroid**				**Shear Wall 75 ft. (22.86m) Eccentric From Centroid**			
	Storey 1	**Storey 25**	**Storey 1**		**Storey 25**		**Storey 1**		**Storey 25**		**Storey 1**		**Storey 25**	
	Moment	**Moment**	**Moment**	**Diff.**	**Moment**	**Diff.**	**Moment**	**Diff.**	**Moment**	**Diff.**	**Moment**	**Diff.**	**Moment**	**Diff.**
	(Kip-ft.)	***(Kip-ft.)***	***(Kip-ft.)***	***(%)***	***(Kip-ft.)***	***(%)***	***(Kip-ft.)***	***(%)***	***(Kip-ft.)***	***(%)***	***(Kip-ft.)***	***(%)***	***(Kip-ft.)***	***(%)***
	-93	-51	-160	72	-41	-21	-227	144	-30	-41	-277	198	-22	-56
	-99	-104	-166	68	-97	-7	-233	135	-90	-13	-284	186	-86	-17
H	-100	-110	-168	67	-104	-6	-235	134	-97	-12	-285	184	-93	-15
	-101	-109	-168	67	-104	-5	-235	133	-98	-11	-286	183	-95	-13
	-101	-127	-163	68	-115	-10	-238	135	-102	-19	-289	186	-96	-25
	-125	-67	-205	64	-56	-16	-292	134	-44	-34	-363	190	-36	-46
	-136	-141	-215	59	-135	-5	-302	123	-127	-10	-373	175	-122	-14
G	-138	-154	-217	58	-147	-5	-305	122	-140	-9	-375	173	-136	-12
	-138	-157	-218	58	-151	-4	-305	121	-145	-8	-376	172	-142	-10
	-138	-154	-220	60	-142	-8	-310	125	-128	-17	-383	178	-119	-15
	-126	-55	-181	43	-57	3	-252	99	-47	-15	-315	149	-40	-28
	-141	-209	-191	36	-134	-36	-262	86	-127	-39	-325	131	-122	-42
F	-153	-155	-193	26	-148	-5	-265	73	-141	-9	-328	114	-137	-12
	-136	-131	-194	43	-153	17	-266	96	-146	12	-329	142	-143	-10
	-137	-149	-196	43	-145	-2	-269	97	-134	-10	-334	144	-125	-16
E	-131	36	-159	21	-53	-248	-212	61	-54	-252	-268	104	-47	-233
	-135	-142	-171	26	-144	2	-228	69	-140	-2	-286	111	-132	-7
D	-131	36	-140	7	37	3	-174	3	-50	-241	-221	68	-53	-250
	-135	-142	-144	7	-140	-1	-187	38	-140	-1	-238	76	-138	-3
C	-126	-55	-116	-8	34	-161	-140	11	38	-169	-176	39	-48	-13
	-137	-149	-120	-13	-144	-4	-144	5	-137	-8	-188	37	-138	-7
B	-125	-67	-88	-30	-63	-6	-100	-20	27	-140	-134	7	38	-156
	-138	-154	-97	-30	-155	0	-103	-25	-144	-7	-138	0	-138	-11
A	-93	-51	-53	-43	-57	12	-48	-48	-52	1	-68	-27	-7	-86
	-101	-127	-60	-41	-133	5	-54	-47	-127	0	-70	-30	-120	-6

Note: 1 Kip ft = 1.35 kN-m

5.3 Column axial forces

Comparison of axial loads in columns due to gravity and lateral loading for four cases of shear wall location, leads to the following points:

1. Generally, axial forces are relatively greater in the lower stories of the building.
2. The variation in column forces at the same level is negligible for a geometric case.
3. At Grid ‘E’, axial forces at storey 1 increase from 26% (Case-2) to 35% (Case-4) in comparison with Case-1. At storey 25, increase in axial forces is from 151% (Case-2) to 168 for Case-3 and 68% for Case-4 in comparison with concentric position of shear wall.
4. The maximum axial forces are around 2700 kips (12000 kN) between Grid ‘B’ to ‘G’ for storey 1. The same is only approximately 100 kips (445 kN) for the top storey.

5.4 Column moments

Comparison of column moments due to gravity and lateral loading with zero eccentricity case are recorded in Table No. 2 and leads to the following points:

1. At Grid ‘H’, moments at storey 1 increase from 183% (Case-2) to 487% (Case-4). At storey 25, increase in moments is from 107% (Case-2) to 281% (Case-4). This increasing trend is in comparison with that of Case-1.
2. At Grid ‘G’, moments at storey 1 is found to enhance from 143% (Case-2) to 413% (Case-4). At storey 25, increase in moments is from 76% (Case-2) to 205% (Case-4).
3. The column moment is generally found to reduce on grids opposite side of shifting of shear wall. And in this way, there is almost negligible increase in column moments at Grid ‘A’ both for lower and upper levels of the building. On the contrary, the decline in column moment is significant on this grid.
4. The value of column torsion is 17.5 Kip-ft (23.62 kN-m), for Case-2, 30.5 Kip-ft (41.18 kN-m) for Case-3 and 37.6 Kip-ft (50.76 kN-m) for Case-4. It follows that twisting moment in columns shows increasing trend with the changing position of shear wall.

Table 3. The comparison of displacement/drift for various eccentric positions of shear wall for earthquake forces in Y-direction

Building Case	**Building Location**	**Displacement in X-Direction (in)**	**Displacement in Y-Direction (in)**	**Drift-X (ft)**	**Drift-Y (ft)**
1	Right	0.0	5.0	0	0.001562
(Disp=0ft)	Left	0.0	5.0		
2	Right	0.5	5.9	0.000071	0.001393
(Disp=25ft)	Left	0.5	4.2	0.000071	0.001629
3	Right	0.9	7.0	0.000102	0.001238
(Disp=50ft)	Left	0.9	3.9	0.000102	0.001578
4	Right	1.3	8.7	0.000145	0.001206
(Disp=75ft)	Left	1.3	4.4	0.000145	0.001689

Note: 1 ft = 300 mm
1 in = 25 mm

Table 2. The comparison of column moments for various eccentric positions of shear wall

Grid Line	Shear Wall Placed at C.G.		Shear Wall 25 ft. (7.62 m) Eccentric From Centroid				Shear Wall 50 ft. (15.24 m) Eccentric From Centroid				Shear Wall 75 ft. (22.86 m) Eccentric From Centroid			
	Storey 1	Storey 25	Storey 1		Storey 25		Storey 1		Storey 25		Storey 1		Storey 25	
	Moment *(Kip-ft.)*	Moment *(Kip-ft.)*	Moment *(Kip-ft.)*	Diff. *(%)*	Moment *(Kip-ft.)*	Diff. *(%)*	Moment *(Kip-ft.)*	Diff. *(%)*	Moment *(Kip-ft.)*	Diff. *(%)*	Moment *(Kip-ft.)*	Diff. *(%)*	Moment *(Kip-ft.)*	Diff. *(%)*
H	190	-13	537	183	-27	107	873	360	-41	211	1114	487	-50	281
	213	76	573	169	59	-23	923	333	41	-47	1174	451	30	-61
	214	118	574	168	102	-13	924	331	87	-26	1176	449	79	-33
	215	124	575	168	109	-12	925	331	95	-24	1176	448	86	-31
	215	136	576	168	118	-13	924	330	101	-26	1176	447	91	-33
	221	123	569	157	110	-11	905	309	96	-22	1146	418	88	-29
G	191	-20	465	143	-35	76	755	295	-49	152	980	413	-60	205
	221	84	509	130	66	-22	817	269	47	-44	1056	377	35	-59
	223	137	512	129	123	-10	820	267	107	-22	1058	373	97	-29
	224	146	513	129	133	-9	820	266	117	-20	1059	373	108	-26
	224	157	513	129	142	-10	820	266	124	-21	1059	373	112	-29
	229	149	501	119	137	-8	793	247	122	-18	1017	345	112	-25
F	191	-10	383	100	-32	228	620	224	-46	374	823	330	-58	493
	223	121	425	91	69	-43	677	204	51	-58	891	300	39	-68
	219	133	425	94	125	-6	677	209	111	-17	892	307	101	-24
	227	121	428	88	137	14	678	198	124	3	893	293	114	-5
	223	137	427	91	148	8	679	204	133	-3	893	300	121	-11
	228	146	420	84	142	-3	658	188	130	-11	861	277	120	-18
E	193	19	302	57	-16	-185	485	152	-40	-305	666	246	-50	-359
	234	305	340	45	110	-64	535	128	55	-82	725	209	43	-86
	220	53	341	55	133	151	537	144	142	168	728	231	133	151
	228	140	338	48	142	1	523	129	136	-3	704	208	128	-9
D	193	19	221	15	17	-13	352	83	-21	-210	510	165	-41	-313
	234	305	265	13	302	-1	392	67	101	-67	559	138	53	-83
	220	53	251	14	51	-4	394	79	127	140	562	155	140	164
	228	140	258	13	139	-1	390	71	137	-2	548	140	132	-6
C	191	-10	140	-27	21	-317	218	14	14	-242	354	85	-22	125
	223	121	178	-20	307	154	260	18	296	145	394	77	102	-16
	223	137	165	-26	55	-60	248	11	45	-67	395	77	122	-11
	228	146	176	-23	142	-3	254	11	136	-7	391	71	135	-8
B	191	-20	57	-70	-2	-92	84	-56	22	-214	198	4	12	-161
	221	84	82	-63	129	53	120	-46	300	255	238	7	290	243
	224	157	83	-63	142	-10	105	-53	57	-64	225	0	46	-71
	229	149	94	-59	151	1	121	-47	140	-6	234	2	134	-10
A	190	-13	-21	-111	-4	-68	-47	-125	4	-132	42	-78	14	-206
	213	76	6	-103	88	15	-32	-115	118	55	67	-69	-29	-138
	215	136	-4	-102	144	6	-32	-115	121	-11	59	-73	69	-49
	221	123	11	-95	131	6	-15	-107	125	1	73	-67	117	-5

Note: 1 Kip ft = 1.35 kN-m

5.5 Storey displacements and drifts

It may be observed from Table 3 that displacements of the building floor at storey 25 for a case when shear walls are placed at center of gravity of the building is uni-directional & symmetric i.e. for seismic force in y-direction, building displaces only in y-direction and vice versa. With the off center position of shear walls, the building displaces in both x and y-directions with a maximum value of x-displacement of 1.3 in (32.5 mm) and y-displacement of 8.71 in (218 mm) for case when shear walls were displaced 75ft. (22.86 m) from centroid of the building. The same is true in case of storey drift, which shows enhancing trend with the increase in the eccentricity.

6. Conclusions

The study of the building having 25 stories and different positions of shear walls displaced on one side along the length leads to the following conclusions:

1. Beam moments at column points due to seismic loading are found to increase towards edge grids opposite to the displaced direction of shear walls at lower stories and on the contrary, the moments are found to have lesser values at the same grids of upper stories. It follows that the behavior becomes reversed for the edge grids from the position of shear walls location for lower stories and vice versa.
2. Torsion in beams increases with the enhancement in eccentricity of shear walls. Torsion in beams due to seismic loading has the maximum effect at top stories with the increase in eccentricity. Its maximum effect is closer to the edge grid of the building away from the displacing direction of shear walls and for members joining shear walls.
3. Column axial forces and moments due to seismic loading are found to increase with the enhancement in eccentricity towards the edge grid opposite to the displaced direction of shear walls. On the contrary, the behavior becomes reversed for the edge grid in the displacing direction of shear walls.
4. Torsion in columns also shows an increasing trend with the enhancement in eccentricity. It increases from base to maximum at storey level 2 to 3 and start decreasing towards upper stories.
5. Comparison of forces in shear walls shows that the eccentricity causes major effect on shear walls. It depends on its location in the building. For a given case, it causes maximum effect on pier members in the direction displaced of shear walls.
6. The displacement of building is uni-directional and uniform for all the grids in the case of zero eccentricity for seismic loading. With the increase in the eccentricity, the building shows non-uniform movement of right and left edges due to torsion.
7. Building receives more drifts with the increase in eccentricity.
8. The study indicates the significant effects on axial and shear forces along with bending and twisting moments of beams and columns at different levels of the building by shifting the shear wall location. Placing shear wall away from center of gravity resulted in increase in most of the members forces. It follows that shear walls should be placed in such a fashion that center of gravity of the building should be coinciding with the centroid of the building.

9. It is clear from the study that non-uniform placement of stiff elements cause the structure more harm than good by introducing torsion besides increase in beam and column moments due to their off-center locations.

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