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]:

\[
f^{\rho\rho} = K^{\rho\rho} a^{\rho}\]

for node ‘i’

\[
a^{\rho} = \begin{bmatrix} u_i \\ v_i \end{bmatrix}\]

\[
f_i^{\rho} = \begin{bmatrix} U_i \\ V_i \end{bmatrix}\]

(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 \( \theta_x \) and \( \theta_y \). This results in the force-displacement relationship as provided in [19]:

\[
f^{eb} = K^{eb} a^b\]

for node ‘i’

\[
a_i^b = \begin{bmatrix} w_i \\ \theta_{xi} \\ \theta_{yi} \end{bmatrix}\]

\[
f_i^b = \begin{bmatrix} W_i \\ M_{xi} \\ M_{yi} \end{bmatrix}\]

(2)

\[
a_i = \begin{bmatrix} u_i \\ v_i \\ w_i \\ \theta_{xi} \\ \theta_{yi} \end{bmatrix}\]

(3)
and the corresponding ‘forces’ as

$$f_i^e = \begin{bmatrix} U_i \\ V_i \\ W_i \\ M_{xi} \\ M_{yi} \\ M_{zi} \end{bmatrix}$$  \hspace{1cm} (4)$$

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 $\theta_z$ 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
$\theta_{zi}$ is placed. For node ‘i’, as shown in Figure 1, the likely displacements are as follows:
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 \( \theta_z \) for node ‘i’ as given, [6]:

\[
K^e_i = \begin{bmatrix}
K^p_{rs} & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0
\end{bmatrix}
\tag{5}
\]

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

\[
a_i = \begin{bmatrix}
a^p_i \\
a^b_i \\
\theta_z_i
\end{bmatrix}
\tag{6}
\]

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\times2 and 3\times3, respectively. Further, to solve the eccentricity problem, between skeletal and continuum elements in y and z directions, can be dealt using a technique provided 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 × 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/m²)
- Superimposed dead load for typical floors = 40 psf (1.92 kN/m²)
- Superimposed dead load for roof = 60 psf (2.86 kN/m²)

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/m²)
Live load at roof = 30 psf (1.31 kN/m²)

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:

\[ V = \left( C_v W \right) / (R T) \]  \hspace{1cm} (1)

The total design base shear should not exceed the following:

\[ V \leq 2.5 C_d W / R \]  \hspace{1cm} (2)

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

\[ V \geq 0.11 C_d W \]  \hspace{1cm} (3)

If \( T \leq 0.7 \) sec, then \( F_t = 0 \) If \( T > 0.7 \) sec, then \( F_t = 0.07 T V \leq 0.25 \) 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]:

\[
\begin{align*}
U &= 1.4 \, D.L + 1.7 \, L.L \\
U &= 1.05 \, D.L + 1.275 \, L.L + 1.275 \, W.L \\
U &= 0.9 \, D.L + 1.3 \, W.L \\
U &= 1.05 \, D.L + 1.275 \, L.L + 1.4025 \, E.L \\
U &= 0.9 \, D.L + 1.43 \, E.L
\end{align*}
\]

4. Building Under Consideration

The building under consideration was a twenty-five storied office building, as shown in Figure 2 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. x 175 ft. (32 x 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 75 ft (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 displaced 75 ft (22.8 m) IN X-DI rection)
Note: 1 ft = 0.305 m and 1 in = 25 mm
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 25th 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

<table>
<thead>
<tr>
<th>Grid Line</th>
<th>Shear Wall Placed at C.G.</th>
<th>Shear Wall 25 ft. (7.62m) Eccentric From Centroid</th>
<th>Shear Wall 50 ft. (15.24m) Eccentric From Centroid</th>
<th>Shear Wall 75 ft. (22.86m) Eccentric From Centroid</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Storey 1</td>
<td>Storey 25</td>
<td>Storey 1</td>
<td>Storey 25</td>
</tr>
<tr>
<td>H</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Moment</td>
<td>Moment</td>
<td>Moment</td>
<td>Diff.</td>
</tr>
<tr>
<td></td>
<td>(Kip-ft.)</td>
<td>(Kip-ft.)</td>
<td>(Kip-ft.)</td>
<td>(%)</td>
</tr>
<tr>
<td>-93</td>
<td>-51</td>
<td>-160</td>
<td>72</td>
<td>-41</td>
</tr>
<tr>
<td>-99</td>
<td>-104</td>
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<td>-168</td>
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<td>64</td>
<td>-56</td>
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<td>-138</td>
<td>-154</td>
<td>-217</td>
<td>58</td>
<td>-147</td>
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<td>-154</td>
<td>-220</td>
<td>60</td>
<td>-142</td>
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<td>-181</td>
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<td>-194</td>
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<td>-153</td>
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<tr>
<td>-137</td>
<td>-149</td>
<td>-196</td>
<td>43</td>
<td>-145</td>
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<tr>
<td>-131</td>
<td>-36</td>
<td>-159</td>
<td>21</td>
<td>-53</td>
</tr>
<tr>
<td>-135</td>
<td>-142</td>
<td>-171</td>
<td>26</td>
<td>-144</td>
</tr>
<tr>
<td>-131</td>
<td>-36</td>
<td>-140</td>
<td>7</td>
<td>37</td>
</tr>
<tr>
<td>-135</td>
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<td>-140</td>
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<tr>
<td>-126</td>
<td>-55</td>
<td>-116</td>
<td>-8</td>
<td>34</td>
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<td>-137</td>
<td>-149</td>
<td>-120</td>
<td>-13</td>
<td>-144</td>
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<td>-53</td>
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<td>-57</td>
</tr>
<tr>
<td>-101</td>
<td>-127</td>
<td>-60</td>
<td>-41</td>
<td>-133</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Building Case</th>
<th>Building Location</th>
<th>Displacement in X-Direction (in)</th>
<th>Displacement in Y-Direction (in)</th>
<th>Drift-X (ft)</th>
<th>Drift-Y (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Right</td>
<td>0.0</td>
<td>5.0</td>
<td>0</td>
<td>0.001562</td>
</tr>
<tr>
<td></td>
<td>Left</td>
<td>0.0</td>
<td>5.0</td>
<td>0</td>
<td>0.001562</td>
</tr>
<tr>
<td>2</td>
<td>Right</td>
<td>0.5</td>
<td>5.9</td>
<td>0.000071</td>
<td>0.001393</td>
</tr>
<tr>
<td></td>
<td>Left</td>
<td>0.5</td>
<td>4.2</td>
<td>0.000071</td>
<td>0.001629</td>
</tr>
<tr>
<td>3</td>
<td>Right</td>
<td>0.9</td>
<td>7.0</td>
<td>0.000102</td>
<td>0.001238</td>
</tr>
<tr>
<td></td>
<td>Left</td>
<td>0.9</td>
<td>3.9</td>
<td>0.000102</td>
<td>0.001578</td>
</tr>
<tr>
<td>4</td>
<td>Right</td>
<td>1.3</td>
<td>8.7</td>
<td>0.000145</td>
<td>0.001206</td>
</tr>
<tr>
<td></td>
<td>Left</td>
<td>1.3</td>
<td>4.4</td>
<td>0.000145</td>
<td>0.001689</td>
</tr>
</tbody>
</table>

Note: 1 ft = 300 mm  
1 in = 25 mm
Table 2. The comparison of column moments for various eccentric positions of shear wall

<table>
<thead>
<tr>
<th>Grid Line</th>
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<tr>
<td></td>
<td>Storey 1</td>
<td>Storey 25</td>
<td>Storey 1</td>
<td>Storey 25</td>
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<tr>
<td></td>
<td>Moment (Kip-ft.)</td>
<td>Moment (Kip-ft.)</td>
<td>Moment Diff. (%)</td>
<td>Moment (Kip-ft.)</td>
</tr>
<tr>
<td>H</td>
<td>190 -13</td>
<td>537 183</td>
<td>-27 107 (%)</td>
<td>873 360</td>
</tr>
<tr>
<td></td>
<td>213 76</td>
<td>573 169</td>
<td>-59 -23 (%)</td>
<td>923 333</td>
</tr>
<tr>
<td></td>
<td>214 118</td>
<td>574 168</td>
<td>102 -13 (%)</td>
<td>924 331</td>
</tr>
<tr>
<td></td>
<td>215 124</td>
<td>575 168</td>
<td>109 -12 (%)</td>
<td>925 331</td>
</tr>
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<td>215 136</td>
<td>576 168</td>
<td>118 -13 (%)</td>
<td>924 330</td>
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<td>221 123</td>
<td>569 157</td>
<td>110 -11 (%)</td>
<td>905 309</td>
</tr>
<tr>
<td></td>
<td>221 -20</td>
<td>465 143</td>
<td>-35 76 (%)</td>
<td>755 295</td>
</tr>
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<td></td>
<td>221 84</td>
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<td>817 269</td>
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<td>512 129</td>
<td>123 -10 (%)</td>
<td>820 267</td>
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<td></td>
<td>224 146</td>
<td>513 129</td>
<td>133 -9 (%)</td>
<td>820 266</td>
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<td></td>
<td>224 157</td>
<td>513 129</td>
<td>142 -10 (%)</td>
<td>826 266</td>
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<tr>
<td></td>
<td>229 149</td>
<td>501 119</td>
<td>137 -8 (%)</td>
<td>793 247</td>
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<tr>
<td></td>
<td>191 -10</td>
<td>383 100</td>
<td>-32 228 (%)</td>
<td>620 224</td>
</tr>
<tr>
<td></td>
<td>223 121</td>
<td>425 91</td>
<td>69 -43 (%)</td>
<td>677 204</td>
</tr>
<tr>
<td></td>
<td>219 133</td>
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<td>677 209</td>
</tr>
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<td>222 121</td>
<td>428 88</td>
<td>137 -14 (%)</td>
<td>679 198</td>
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<tr>
<td></td>
<td>223 137</td>
<td>427 91</td>
<td>148 -8 (%)</td>
<td>679 204</td>
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<td></td>
<td>228 146</td>
<td>420 84</td>
<td>142 -2 (%)</td>
<td>658 188</td>
</tr>
<tr>
<td>G</td>
<td>193 19</td>
<td>302 57</td>
<td>-16 -185 (%)</td>
<td>485 152</td>
</tr>
<tr>
<td></td>
<td>234 305</td>
<td>340 45</td>
<td>110 -64 (%)</td>
<td>535 128</td>
</tr>
<tr>
<td></td>
<td>220 53</td>
<td>341 55</td>
<td>133 151 (%)</td>
<td>537 144</td>
</tr>
<tr>
<td></td>
<td>228 140</td>
<td>338 48</td>
<td>142 1 (%)</td>
<td>523 129</td>
</tr>
<tr>
<td>E</td>
<td>193 19</td>
<td>221 15</td>
<td>17 -13 (%)</td>
<td>352 83</td>
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<tr>
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<tr>
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<td>251 14</td>
<td>51 -4 (%)</td>
<td>394 79</td>
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<td>228 140</td>
<td>258 13</td>
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<td>390 71</td>
</tr>
<tr>
<td>D</td>
<td>191 -10</td>
<td>140 27</td>
<td>21 -317 (%)</td>
<td>218 14</td>
</tr>
<tr>
<td></td>
<td>223 121</td>
<td>178 -20</td>
<td>307 154 (%)</td>
<td>260 18</td>
</tr>
<tr>
<td></td>
<td>223 137</td>
<td>165 -26</td>
<td>55 -60 (%)</td>
<td>248 11</td>
</tr>
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<td>228 146</td>
<td>176 -23</td>
<td>142 3 (%)</td>
<td>254 11</td>
</tr>
<tr>
<td>C</td>
<td>191 -20</td>
<td>57 -70</td>
<td>-2 -92 (%)</td>
<td>84 -56</td>
</tr>
<tr>
<td></td>
<td>221 84</td>
<td>82 -63</td>
<td>129 53 (%)</td>
<td>120 -46</td>
</tr>
<tr>
<td></td>
<td>224 157</td>
<td>83 -63</td>
<td>142 -10 (%)</td>
<td>105 -53</td>
</tr>
<tr>
<td></td>
<td>229 149</td>
<td>94 -59</td>
<td>151 1 (%)</td>
<td>121 -47</td>
</tr>
<tr>
<td>B</td>
<td>190 -13</td>
<td>-21 -111</td>
<td>-4 -68 (%)</td>
<td>-47 -125</td>
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<td></td>
<td>213 76</td>
<td>-103 88</td>
<td>15 -32 -115 118 55 (%)</td>
<td>67 -69</td>
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<td>215 136</td>
<td>-102 144</td>
<td>6 -32 -115 121 11-11 (%)</td>
<td>59 -73</td>
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<tr>
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<td>221 123</td>
<td>11 -95</td>
<td>131 6 (%)</td>
<td>-15 -107</td>
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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.

References

9. ACI 318-99, Building Code Requirements for Structural Concrete (318-99) and Commentary (318R-99), New York, 1999