Design Application Procedures of 15 Storied 3D Reinforced Concrete Shear Wall-Frame Structure

H. Nikzad, S. Yoshitomi

Abstract—This paper presents the design application and reinforcement detailing of 15 storied reinforced concrete shear wall-frame structure based on linear static analysis. Databases are generated for section sizes based on automated structural optimization method utilizing Active-set Algorithm in MATLAB platform. The design constraints of allowable section sizes, capacity criteria and seismic provisions for static loads, combination of gravity and lateral loads are checked and determined based on ASCE 7-10 documents and ACI 318-14 design provision. The result of this study illustrates the efficiency of proposed method, and is expected to provide a useful reference in designing of RC shear wall-frame structures.

Keywords—Structural optimization, linear static analysis, ETABS, MATLAB, RC shear wall-frame structures.

I. INTRODUCTION

Earthquake causes significant damages to the reinforced concrete high-rise building structures. Design engineers in consideration of lateral loadings should design a structure in such a way that the building should withstand all the gravity and lateral loads, and should fully fulfill all the requirements that a structure is designed for. As an earthquake resistant system, shear wall-frame structures are widely used rather than any other systems in seismic region in Afghanistan. They provide considerable stiffness and strength to the structure for strong earthquake ground motions. Properly-designed shear walls can provide safety and are significantly cost effective in RC structures during seismic activity. Therefore, special consideration of designing and detailing of shear wall capable of resisting earthquake motions without undesirable loss of strength and stiffness is required.

Extensive studies and research exist on using, designing, detailing, and requirements of RC shear wall structures. For structures assigned to seismic design categories D, E or F subjected to strong ground motions shall be provided by special moment frames, structural walls or a combination of shear wall-frame or dual system as seismic force resisting systems [1], [2]. A dual system in which the moment frames itself shall be capable of resisting at least 25% of the design seismic forces and the total seismic force resistance is provided by combination of moment frames and shear walls [3]. These types of structures, however, are considered to perform well during earthquakes; some mid-rise and high-rise RC shear wall buildings have suffered damages in the past earthquakes which had been designed well to modern building codes. Therefore, evaluation and recommendations for improving shear wall design requirements in seismic design of tall building have been carried out [4].

This study follows the seismic design of reinforced concrete building code requirements of the 2012 International Building Code (2012 IBC), American Concrete Institute Building Code (ACI 318-14), American Society of Civil Engineering (ASCE 7-10) standards. Typical reinforcement requirements for structural walls, beams and columns, special confinement at the wall edges and columns are discussed and presented. In addition, the structure is designed for lateral loadings based on equivalent linear static analysis. The method involves maximum values of displacements and member forces for earthquake motions. The procedures of seismic loadings are done by help of ETABS. Member forces such as beams, columns and shear walls are calculated and checked separately by each program and the results are compared. To avoid overstressed members, a stress constraint ratio is proposed and introduced for the most critical load combinations and structural members. The placement and details of reinforcements of structural members are explained and discussed as conclusion.

II. BASIC SEISMIC-FORCE-RESISTING SYSTEM DEFINITION

Building is designed in such a way that lateral load resistance is provided by shear walls and frames. The structure is 15 stories with typical height of 3 m. It has 3 spans with length of 9 m, 7.5 m, and 7 m in each horizontal direction, respectively. Section sizes of members are generated based on structural optimization method using Active-Set-Algorithm optimization method based on linear static analysis [5]. The compressive strength of concrete is 27 MPa and yield strength of reinforcement is 420 MPa for all member sizes, respectively. Lateral loads are calculated based on ASCE 7-10, where the site class of the structure is D, Response modification factor R=6, and S1 and Ss are 0.51 and 1.28, for Kabul region. The assumed dead load on each frame of the building is 7.5 kN/m, and assumed dead and live loads on each slab of the building are 5 kN. Fig. 1 illustrates target model of RC shear wall-frame building structure.

Section sizes of beam and column, and the thickness of shear walls are predetermined as five sizes for every three floors. Table I shows combination of section sizes of structural element. Further details could be found on [6].
TABLE I

<table>
<thead>
<tr>
<th>Element type</th>
<th>Story Number</th>
<th>Central</th>
<th>Edge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column</td>
<td>1-3</td>
<td>100x100</td>
<td>87x87</td>
</tr>
<tr>
<td></td>
<td>4-6</td>
<td>100x100</td>
<td>85x85</td>
</tr>
<tr>
<td></td>
<td>7-9</td>
<td>100x100</td>
<td>79x79</td>
</tr>
<tr>
<td></td>
<td>10-12</td>
<td>95x95</td>
<td>76x76</td>
</tr>
<tr>
<td></td>
<td>13-15</td>
<td>59x59</td>
<td>47x47</td>
</tr>
<tr>
<td>Beam</td>
<td>1-3</td>
<td>61x36</td>
<td>62x41</td>
</tr>
<tr>
<td></td>
<td>4-6</td>
<td>57x38</td>
<td>65x43</td>
</tr>
<tr>
<td></td>
<td>7-9</td>
<td>65x41</td>
<td>65x43</td>
</tr>
<tr>
<td></td>
<td>10-12</td>
<td>62x41</td>
<td>65x43</td>
</tr>
<tr>
<td></td>
<td>13-15</td>
<td>63x37</td>
<td>65x40</td>
</tr>
<tr>
<td>Shear wall</td>
<td>1-3</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4-6</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7-9</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td></td>
<td>10-12</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td></td>
<td>13-15</td>
<td>36</td>
<td></td>
</tr>
</tbody>
</table>

III. EQUIVALENT LATERAL FORCE PROCEDURES

Generally, the equivalent lateral force is permitted for regular building structures up to about 20 stories [7]. Based on ASCE7-10, the structures suitable for ELF procedures include:

1. Regular or irregular buildings in seismic design class B and C.
2. All the frame buildings or regular buildings whose natural period is smaller than 3.5s, or only horizontal/vertical irregular buildings in seismic design class D, E and F.

The equivalent lateral force (ELF) procedure, as preliminary design of all structures and final design of most of the structures, provides a simple way to incorporate the effects of inelastic dynamic response into a linear static analysis [8]. In this paper, a 15 storied shear wall-frame building structure is selected as lateral-force-resisting system, and the preliminary design of the structure is done using ETABS software. For the simplicity, a combination of section sizes of beam, column and shear wall is selected for final design application; however, all section sizes are designed and checked for the most critical load combinations. Following steps can be considered for ELF procedures:

A. Determination of Design Base Shear

Design base shear is the total lateral or shear at the base of the building equal to the sum of the seismic design force at each level of the building. The seismic base shear in a given direction shall be calculated as:

\[ V = C_s W \]  
\[ \text{(ASCE 12.8-1)} \]

where \( C_s \) is the seismic response coefficient for the building, and \( W \) is the effective seismic weight of the building consisting of the weight of all materials of construction incorporated into the building (based on specified mass).

The seismic response coefficient for the building is determined as:

\[ C_s = \frac{S_{DS}}{R/I} \]  
\[ \text{(ASCE 12.8-2)} \]

where \( S_{DS} \) = the design spectral response acceleration parameter in the short period range, \( R \) = the response modification factor that accounts for the reduction in seismic loads caused by inelastic action and energy dissipation, and \( I \) = the earthquake importance factor for the building and its occupancy.

The value of \( C_s \) shall not exceed the following:

\[ C_s = \frac{S_{DS}}{T(R/I)} \quad \text{for} \quad T \leq T_L \]  
\[ \text{(ASCE 12.8-3)} \]

\[ C_s = \frac{S_{DS}T_L}{T^*(R/I)} \quad \text{for} \quad T > T_L \]  
\[ \text{(ASCE 12.8-4)} \]

In addition, the value of \( C_s \) shall not be less than

\[ C_s = 0.044S_{DS}/I \geq 0.01 \text{(ASCE 12.8-5)} \]

For structures located where \( S_i \) is equal to or greater than 0.6g , \( C_s \) shall not be less than

\[ C_{s_{min}} = \frac{0.5S_i}{(R/I)} \]  
\[ \text{(ASCE 12.8-6)} \]

\( S_{DS} \) = the design spectral response acceleration parameter, \( T \) = the fundamental period of the structure, \( T \) =12 seconds, is long-period transition period. \( S_i = 0.51 \text{ g} \) for Kabul city, is the mapped maximum considered earthquake spectral response acceleration parameter.

The design earthquake spectral response acceleration parameter at short period, \( S_{DS} \) , and at 1 second, \( S_{D1} \) , shall be determined with:

\[ S_{DS} = \frac{2}{3} S_{ab} \]  
\[ \text{(ASCE 11.4-3)} \]
The approximate fundamental period value of $T_a$, in second, shall be obtained from

$$T_a = \frac{0.0019}{\sqrt{C_v}} \cdot h_a \quad \text{(ASCE 12.8-9)}$$

$C_v$ is calculated as follows

$$C_v = \frac{100}{\sum \left( \frac{h_i}{h_a} \right)^2} \cdot \left[ \frac{A_i}{1 + 0.83 \left( \frac{h_i}{D_i} \right)^2} \right] \quad \text{(ASCE 12.8-10)}$$

where $A_i$ = area of base of structure, ft$^2$ or m$^2$, $A_i$ = web area of shear wall $i$, m$^2$ or ft$^2$, $D_i$ = length of shear wall, m or ft, $h_i$ = height of shear wall $i$, m or ft and $x = \text{number of shear walls in the building effective in resisting lateral forces in the direction under consideration}$, $h_a$ = structural height, and the coefficients $C_i$ and $x$ can be determined from Table II.

### Table II

<table>
<thead>
<tr>
<th>Structure Type</th>
<th>$C_i$</th>
<th>$x$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel moment-resisting frames</td>
<td>0.028</td>
<td>0.8</td>
</tr>
<tr>
<td>Concrete moment-resisting frames</td>
<td>0.016</td>
<td>0.9</td>
</tr>
<tr>
<td>Steel eccentrically braced frames</td>
<td>0.03</td>
<td>0.75</td>
</tr>
<tr>
<td>Steel buckling-restrained braced frames</td>
<td>0.03</td>
<td>0.75</td>
</tr>
<tr>
<td>All other structural systems</td>
<td>0.02</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Table III lists the applicable seismic design factors based on ASCE 14-10.
consideration. Table IV summarizes lateral loads calculated based linear static analysis of ETABS.

### TABLE IV

<table>
<thead>
<tr>
<th>Story number</th>
<th>h(m)</th>
<th>W, KN</th>
<th>$w_i h_i^k$</th>
<th>$C_{xx}$</th>
<th>Lateral force</th>
<th>Story shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>45</td>
<td>9339.9</td>
<td>420295.7</td>
<td>0.117</td>
<td>1470.4</td>
<td>1470.4</td>
</tr>
<tr>
<td>14</td>
<td>42</td>
<td>9715.2</td>
<td>408039.9</td>
<td>0.114</td>
<td>1394.2</td>
<td>2864.6</td>
</tr>
<tr>
<td>13</td>
<td>39</td>
<td>9715.2</td>
<td>378894.2</td>
<td>0.106</td>
<td>1262.1</td>
<td>4126.7</td>
</tr>
<tr>
<td>12</td>
<td>36</td>
<td>9970.5</td>
<td>358937.7</td>
<td>0.100</td>
<td>1163.2</td>
<td>5289.9</td>
</tr>
<tr>
<td>11</td>
<td>33</td>
<td>10143.5</td>
<td>334723.9</td>
<td>0.094</td>
<td>1052.8</td>
<td>6342.8</td>
</tr>
<tr>
<td>10</td>
<td>30</td>
<td>10143.5</td>
<td>304294.4</td>
<td>0.085</td>
<td>926.4</td>
<td>7269.9</td>
</tr>
<tr>
<td>9</td>
<td>27</td>
<td>10186.5</td>
<td>275035.7</td>
<td>0.077</td>
<td>807.6</td>
<td>8076.8</td>
</tr>
<tr>
<td>8</td>
<td>24</td>
<td>10206.8</td>
<td>244963.5</td>
<td>0.068</td>
<td>690.8</td>
<td>8767.6</td>
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<tr>
<td>7</td>
<td>21</td>
<td>10206.8</td>
<td>214343.1</td>
<td>0.060</td>
<td>577.4</td>
<td>9345.0</td>
</tr>
<tr>
<td>6</td>
<td>18</td>
<td>10127.5</td>
<td>182295.9</td>
<td>0.051</td>
<td>465.7</td>
<td>9810.8</td>
</tr>
<tr>
<td>5</td>
<td>15</td>
<td>10156.0</td>
<td>152340</td>
<td>0.043</td>
<td>365.6</td>
<td>10176.5</td>
</tr>
<tr>
<td>4</td>
<td>12</td>
<td>10156.0</td>
<td>121872</td>
<td>0.034</td>
<td>270.9</td>
<td>10447.4</td>
</tr>
<tr>
<td>3</td>
<td>9</td>
<td>10121.9</td>
<td>91096.74</td>
<td>0.025</td>
<td>183.5</td>
<td>10630.9</td>
</tr>
<tr>
<td>2</td>
<td>6</td>
<td>10129.1</td>
<td>60774.72</td>
<td>0.017</td>
<td>106.5</td>
<td>10737.5</td>
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<tr>
<td>1</td>
<td>3</td>
<td>10129.1</td>
<td>30387.36</td>
<td>0.008</td>
<td>42</td>
<td>10779.5</td>
</tr>
<tr>
<td>Total</td>
<td>45</td>
<td>150447</td>
<td>3578294.8</td>
<td>1.000</td>
<td>10779.5</td>
<td>10779.5</td>
</tr>
</tbody>
</table>

D. Overturning Moment

According to ASCE section 12.8.5, the structure shall be designed to resist overturning moments produced by the lateral seismic forces, $F_i$, and should be calculated using:

$$M_x = \tau \sum_{i=1}^{n} F_i (h_i - h_x)$$

(17)

where $M_x =$ overturning moment at level x; $F_i =$ portion of seismic base shear, $V$, induced at level i, $h_i, h_x =$ height from base to level i and x; $\tau =$ overturning moment reduction factor which, $\tau = 1.0$ for top 10 stories, $\tau = 0.8$ for twentieth story from the top and below, and $\tau =$ linear interpolation between 1.0 and 0.8 for stories between twentieth and tenth stories below top.

Reduction factor, $\tau$, is permitted to be taken as 1.0 for the full height of the structure. The overturning moment shall satisfy:

$$SF = \frac{M_x}{M_s} > 1.75$$

(18)

where

$$M_s = W \times x_c$$

(19)

$M_s =$ resisting moment, KN·m, $W =$ weight of the building, KN, and $x_c =$ center of mass of the building, m. Overturning moment calculated by ETABS is:

$$M_s = 348407 \text{KN} - m$$

$$M_s = 150447 \times 11.83 = 1779788 \text{KN} - m$$

E. Determination of Story Drift

The story drift, $\Delta$, can be defined as the relative displacement between adjacent stories due to the design lateral forces, and can be computed as the difference between the deflection of the center of mass at the top and bottom of the story being considered. For structures assigned to seismic design category C, D, E or F having horizontal torsional irregularity or extreme torsional irregularity, the design story drift shall not exceed the allowable story drift given in Table V. The deflection used to determine the design story drift shall be computed as:

$$\delta_x = \frac{C_d \cdot \delta_{\Delta \text{m}}}{I} (ASCE 12.8-15)$$

(20)

where $C_d =$ deflection amplification factor, from table 12.2-1 of ASCE, $\delta_{\Delta \text{m}} =$ the deflection determined by elastic analysis.

### TABLE V

<table>
<thead>
<tr>
<th>Structure Occupancy category</th>
<th>I or II</th>
<th>III</th>
<th>IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structure, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drift.</td>
<td>0.025h</td>
<td>0.020h</td>
<td>0.010h</td>
</tr>
<tr>
<td>Masonry cantilever shear structures</td>
<td>0.010h</td>
<td>0.010h</td>
<td>0.010h</td>
</tr>
<tr>
<td>Other masonry shear wall structures</td>
<td>0.007h</td>
<td>0.007h</td>
<td>0.007h</td>
</tr>
<tr>
<td>All other structures</td>
<td>0.020h</td>
<td>0.015h</td>
<td>0.010h</td>
</tr>
</tbody>
</table>

Fig. 2 shows story drift of structural model by MATLAB and ETABS.

IV. COMPARISON OF STRESSES OF STRUCTURAL MEMBERS

In this section, comparison is shown for stresses in structural
members through output files of ETABS and MATLAB programs. It is concerned that the feasible solution by MATLAB is judged not to be feasible by ETABS, caused by the differences of themes. In order to avoid over-stresses of elements by ETABS, a modified optimization method is proposed which reflects the differences between two analysis programs to the allowance limitation value in optimization procedures by MATLAB as:

\[ g(x) = \frac{R - R_{\text{limit}}}{R_{\text{limit}}} \leq \tilde{\tau} \]  

(21)

where, \( R, R_{\text{limit}} \) indicate response and response limit values, and \( \tilde{\tau} \) is the ratio of the response of MATLAB to ETABS for the most critical members and load combinations obtained from comparisons below.

The comparison includes dead, live and earthquake loads as well as all design load combinations. Figs. 5 and 6 show stresses in beams for dead, live and earthquake loads by both programs.

Figs. 7-12 show stresses in column and shear wall for dead, live and earthquake loads.

**V. STRUCTURAL DESIGN OF THE BUILDING**

Structural members must be designed to support all the loads acting on the structure greater than the service or actual loads in order to provide sufficient safety against failure. Loads can be forces for which a given structure might be proportioned such as dead, live or lateral loads. Based on ACI design code, the member is designed to resist factored loads multiplied by load factors. In addition, a strength reduction factor is considered to
account the degree of accuracy and variation of materials in strength design method and can be simply explained as:

Strength provided \( \geq \) strength required to carry factored load.

1. Beam number 17, \( b= 43 \text{ cm}, h= 65 \text{ cm} \) and \( L= 700 \text{ cm} \)
2. Column number 4, \( b= 95 \text{ cm}, D= 95 \text{ cm} \)
3. Shear wall number 2, \( t= 40 \text{ cm} \)

The strength required is obtained from structural analysis and applied safety factor, where the strength provided is illustrated in design code. Following structural members are selected for design, check and reinforcement detailing:

- Beam width \( b= 42 \text{ cm} \)
- Depth to tension reinforcement \( d= 61 \text{ cm} \)
- Total beam depth \( h= 65 \text{ cm} \)
- Clear cover \( d'= 4 \text{ cm} \)
- Concrete compressive strength \( f'_c = 27 \text{ MPa} \)
- Reinforcing yield strength \( f_y = 414 \text{ MPa} \)

1) Determine the Required Flexural Reinforcing of Beam

A simplified rectangular stress block approach based on flexural design procedure is considered, where the maximum depth of compression zone, \( c_{\text{max}} \), shall be calculated based on limitation of tensile steel tension not less than \( \varepsilon_{r_{\text{min}}} \) for tension controlled, where \( \varepsilon_{r_{\text{min}}} = 0.005 \), and \( \varepsilon_{r_{\text{max}}} = 0.003 [9] \).
The design steps to determine the required flexure and shear reinforcements are presented based on linear static analysis. The design process begins by identifying the required positive and negative moment capacities at mid-span and each support through determining of member loads, and then required reinforcement area is obtained using static approach discussed in Chapter III. Finally, the required reinforcement and its arrangement are checked to ensure that the minimum and maximum allowed are met.

Section 1 (L=0.5m), End-I:
Determine the required positive and negative flexural capacities at supports:

\[ M_{u,\text{top}} = -647.12 \text{KN-m} \]
\[ M_{u,\text{bottom}} = 323.56 \text{KN-m} \]

Determine the depth of the compression block:

\[ a_t = d - \frac{2M_u}{0.85 f'c \phi b} \]  
(22)

\[ a_t = 61 - \sqrt{\frac{2 \times 64712}{0.85 \times 2.7 \times 0.9 \times 41.4}} = 14.02 \text{cm} \]

\[ a_t = d - \frac{2M_u}{0.85 f'c \phi b} \]  
(23)

\[ a_t = 61 - \sqrt{\frac{2 \times 32356}{0.85 \times 2.7 \times 0.9 \times 41.4}} = 5.86 \text{cm} \]

The maximum depth of the compression zone can be calculated as:

\[ c_{\text{max}} = \frac{\varepsilon_{u,\text{max}} d}{\varepsilon_{u,\text{max}} + \varepsilon_{u,\text{min}}} \]

\[ c_{\text{max}} = \frac{0.003}{0.003 + 0.005} = 22.87 \text{cm} \]

The maximum allowable depth of the rectangular compression block can be given by:

\[ a_{\text{max}} = \beta_c c_{\text{max}} \]

\[ a_{\text{max}} = 0.85 \times 22.87 = 19.44 \text{cm} \]

If \( a \leq a_{\text{max}} \), the area of flexural steel reinforcement is given by formulas below, and the reinforcement is to be placed at the bottom for \( +M_u \) or the top for \( -M_u \).

\[ A_{s,\text{top}} = \frac{M_u}{\phi f'c (d - a)} \]

\[ A_{s,\text{top}} = \frac{64712}{0.9 \times 41.4 (61 - 14.02)} = 32.16 \text{cm}^2 \]

\[ A_{s,\text{bottom}} = \frac{32356}{0.85 \times 41.4 (61 - 5.86)} = 15.83 \text{cm}^2 \]

\[ A_{s,\text{bottom}} = \frac{4}{3} A_{s,\text{required}} \]

Applied reinforcing: 6ø2+5ø22

\[ A_{s,\text{bottom}} = \frac{M_u}{\phi f'c (d - a)} \]

\[ A_{s,\text{bottom}} = \frac{32356}{0.85 \times 41.4 (61 - 5.86)} = 15.83 \text{cm}^2 \]

\[ A_{s,\text{bottom}} = \frac{4}{3} A_{s,\text{required}} \]

Applied reinforcing: 5ø22

Calculate minimum and maximum area of flexural reinforcement:

\[ A_{s,\text{min}} = \frac{0.25 \sqrt{f'c}}{f_y} \]

\[ A_{s,\text{min}} = \frac{0.25 \sqrt{27}}{414} = 42 \times 61 = 8.03 \text{cm}^2 \]

\[ A_{s,\text{min}} = \frac{1.4 \times 42 \times 61}{414} = 8.66 \text{cm}^2 \]

\[ A_{s,\text{min}} = \frac{1.4 \times 42 \times 61}{414} = 8.66 \text{cm}^2 \]
Section 2 (L=4.7):

\[ M_{u,\text{top}} = -161.78 \text{KN-m} \]
\[ M_{\text{bottom}} = 263.24 \text{KN-m} \]

The maximum depth of the compression zone

\[ a_i = d - \sqrt{d^2 - \frac{2M_{u}}{0.85f'c \phi_b}} \]
\[ a_i = \frac{61 - \sqrt{61^2 - \frac{2 \times 16178}{0.85 \times 2.7 \times 0.9 \times 41.4}}}{2} = 3.18 \text{cm} \]
\[ a_{ii} = \frac{d - \sqrt{d^2 - \frac{2M_{u}}{0.85f'c \phi_b}}}{2} \]
\[ a_{ii} = \frac{61 - \sqrt{61^2 - \frac{2 \times 26324}{0.85 \times 2.7 \times 0.9 \times 41.4}}}{2} = 5.27 \text{cm} \]

Required flexural reinforcement:

\[ A_{u,\text{top}} = \frac{M_{u}}{\phi f_y(d - \frac{d}{2})} \]
\[ A_{u,\text{top}} = \frac{64964}{0.9 \times 41.4 \times (61 - 14.07)} = 32.3 \text{cm}^2 \]
\[ A_{u,\text{top}} = \frac{4}{3} \times 32.3 \text{cm}^2 = 43.06 \text{cm}^2 \]

Applied reinforcing: 6ø22+5ø22

The minimum flexural reinforcement required in a beam section is the minimum of the following:

\[ A_{\text{min}} = \frac{0.25f_y}{f_y} \text{bd} \]
\[ A_{\text{min}} = \frac{0.25 \times 42 \times 61}{414} = 8.03 \text{cm}^2 \]
\[ A_{\text{min}} = \frac{1.4}{f_y} \text{bd} \]
\[ A_{\text{min}} = \frac{1.4}{414} \times 42 \times 61 = 8.66 \text{cm}^2 \]

The beam flexural reinforcement is limited to a maximum given by:

\[ A_{i} \leq 0.025 bd \]
\[ A_{i} \leq 0.025 \times 42 \times 61 = 64.05 \text{cm}^2 \]

The balanced reinforcement ratio is given by:
The maximum allowable ratio of reinforcement is calculated as:
\[ \rho_{\text{max}} = \frac{0.85 f'_c}{f_y} \left( \frac{3}{7} \right) \]

The maximum allowable area of reinforcement is:
\[ A_{s,\text{max}} = \rho_{\text{max}} b d \]

The upper limit of 0.04 times the gross area of the tension reinforcement:
\[ A_t \leq 0.04 b d \]

The design shear force is obtained based on computer analysis by etabs as following:
\[ V_s = 0.17 \lambda \sqrt{f'_c b d} \]

Applied shear reinforcement: 104ø10@67mm c-c

Table VI shows flexural and shear reinforcements of beam

<table>
<thead>
<tr>
<th>Beam length (cm)</th>
<th>Beam depth h (cm)</th>
<th>Beam width b (cm)</th>
<th>Flexural reinforcement</th>
<th>Shear reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>700</td>
<td>65</td>
<td>42</td>
<td>top</td>
<td>11ø22</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>middle</td>
<td>5ø22</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>bottom</td>
<td>5ø22</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>104ø<a href="mailto:10@6.7cm">10@6.7cm</a></td>
</tr>
</tbody>
</table>

**B. Column Design**

1) Determine the Required Longitudinal Reinforcing of Column

The required amount of reinforcement was obtained by the help of etabs based on design internal forces as follows:

\[ P_u = 2856.2KN \]
\[ M_{u2} = 492.12KNm - m \]
\[ M_{u3} = -125.68KNm - m \]
\[ D/C \text{ Ratio} = 0.281 \]

The minimum and maximum longitudinal reinforcement is limited:
\[ A_{s,\text{min}} = 0.01 A_g \]
\[ A_{s,\text{max}} = 0.06 A_g \]

Provided longitudinal reinforcement falls between minimum and maximum allowable limit. Fig. 14 shows interaction diagrams of concrete member subjected to combined flexure and axial loads. The diagram shows the relationship between axial load and bending moments at failure.

The interaction surface points which represent the internal forces in each combination are inside the volume by critical curvature.

2) Determine the Required Shear Reinforcing of Column

The shear reinforcement is designed for each design combination in major and minor directions of the column. The nominal shear force shall not exceed the shear strength.

\[ V_s \leq \phi V'_s, \text{ where } V'_s = V_s + V_{V} \]

The design shear force is obtained based on computer analysis by etabs as following:

Major shear:
\[ V'_s = 446.57KN \]
For special moment resisting frame design, if the factored axial compressive force, $P_a$, including earthquake effects is small of \( (P_a \leq 2.7 \times 9025 / 20) \); if the shear force contribution from earthquake, $V_e$, is greater than half of the total factored maximum shear force, \( V_e \geq 0.5V'_c \), then the concrete capacity is taken zero, \( V_c = 0 \).

\[
(P_a \leq 2.7 \times 9025 / 20) = 1218.4KN < P_a
\]

and \( V_e = 399.5 > 0.5 \times 446.57 = 223.3KN \), then \( V_c = 0 \).

\[
\frac{A_s}{s} = \frac{V_e - \phi V_c'}{\phi f_yd}
\]

\[
\frac{A_s}{s} = \frac{V_e - 0}{\phi f_yd} = \frac{(446.57 - 0) \times 10^4}{0.65 \times 41.4 \times 91} = 1823 \text{ mm}^2 \text{ m}^{-1}
\]

Provided shear reinforcement: 17ø10@170mm c-c

Minor shear $V_1$:

\[
V_1 = 498.3KN
\]

\[
P_a = 2856.27KN
\]

\[
M_a = 489.16KN - m
\]

Shear force carried by concrete:

\[
(P_v \leq f'_{c}A_s / 20)
\]

\[
(P_v \leq 2.7 \times 2856.27 / 20) = 3855.9KN < P_a
\]

and \( V_e \geq 0.5V'_c \); \( V_e = 374.7 > 0.5 \times 498.3 = 249.15KN \), then

\[
\frac{A_s}{s} = \frac{V_e - \phi V'_{c}}{\phi f_yd}
\]

\[
\frac{A_s}{s} = \frac{(498.3 - 0) \times 10^4}{0.65 \times 41.4 \times 91} = 2035 \text{ mm}^2 \text{ m}^{-1}
\]

Provided shear reinforcement: 20ø10@150mm c-c

Table VII shows reinforcement provided in column under consideration.

<table>
<thead>
<tr>
<th>Station</th>
<th>Rebar Area (mm²)</th>
<th>Required Reinf Ratio</th>
<th>Current Reinf Ratio</th>
<th>Wall $A_g$ mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
<td>5686</td>
<td>0.0047</td>
<td>0.0026</td>
<td>1200000</td>
</tr>
<tr>
<td>Bottom</td>
<td>3000</td>
<td>0.0025</td>
<td>0.0026</td>
<td>1200000</td>
</tr>
</tbody>
</table>

C. Shear Wall Design

1) Determine the Required Longitudinal Reinforcing of Shear Wall

Design and check of shear wall is done based on stresses on shear wall by help of etabs, then the required reinforcement is checked manually. Table VIII shows the design load combination associated with the specified required reinforcing area.

<table>
<thead>
<tr>
<th>Station</th>
<th>Flexural Combo</th>
<th>$P_u$ kN</th>
<th>$M_{u2}$ kN-m</th>
<th>$M_{u3}$ kN-m</th>
<th>Length mm</th>
<th>Thickness mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
<td>DWal4</td>
<td>4342.6</td>
<td>643.6</td>
<td>4195.3</td>
<td>3000</td>
<td>400</td>
</tr>
<tr>
<td>Bottom</td>
<td>DWal10</td>
<td>4186.16</td>
<td>-524.5</td>
<td>-2526.3</td>
<td>3000</td>
<td>400</td>
</tr>
</tbody>
</table>

Table IX shows the amount of reinforcement corresponding to the above load combination.

<table>
<thead>
<tr>
<th>Column depth D(cm)</th>
<th>Column width b(cm)</th>
<th>Flexural reinforcement</th>
<th>Shear/transvers reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>95</td>
<td>95</td>
<td>24 ø22</td>
<td>20 10@15cm</td>
</tr>
</tbody>
</table>

Based on ACI design code, the minimum longitudinal area of reinforcement shall be \( A_{s,\text{min}} = 0.0025A_g \)

\[
A_{s,\text{min}} = 0.0025 \times 400 \times 3000 = 3000 \text{ mm}^2
\]

Based on computer analysis, the required longitudinal reinforcement ratio is less than current reinforcement ratio, so we consider the amount of reinforcement ratio required by analysis:

\[
A_{s,\text{required}} = 5686 \text{ mm}^2 > A_{s,\text{min}} = 3000 \text{ mm}^2
\]

Applied reinforcement: 12ø 22 in two layers.
2) Boundary Element Check

There are two approaches to check the requirements for boundary element, a) if the maximum extreme fiber compressive strength, $\sigma \geq 0.2 f'c$, or b) $c \geq \frac{l_u}{600(1.5\delta_y / h_u)}$, where $\delta_y / h_u \geq 0.005$, then boundary element is required.

\[
\delta_y = \delta_{y, \text{elastic analysis}} \left( \frac{C_y}{f'} \right)
\]

\[
\delta_y = 19\left( \frac{5}{1} \right) = 95\text{mm}
\]

\[
c \geq \frac{l_u}{600(1.5\delta_y / h_u)}
\]

where $c = 852\text{mm}$ by etabs $\frac{3000}{600(1.5 \times 95/3000)} = 105\text{mm} < c$; the boundary element is required.

The boundary element shall extent vertically above and below the critical section at least:

\[
M_u = 3018.6\text{KN} - \text{m} = 940\text{mm}, \quad h_u = 1300\text{mm}
\]

\[
M_u = \frac{3018.6\text{KN} - \text{m}}{4 \times 802.56\text{KN}} = 940\text{mm}, \quad h_u = 1300\text{mm}
\]

The minimum required length of boundary zone at each end of the wall shall be:

\[
l_b = \max \left\{ \frac{c}{2}, c - 0.1l_u \right\}
\]

\[
l_b = \max \left[ \frac{852}{2}, 852 - 0.1 \times 3000 \right] \text{mm} = \left\{ 426, 552 \right\} \text{mm}
\]

The required shear reinforcement can be obtained as:

\[
A_s = \frac{(\Phi V_n - \Phi V_c)}{\phi f_y d}
\]

\[
d = 0.8l_u
\]

\[
A_s = \frac{1676.6 - 931.9}{0.75 \times 41.4 \times 0.8 \times 300} = \frac{1676}{12} \times 4 \times 975^2 = 5.3
\]

After checking of applied flexural reinforcement, the amount of provided flexural reinforcement was not sufficient to resist corresponding load combinations. Therefore, the shear wall was designed using section designer tool, and the applied reinforcement for wall and boundary zones are as following:

- Total wall length: 3000 mm
- Clear cover: 25 mm
- Reinforcement of wall: $8\phi 18@221 \text{ mm c-c each layer}$
- Boundary length: 550 mm
- Boundary reinforcement: $4\phi 20@168 \text{ c-c each layer}$

### Table X

<table>
<thead>
<tr>
<th>Station Location</th>
<th>Edge Length (mm)</th>
<th>Governing Combo</th>
<th>$P_o$ (kN)</th>
<th>$M_o$ (kN-m)</th>
<th>Stress Comp Limit (MPa)</th>
<th>Stress Limit (MPa)</th>
<th>C (Depth Limit mm)</th>
<th>C (Depth Limit mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top–Left</td>
<td>488.2</td>
<td>DWal3</td>
<td>4076.2</td>
<td>-3018.6</td>
<td>8.4</td>
<td>5.4</td>
<td>788.2</td>
<td>112.2</td>
</tr>
<tr>
<td>Top–Right</td>
<td>538.7</td>
<td>DWal3</td>
<td>4541.6</td>
<td>588.2</td>
<td>4.8</td>
<td>5.4</td>
<td>837.6</td>
<td>666.7</td>
</tr>
<tr>
<td>Bottom–Left</td>
<td>552.5</td>
<td>DWal6</td>
<td>4651.8</td>
<td>-271.3</td>
<td>4.3</td>
<td>5.4</td>
<td>852.5</td>
<td>666.7</td>
</tr>
<tr>
<td>Bottom–Right</td>
<td>528.8</td>
<td>DWal6</td>
<td>4452.9</td>
<td>1787.5</td>
<td>6.7</td>
<td>5.4</td>
<td>828.8</td>
<td>112.2</td>
</tr>
</tbody>
</table>

### Table XI

<table>
<thead>
<tr>
<th>Station Location</th>
<th>ID</th>
<th>Shear Combo</th>
<th>$P_o$ (kN)</th>
<th>$M_o$ (kN-m)</th>
<th>$V_o$ (kN)</th>
<th>$\Phi V_o$ (kN)</th>
<th>$\Phi V_n$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
<td>Leg 1</td>
<td>DWal4</td>
<td>4342.6</td>
<td>4195.3</td>
<td>802.6</td>
<td>918.4</td>
<td>1663</td>
</tr>
<tr>
<td>Bottom</td>
<td>Leg 1</td>
<td>DWal4</td>
<td>4452.7</td>
<td>1787.6</td>
<td>802.6</td>
<td>932</td>
<td>1676.6</td>
</tr>
</tbody>
</table>

The required shear reinforcement can be obtained as:

\[
A_s = \frac{(\Phi V_n - \Phi V_c)}{\phi f_y d}
\]

\[
d = 0.8l_u
\]
Total height of wall: $5.3 \times 3 = 16$
Provided shear reinforcement: $16 \varnothing 10@187\text{mm c-c}$

Table XII shows provided reinforcement of shear wall under consideration.

| TABLE XII |
|---|---|---|---|---|---|
| TOTAL SHEAR WALL LENGTH (cm) | FLEXURAL REINFORCEMENT | TRANSVERS/SHEAR REINFORCEMENT | BOUNDARY LENGTH | FLEXURAL REINFORCEMENT | SHEAR/TRANSVERS REINFORCEMENT |
| 300 | 8\varnothing 18@22cm | 16\varnothing 10@18cm | 55 | 4\varnothing 20@16.8cm | 30\varnothing 10@10cm |

VI. CONCLUSION

In this study, the seismic design of structure was done following International Building Code (IBC 2012), American Society of Civil Engineering (ASCE 7-10) standards, and American Concrete Institute Building Code (ACI 318-14). Typical reinforcement requirements for structural wall, beam and column were discussed and presented using ETABS structural analysis software. The placement and detailing of reinforcement of structural members were explained and discussed as conclusion.

ACKNOWLEDGMENT

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REFERENCES

[1] Building Code Requirements for Structural Concrete (ACI 318M-14) and Commentary (ACI 318-RM-14)