Re-planning of Concrete Structures in the Ngoro Dormitory Project in Surabaya
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Abstract
The current trend of building construction is using steel frames and reinforced concrete frames. The construction design concept is based on strength limit analysis (ultimate strength) which has sufficient ductility to absorb earthquake energy according to applicable regulations. In addition to this, it is also necessary to consider the economic aspect. All of these structural considerations will influence planning alternatives, such as column layout, beam length, and span. Dormitori Ngoro Surabaya is one of the low-rise buildings that will be redesigned to comply with SNI 2847:2013. This research is expected to answer the need for earthquake-resistant building construction using the ETABS program and according to SNI 2847:2013 standards. In this study the method used with the concept of Flowcharts in order to facilitate the research stage. As for this study, it produced a maximum moment of 137.58 KNm with the use of beam dimensions of 40 x 75 cm, using 8D16 top main reinforcement and 3D16 bottom main reinforcement with stirrups D10-200 in the support area and D10-100 in the field area. Whereas the ratio of beam reinforcement based on manual calculations is 0.24% and the ETABS results are 0.22%.

Keywords: Construction; design; earthquake; etabs; reinforcement

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1. Introduction

Earthquake disaster is a natural disaster event that often occurs in Indonesia (Indriasari, 2018). For example, the earthquake in Bantul, Yogyakarta in 2006 (Hamid, 2020), although the magnitude of the earthquake was not very large (Somantri, 2021; Siswanto & Salim, 2018), it caused extensive damage and claimed many victims (Maulina, 2017; Kusno, 2020). The earthquake that occurred in Bantul Regency, Yogyakarta is a lesson for us that we need to be prepared in facing earthquake disasters in the future (Irawan et al., 2022).

Sometimes the earthquake itself does not cause disaster directly to humans (Hadibroto & Ronitua, 2018). But treasures and objects (Chazawi, 2021). Besides that, there was also a structural collapse of modern buildings (Baso et al., 2017; Aristyawati et al., 2021). However, along with the development of technology (Azis, 2022; Azis et al., 2021) and progress (Azis et al., 2020), people’s thinking patterns view technology as one of the main elements in planned development (Krisnani et al., 2017; N. Azis, 2021). Then the concept of earthquake-resistant buildings emerged among the public and building construction practitioners (Rumbyarso & Pribadi, 2022; Husein, 2016; Zega et al., 2022).

Surabaya is the second largest city after Jakarta besides that it is a potential place for investors (Harnindra et al., 2017). The city of Surabaya is passed by two active faults that have the potential for an earthquake measuring 6.5M (Farichah & Kumala Sari, 2019). Seeing this phenomenon, the construction of structures and infrastructure should pay attention to seismic aspects to avoid losses due to earthquakes (Simanjuntak, 2020). Seeing the phenomenon above,

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this research is focused on one of the buildings in the Surabaya area, namely Dormitory Ngoro Surabaya, which is one of the low-rise buildings that will be redesigned to comply with SNI 2847:2013 (Febriana et al., 2022)

2. Research Method

In the research method used is to use flow charts or flow charts. The flow chart method is a method that represents a process, system, or computer algorithm and is commonly used to document, plan, refine, or describe a workflow with many steps. The flowchart in this study is presented in Figure 1.

![Fig 1. Research Flowchat](image)

3. Result and Discussion

In the early stages, pre-planning calculations were carried out which were calculated manually to calculate the dimensions of structural plans such as slabs, beams and columns in order to obtain an optimal value. The calculation includes:

a. Pre Planned Plates
b. Pre Beam Plan
c. Pre Plan Column
Then proceed with loading. The structure that is carried out is:

a. Additional Dead and Dead Loads
b. Live Load
c. Earthquake Loading
d. Loading Combination

From the combined loading test, data is obtained as shown in table 1.

### Table 1. Load Combination Calculation

<table>
<thead>
<tr>
<th>Number</th>
<th>Combination</th>
<th>D</th>
<th>L</th>
<th>SIDL</th>
<th>SPECX</th>
<th>SPECY</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>U1</td>
<td>1.40</td>
<td>0</td>
<td>1.40</td>
<td>0</td>
<td>0</td>
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<tr>
<td>2</td>
<td>U2</td>
<td>1.20</td>
<td>1.6</td>
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<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>U3-1</td>
<td>1.20</td>
<td>1</td>
<td>1.20</td>
<td>0</td>
<td>0</td>
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<td>4</td>
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<td>1.29</td>
<td>0.3</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>U5-2</td>
<td>1.29</td>
<td>1</td>
<td>1.29</td>
<td>0.3</td>
<td>-1</td>
</tr>
<tr>
<td>6</td>
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<tr>
<td>7</td>
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<td>-1</td>
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<tr>
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<td>U5-5</td>
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<td>1</td>
<td>1.29</td>
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<td>1.29</td>
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<td>-0.3</td>
</tr>
<tr>
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<td>1.29</td>
<td>-1</td>
<td>0.3</td>
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<tr>
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<td>1</td>
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<td>-1</td>
<td>-0.3</td>
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<tr>
<td>12</td>
<td>U7-1</td>
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<td>0</td>
<td>0.81</td>
<td>0.3</td>
<td>1</td>
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<tr>
<td>13</td>
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<td>0</td>
<td>0.81</td>
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<td>17</td>
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<td>1</td>
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<tr>
<td>18</td>
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<td>0.81</td>
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<td>0.3</td>
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<td>19</td>
<td>U7-8</td>
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<td>0.81</td>
<td>-1</td>
<td>-0.3</td>
</tr>
</tbody>
</table>

From the data obtained, the fundamental period is checked with the provisions imposed in SNI Earthquake 1727: 2013, then a comparison is made according to SNI requirements between the calculation fundamental period and the fundamental period from ETABS. The ETABS fundamental period can be obtained from displayà show table à modal information à modal participating mass ratios, here is the ETABS fundamental period data. As presented in Table 2.

### Table 2. Mass Modal Period Data from ETABS

<table>
<thead>
<tr>
<th>Mode</th>
<th>Period</th>
<th>SumUX</th>
<th>SumUY</th>
<th>SumUZ</th>
<th>RX</th>
<th>RY</th>
<th>RZ</th>
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</thead>
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<td>1.989</td>
<td>84.729</td>
<td>0.002</td>
<td>0</td>
<td>0.002</td>
<td>99.106</td>
<td>0.277</td>
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<tr>
<td>2</td>
<td>1.801</td>
<td>84.737</td>
<td>0.00</td>
<td>84.456</td>
<td>98.461</td>
<td>0.009</td>
<td>0.633</td>
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<tr>
<td>3</td>
<td>1.662</td>
<td>84.998</td>
<td>86.086</td>
<td>0</td>
<td>0.813</td>
<td>0.293</td>
<td>85.072</td>
</tr>
<tr>
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<td>95.495</td>
<td>86.086</td>
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<td>0.339</td>
<td>0.037</td>
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<tr>
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<td>95.498</td>
<td>96.065</td>
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<td>0.501</td>
<td>0.000</td>
<td>0.097</td>
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<tr>
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<td>95.535</td>
<td>96.164</td>
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<td>0.004</td>
<td>0.001</td>
<td>9.933</td>
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<td>96.165</td>
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<td>0.000</td>
<td>0.250</td>
<td>0.018</td>
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<tr>
<td>8</td>
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<td>99.149</td>
<td>99.306</td>
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<td>0.213</td>
<td>0.000</td>
<td>0.040</td>
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<tr>
<td>9</td>
<td>0.330</td>
<td>99.167</td>
<td>99.346</td>
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<td>0.001</td>
<td>3.186</td>
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<tr>
<td>10</td>
<td>0.280</td>
<td>99.984</td>
<td>99.352</td>
<td>0</td>
<td>0.000</td>
<td>0.000</td>
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<tr>
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<td>0.000</td>
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</table>

Furthermore, the calculation of the deviation between floors in the X and Y directions was carried out and the data obtained as presented in Table 3.
Then continue the calculation of P-Delta in the X and Y directions. As presented in Table 5 and Table 6.

**Table 5. X-Direction P-Delta Calculation**

<table>
<thead>
<tr>
<th>STORY</th>
<th>Dynamic Earthquake</th>
<th>High (H) (mm)</th>
<th>Elevation Hx (mm)</th>
<th>DriftX (mm)</th>
<th>IZIN (0.02*H) (m)</th>
<th>DriftX*H(mm²)</th>
<th>Δ (DriftX<em>H</em>Cd)(DriftX<em>H</em>4,5)</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>LT.5</td>
<td>SPECX</td>
<td>4000</td>
<td>16000</td>
<td>0.001100</td>
<td>80</td>
<td>4.004</td>
<td>18.018</td>
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</tr>
<tr>
<td>LT.4</td>
<td>SPECX</td>
<td>4000</td>
<td>12000</td>
<td>0.00164</td>
<td>80</td>
<td>6.556</td>
<td>29.502</td>
<td>OK</td>
</tr>
<tr>
<td>LT.3</td>
<td>SPECX</td>
<td>4000</td>
<td>8000</td>
<td>0.00208</td>
<td>80</td>
<td>8.320</td>
<td>37.440</td>
<td>OK</td>
</tr>
<tr>
<td>LT.2</td>
<td>SPECX</td>
<td>4000</td>
<td>4000</td>
<td>0.00169</td>
<td>80</td>
<td>6.740</td>
<td>30.330</td>
<td>OK</td>
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</tbody>
</table>

**Table 6. Y Direction P-Delta Calculation**

<table>
<thead>
<tr>
<th>STORY</th>
<th>Dynamic Earthquake</th>
<th>High (H) (mm)</th>
<th>Elevation Hx (mm)</th>
<th>DriftY (mm)</th>
<th>IZIN (0.02*H) (m)</th>
<th>DriftY*H(mm²)</th>
<th>Δ (DriftY<em>H</em>Cd)(DriftY<em>H</em>4,5)</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>LT.5</td>
<td>SPECY</td>
<td>4000</td>
<td>16000</td>
<td>0.00745</td>
<td>80</td>
<td>2.980</td>
<td>13.410</td>
<td>OK</td>
</tr>
<tr>
<td>LT.4</td>
<td>SPECY</td>
<td>4000</td>
<td>12000</td>
<td>0.001261</td>
<td>80</td>
<td>5.004</td>
<td>22.698</td>
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</tr>
<tr>
<td>LT.3</td>
<td>SPECY</td>
<td>4000</td>
<td>8000</td>
<td>0.00164</td>
<td>80</td>
<td>6.560</td>
<td>29.520</td>
<td>OK</td>
</tr>
<tr>
<td>LT.2</td>
<td>SPECY</td>
<td>4000</td>
<td>4000</td>
<td>0.00145</td>
<td>80</td>
<td>5.780</td>
<td>26.010</td>
<td>OK</td>
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</tbody>
</table>

**Table 7. Earthquake-induced Shear Force in X-Direction**

<table>
<thead>
<tr>
<th>STORY</th>
<th>Column</th>
<th>Load Case Combo</th>
<th>Station</th>
<th>P kN</th>
<th>V2 Kn</th>
<th>V3 Kn</th>
<th>T Kn-m</th>
<th>M2 Kn-m</th>
<th>M3 Kn-m</th>
</tr>
</thead>
<tbody>
<tr>
<td>LT.5</td>
<td>C18</td>
<td>COMB2</td>
<td>0</td>
<td>-206.53</td>
<td>18.21</td>
<td>14.21</td>
<td>-0.15</td>
<td>26.89</td>
<td>32.47</td>
</tr>
<tr>
<td>LT.5</td>
<td>C18</td>
<td>COMB2</td>
<td>1.625</td>
<td>-195.06</td>
<td>18.21</td>
<td>14.21</td>
<td>-0.15</td>
<td>3.80</td>
<td>2.88</td>
</tr>
<tr>
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<td>C18</td>
<td>COMB2</td>
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<td>-183.58</td>
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<td>14.21</td>
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<td>-19.28</td>
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</tr>
<tr>
<td>LT.5</td>
<td>C18</td>
<td>COMB4</td>
<td>0</td>
<td>-210.15</td>
<td>17.39</td>
<td>14.07</td>
<td>-0.14</td>
<td>26.42</td>
<td>30.59</td>
</tr>
<tr>
<td>LT.5</td>
<td>C18</td>
<td>COMB5</td>
<td>0</td>
<td>-210.15</td>
<td>17.39</td>
<td>14.07</td>
<td>-0.14</td>
<td>26.42</td>
<td>30.59</td>
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<td>COMB6</td>
<td>0</td>
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<td>LT.5</td>
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<td>14.07</td>
<td>-0.14</td>
<td>26.42</td>
<td>30.59</td>
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</tbody>
</table>

Then to find out the maximum moment must know the beam forces. The data results from etabs the largest maximum moment is at the location of the 8 meter beam span with a moment of 137.58 kN.m in the field and 105.21 kN.m at the pedestal. For the maximum moment of the column column forces / column strength of the largest etabs on the 3rd...
floor with a column height of 4 meters the field moment is 883.37 kn.m and the pedestal moment is 32.21 kn.m. The obtained shear forces arising from the earthquake in the X direction and Y direction can be seen in Table 7 and Table 8.

From the results of the ETABS analysis, it can be seen that the shear force due to the X-direction earthquake is the column with code C18 located on the 5th floor or roof floor with a maximum load in the x-direction of the building of 18.21 Kn and the y-direction of 14.21 Kn.

Table 8. Earthquake-induced Shear Force in Y-Direction

<table>
<thead>
<tr>
<th>STORY</th>
<th>Column</th>
<th>Load Case Combo</th>
<th>Station</th>
<th>P</th>
<th>V2</th>
<th>V3</th>
<th>T</th>
<th>M2</th>
<th>M3</th>
</tr>
</thead>
<tbody>
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<td>C18</td>
<td>COMB2</td>
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<td>-150.95</td>
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<td>COMB2</td>
<td>1.625</td>
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<td>29.34</td>
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<td>COMB2</td>
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<td>29.34</td>
<td>-0.09</td>
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<td>-0.09</td>
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<td>COMB6</td>
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<td>COMB7</td>
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<td>4.29</td>
<td>28.36</td>
<td>-0.09</td>
<td>53.28</td>
<td>7.05</td>
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</table>

From the results of the ETABS analysis, it can be seen that the shear force due to the Y-direction earthquake is the column with code C23 located on the 5th floor or roof floor with a maximum load in the y-direction of the building of 29.34 Kn and the x-direction of 4.46 Kn.

The last step is the Reinforcement of Beams and Columns

Beam Reinforcement Design 40x75

Details of the area of 40x75 beam reinforcement are presented in Figure 2.

Deformed reinforcement (D16) is used, then:

$$A_{ce} = \frac{1}{4} \pi x d^2 = \frac{1}{4} \times 3.14 \times 162 = 200.96 \text{ mm}^2$$

Deformed reinforcement (D13) is used, then:

$$A_{ce} = \frac{1}{4} \pi x d^2 = \frac{1}{4} \times 3.14 \times 132 = 132.67 \text{ mm}^2$$

a. The main reinforcement of the support area:

$$\text{Upper reinforcement area} = 477, \text{amount of reinforcement} = 477/200.96 = 2.4 \sim 3$$

$$\text{Bottom reinforcement area} = 418, \text{amount of reinforcement} = 418/132.67 = 3.15 \sim 3$$

b. Field area main reinforcement:

$$\text{Upper reinforcement area} = 198, \text{amount of reinforcement} = 198/132.67 = 1.49 \sim 2$$

$$\text{Bottom reinforcement area} = 657, \text{amount of reinforcement} = 657/200.96 = 3.26 \sim 3$$

Using plain stirrup reinforcement diameter 8, then:

$$A_{ce} = \frac{1}{4} \pi x d^2 = \frac{1}{4} \times 3.14 \times 102 = 78.5 \text{ mm}^2$$
a. Shear reinforcement area of support:
   Assuming stirrups are used 2 D10-150
   Area of reinforcement per 1 meter = 2 x ¼ d2 x 1000/150
   Rebar area per 1 meter = 2 x ¼ . 3.14 . 102 x 1000/150 = 1046.66 mm²
   So the area per meter of length = 1.046 mm²
   Security control : Ldesign > Lplan = 1.046 > 0.06 (OK)

b. Field area shear reinforcement:
   Assuming 2D10-100 stirrups are used
   Area of reinforcement per 1 meter = 2 x ¼ d2 x 1000/100
   Rebar area per 1 meter = 2 x ¼ . 3.14 . 102 x 1000/100 = 1570 mm²
   So the area per meter of length = 1.570 mm²
   Security control : Ldesign > Lplan = 1.570 > 0.07 (OK)

Based on SNI Concrete 03-2847-2013, SRPMM flexible structural components must meet the following requirements:

a. The factored axial force on the beam is limited to a maximum Ag x fc'/10
   Check: (400x750) x 30 / 10 = 900 KN
   From the ETABS calculation the axial force that occurs is 0.6470 KN, then:
   0.6470 < 900 KN (OK)

b. A structural clear span of at least 4x the effective height
   Effective height (d) = h – (SB + Dsengkang + ½ x Dutama)
   = 750 – (50 + 10 + ½ x 16) = 682 mm x 4 = 2728 mm
   Beam clear span = beam span – column dimensions
   = 8000 – 500 = 7500 mm
   Then 7500 > 2728 (OK)

c. The minimum width and height ratio is 0.3
   b = 400 mm, h = 750 mm, b/h = 0.53
   So, 0.53 > 0.3 (OK)

d. Element width must not be:
   Less than 250mm
   b = 400mm > 250mm (OK)

e. Longitudinal reinforcement requirements
   The area of top and bottom reinforcement shall not be less than the minimum reinforcement requirements
   according to SNI 2847-2013:
   As min = (0.25 x √fc')/(fy ) x bxdt = (0.25 x √30)/(400 ) x400x682= 933.86mm
   As min = (1.4 xbx s)/fy = (1.4 x 400 x 682)/400 = 954.8 mm
   Based on the ETABS output:
   Top reinforcement area = 711 mm
   Area of bottom reinforcement = 344 mm
   So: 198 + 657 = 1055 mm > 954.8 mm (OK)

f. Check reinforcement ratio
   ρ = As/(bxd) = 1055/400x750 = 0.0035
   ρb = β 0.85fc'/fy x 600/(600+fy) = 0.85 0.85x30/400 x 600/(600+400) = 0.03251
   ρmax = 0.75 x ρb = 0.75 x 0.03251 = 0.0243 = 0.24%
The limit of reinforcement ratio used is 0.03251, then:
\[ \rho < \rho_{\text{max}} \text{ and } \rho < 0.0243 \text{ (OK)} \]

From the calculation above, it is obtained:

a. Upper main reinforcement, (2D16) and lower main reinforcement (3D16)
b. The stirrup reinforcement used is 2D10-100 and 2D10-150

For the display of reinforcement can be seen in Figure 3.

![Figure 3. Details of Beam Reinforcement 40x75 cm](image)

For the design of column reinforcement. The area of the main column reinforcement can be automatically determined by means of Design – Concrete Frame Design – Display Design Info – Longitudinal Reinforcing. The following is an image of a floor plan of one of the column types that will be analyzed and the reinforcement calculation will be included. Which can be seen in figure 4.

![Figure 4. Field Support Reinforcement](image)

Column dimensions : 500 x 500 mm  
Concrete quality : 40 Mpa  
Reinforcement quality : 400 Mpa  
Diameter of longitudinal reinforcement : 20 mm  
Diameter of transverse reinforcement : 10 mm  
Concrete covers : 50 mm  
Reinforcement Requirement \( A_s = \% \text{ ETABS x Cross-sectional area} = 1\% \times (500\times500) = 2500 \text{ mm}^2 \)  
Reinforcement Diameter Area = \( \frac{1}{4} \times \pi x d^2 \) = \( \frac{1}{4} \times 3.14 \times 222 = 379.94 \text{ mm}^2 \)  
Number of reinforcement = 2500/379.94 = 6.57 = 8 pieces
Total area of reinforcement = 8 x 379.94 = 2512 mm²
Design = 2512/(250000) = 1.00048%
As < Total Bar Area OK
2500 < 2512 OK
Reinforcement Needs = 8D22

From the calculation above, it is obtained:

a. Main reinforcement (8D22)
b. The stirrup reinforcement used is 2D10-150

Detailed Images of Column Reinforcement 50x50 cm can be seen in the figure 5.

**Fig 5. Column Reinforcement Details 50x50 cm**

4. Conclusion

Based on the structural analysis results of ETABS V9.6.0 for columns, beams and slabs, it is obtained: 1). The structural design steps using ETABS and SNI standards start from the preliminary (Dimensional Planning), material specification planning, structural modeling, load calculations (dead loads, live loads and earthquake loads), and the final stage is conducting structural analysis. 2). Obtained loading analysis as follows: a). From the results of the ETABS analysis it can be seen that based on the dimensions of the beams and columns there is no over strength found, it can be said that they meet the SNI requirements. b). From the results of the ETABS analysis run in Shape 1 and Shape 2 modes, no torsion was found and a displacement of 0.29% was obtained. c). Based on the calculation, the maximum moment is 137, 58 KNm For beam dimensions 40x75 cm using 8D16 top main reinforcement and 3D16 bottom main reinforcement with stirrups D10-200 in the support area and D10-100 in the field area. d). The ratio of beam reinforcement based on manual calculations is 0.24% and 0.22% ETABS results. e). For columns with dimensions of 50x50 cm using main reinforcement 8D22 and D10-150 for stirrups. f). Column reinforcement ratio based on manual calculation is 1.00048% according to the results of 1% etabs. g). For slabs, use the same reinforcement dimensions as the existing one due to the use of bondek. h). Deflection of the beam at the longest span is obtained at 3.7 cm. So, 3.7 cm > 800/360 cm 3.7 cm > 2, 22 cm, in this case the height of the beam must be increased. 3). The column dimensions are 50x50 cm, the main beam dimensions at the longest stretch are 8 meters 40x75 cm, dimensions of the main beam on a stretch of 4 meters 30x50 and 25x40 cm joists. The thickness of the 2nd and 3rd floor plates is 150 mm while the 4th floor and roof floor are 120 mm.


References


