Design of reinforced concrete truss systems in earthquake-resistant high-rise buildings

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ABSTRACT
This study aims to find out how to plan the Reinforced Concrete Bar Frame Structure which includes the structure of columns and beams and their reinforcement to meet the design concept of capacity, namely strong columns and weak beams. In this study, it will be planned on the Nusa Putra Islamic Boarding School building which amounts to 3 floors using the Special Moment Resistant Frame System (SRPMK) in accordance with SNI 1726-2012. The earthquake load used is Spectrum Response Analysis by taking into account three different types of soil conditions namely hard, medium and soft soils. Moment Resistant Frame System is a spatial frame system in which structural components and their joints resist forces acting through bending, sliding and axial action. The quality of the concrete material used is 25 MPa and the reinforcing steel material used 400 MPa threaded iron while the beam dimensions are 300 mm x 400 mm and the column is 500 mm x 500 mm. The results obtained on the beam structure in hard soil conditions Mu = -85.5012 kN (support for) 4D19; Mu = 42.7506 kN (pedestal under) used 3D16; and Mu = 30.2581 kN (in the middle span) used 3D16; on medium soil Mu = 92.0741 kN (support for) 4D19; Mu = 46.03705 kN (lower pedestal) used 3D16; and Mu = 59.4276 kN (center span) used 3D16 + 1D13; on soft soil Mu = -107.842 kN (upper support) 5D19 was used; Mu = 53.921 kN (pedestal under) used 4D16; and Mu = 63.4546 kN (center span) used 4D16; Axial force occurs in the main column due to the combination of the three types of soil is not too significantly different, on hard soil = 337,949 kN, medium soil = 339,785 kN, soft soil = 342,954 kN, so column reinforcement in all three uses 12D22.

KEYWORDS
Concrete truss
SRPMK
Spectrum Response Analysis
Reinforcement diameter

1. Introduction
Most of Indonesia is an area that has a high level of vulnerability to earthquakes [1]. This is because Indonesia is located between three confluence of the world's plates, namely the Australian Plate, the Eurasian Plate, and the Pacific Plate [2]. In addition, Indonesia is also included in the Pacific Ring of Fire, which is a group of volcanoes in the world [3]. As a result of plate movements and volcanic activity, we can see that recently there have been many earthquakes in several areas in Indonesia which caused many fatalities to be lost, and one of them that often occurs due to earthquakes is the damage to a high-rise building [4].

Based on SNI-1726-2012 [5], Earthquake Resistance Planning Procedures for Building and Non-Building Structures states that Indonesia is divided into 6 seismic design categories, namely, KDS A, B, C, D, E, and F, to ensure a building is able to minimize the damage caused by the earthquake, must be planned in such a way and in as much detail as possible the structure of the building so that the building made is safe against all forces caused, especially during an earthquake [6]. In planning a building, of course, it has a different location, so the type of soil at that location will be different [7]. The difference in the type of soil will determine the difference in the earthquake response generated in the structure of the building to be planned [8]. Therefore, buildings located in a certain earthquake area with an acceleration of bedrock peaks for a 500-year return period do not necessarily have the same acceleration...
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Fig. 1. Research flow
3. Results and Discussion

3.1. Building Planning Information

In planning a building, of course, the need for space in a building has been determined, and the floor height, the number of floors, building area of each floor and the location of the building structures have been determined, as an illustration and initial information in determining the structural system, structural dimensions, and structural reinforcing steel. Initial information as a basis for structural design based on information from building planning consultants explained building plans with structural planning plans as shown in Figure 2.

![Fig. 2. Structural plans](image)

To determine the dimensions and reinforcement of the structure of course requires complete information about the plan of a building, but presents the structural planning plan in Figure 2 as an example that is relevant to structural analysis [16]. As information data to carry out further analysis, it is necessary to know information from a building including building specifications, materials, structural dimension plans, and structural loading plans. Information about these buildings is then presented in Table 1.

### Table 1. Specifications of Building Structures, Materials, Dimensions of Structures, and Planned Loads of Structures

| Building Function                  | Islamic Boarding School Building |
|------------------------------------|-----------------------------------|
| Building area                      | 600 m²                            |
| Number of Floors                   | 3 floors                          |
| Height of Each Floor               | 4 m                               |
| Roof Construction                  | Light steel truss                 |
| Roof Cover                         | Clay roof tiles                  |
| Mutu Beton (f’c)                   | 25 Mpa                            |
| Non-threaded Profile Reinforcement Steel Quality (fy) | 240 Mpa                          |
| Thread Profile Reinforcement Steel Quality (fy) | 400 Mpa                          |

| Dimensions of Structural Planning  |
|------------------------------------|
| Main Column (K1)                   | 500 mm x 500 mm                  |
| Second column (K2)                 | 150 mm x 250 mm                  |
| L. column (KL)                     | 150 mm x 900 mm x 100 mm         |
| Main Beam (B1)                     | 300 mm x 400 mm                  |
| Floor plate 2 and 3               | 120 mm                           |

| Structure Load                     |
|------------------------------------|
| Live load (LL)                     | 250 kg/m²                        |
| Dead load on floor slab            | 1722 kg/ m²                      |
| Wind load on the roof              | 6.72 kg/m² (tekan), -13.44 kg/m² (hisap) |
| Earthquake load response spectrum  | Not yet known in planning        |

After knowing the specifications of the building structure, materials, dimensions of the structure, and the plan for loading the structure, then it is necessary to know the earthquake load from the response spectrum. The response spectrum of each earthquake area with an acceleration of bedrock peaks for a
500-year return period does not necessarily have the same acceleration of earthquake response, the spectrum response will depend on the type of soil on which a building stands [17]. Soil types that affect the response spectrum consist of hard soil, medium soil, and soft soil [18]. Response spectrum based on soil type as illustrated in Figure 3.

![Design Spectrum Response Graph](image)

**Fig. 3.** Response Spectrum Design

Based on initial information about the technical specifications of the building, it is necessary to calculate the moments that occur in a structure. This study calculates the moments using a computer application [19], and the results of the analysis show that the moments that occur in a structure are as shown in Table 2.

| Cross Section Profile(cm) | Negative support moment (kNm) | Positive support moment (kNm) | Field Moment (kNm) | Shear (kN) | Axial (kN) |
|---------------------------|-------------------------------|-------------------------------|--------------------|-----------|-----------|
| B1 (25x50)                | -130.214                      | 65.107                        | 130.668            | -92.753   | 6.031     |
| B2 (20x30)                | -57.429                       | 28.7145                       | 29.125             | -57.29    | -5.175    |
| B3 (15x25)                | -20.325                       | 10.1625                       | 10.944             | -28.752   | -60.94    |
| Sloof (20x25)             | -22.506                       | 11.253                        | 10.934             | -29.832   | -4.144    |
| Kolom (35x35)             | -42.131                       | -12.353                       | -465.612           |           |           |

### 3.2. Analysis of Bending Resistant Reinforcing Steel

#### 3.2.1. Condition 1

Right interior column, the negative moment of support, wobble to the right \( M_u = -85,5012 \) Kn.m. Reinforcement steel required for bending as an initial trial, use D19: \( d = 400-(40+10+19+20) = 311 \) mm. Initial assumptions: \( j = 0,85 \) (coefficient of moment arm); \( \Phi = 0,8 \) (moment reduction factor), and \( \beta_1 = 0,85 \). Further analysis of the diameter of the reinforcement using Equation 5 to determine the height of the beam is as follows:

\[
A_s = \frac{M_u}{\Phi \beta_1 d} = \frac{85,5012 \times 10^6}{0,8 \times 0,85 \times 311} = 1010,75 \text{ mm}^2
\]  

(5)

Since the reinforcement used is D19 with a total of 4 bars, and \( A_s = 1134 \text{ mm}^2 \), \( d_{\text{new}} = 311 \text{ mm} \), the actual equivalent compression stress block height is calculated in Equation 6 below:

\[
\sigma = \frac{A_s F_y}{0,85 f'_{\text{cb}}} = \frac{1134,400}{0,85 \times 25,300} = 71,28 \text{ mm}
\]  

(6)

Based on the results of calculations using Equation 5 and Equation 6, then it is necessary to check the actual nominal moment using Equation 7 as follows:
\[
\omega M_e = \alpha As,F_y (d - \alpha / 2) = 0.8 \times 1134 \times 400 \times (311 - 71.28/2).10^6
\]  
(7)

Actual nominal moment control \(\omega M_e = 99,9 \text{ K}-\text{m} > -85,5012 \text{ K}-\text{m} \), declared Ok

Next, check As using Equation 8 provided that As cannot be less than the minimum As. The minimum axle is calculated using Equation 9. Control As is calculated as follows:

\[
\frac{1.4}{4f_y} \times \frac{bw.d}{300.311} = 326,55 \text{ mm}^2
\]  
(8)

\[
As_{\text{min}} = \sqrt{\frac{7\pi}{4f_y}} \times \frac{bw.d}{300.311} = 291,56 \text{ mm}^2
\]  
(9)

Based on the analysis, it is known that As has a value of 326.55 mm2 > As a minimum is 291.56 mm2, thus As is declared Ok, because the minimum reinforcement requirements are met.

The next step is to check the reinforcement ratio with 2 stages, stage 1 uses Equation 10, and stage 2 uses Equation 11. The reinforcement check is calculated as follows:

\[
\rho = \frac{As}{bw.d} = \frac{1134}{300.311} = 0,01217
\]  
(10)

\[
\rho_b = \beta \frac{0.85f_y}{f_y} (600 / 600 + f_y) = 0,85 \times \frac{0.85 \times 400}{400} = 0,02709
\]  
(11)

The maximum reinforcement limit based on SNI Concrete Article 21.5.2.1 is 0.025. The results of the analysis are declared Ok, because \(\rho < 0,75 \rho_b \) and \(\rho < 0,025 \) states that the maximum reinforcement requirements are met.

The last step of condition 1 is to check whether the tension-controlled cross section meets the requirements. This control can be calculated using Equation 12 as follows:

\[
\frac{\omega - 71.28}{340.5} = 0.20933 < 0,8 \times \frac{atcl}{dt} = 0.375. \beta_1 = 0.375 \times 0.85 = 0.31875
\]  
(12)

The design of the under reinforced reinforcement is based on the results of the analysis using Equation 8, because 0.20933 < 0.31875 so that it is declared OK to meet the requirements.

3.2.2. Condition 2

Left interior column, a negative moment of support, swaying to the left, the need for detailing the cross-section is the same as condition 1, that is, it takes D19 number of reinforcements 4 to carry Mu = -85,5012 K-nm.

3.2.3. Condition 3

Right exterior column, positive moment of support, left sway with moment Mu is 42.7506 K-nm. Reinforcement steel required for bending as an initial trial, use D19: d = 400-(40+10+19/2) = 340.5 mm. Initial assumptions: \(j = 0.85 \) (coefficient of moment arm); \(\Omega = 0.8 \) (moment reduction factor) and \(\beta_b = 0.85 \). Furthermore, based on the diameter of the reinforcement in condition 3, it is necessary to determine the height of the beam, this can be calculated using Equation 5 and Equation 6 with the following analysis:

\[
As = \frac{m_u}{94f_yd} = \frac{42,7506.10^6}{0.8400.0.85.340.5} = 434,437 \text{ mm}^2
\]  
(5)

Because the reinforcement used is D16 with a total of 3, and As = 603 mm2, d-new = 342 mm. The actual equivalent compressive stress block height is calculated using Equation 6 as follows:

\[
\sigma = \frac{As,F_y}{0.85f_{cb}} = \frac{603.400}{0.85.25.300} = 37,835 \text{ mm}
\]  
(6)

Check the actual nominal moment using Equation 7 as follows:

\[
\omega M_e = \omega As,F_y (d - \sigma / 2) = 0.8 \times 603 \times 400 (342 - 37,835/2) .10^6
\]  
(7)

\[
\omega M_e = 62,34 \text{ K}-\text{m} \cdot \text{Ok}
\]
Check \( A_s \) using Equation 8 provided that it cannot be less than the minimum \( A_s \) in Equation 9. This analysis is calculated as follows:

\[
1.4/4f_y \cdot b \cdot d = 1.4/400 \cdot 300.342 = 359.1 \text{ mm}^2
\]  

(8)

\[
A_{s, \text{min}} = \frac{\sqrt{f_c}}{4f_y} \cdot b \cdot d = \frac{\sqrt{25}}{4 \cdot 400} \cdot 300.342 = 320.625 \text{ mm}^2
\]  

(9)

The minimum reinforcement requirements are met, then it is declared OK.

Check the reinforcement ratio for condition 3 through 2 stages, stage 1 uses Equation 10 and continues using Equation 11, as calculated as follows:

\[
\rho = \frac{A_s}{b \cdot d} = \frac{603}{300.342} = 0.00587
\]  

(10)

\[
\rho_b = \beta_1 \frac{0.85f'_c}{f_y} \leq (600 / 600 + f_y) = 0.85 \frac{0.85f'_c}{400} (600 / 600 + 400)
\]  

(11)

\[
\rho_b = 0.02709
\]

The maximum reinforcement limit based on SNI Concrete Article 21.5.2.1 is 0.025 [20]. The results of the analysis are declared OK because \( \rho < 0.025 \) states that the maximum reinforcement requirements are met.

The last step of condition 3 is to check whether the tension-controlled cross section meets the requirements. This control can be calculated using Equation 12 as follows:

\[
\alpha = \frac{71.28}{340.5} = 0.20933 < \frac{\alpha_{\text{cl}}}{} = 0.375, \beta_1 = 0.375 \times 0.85 = 0.31875
\]  

(12)

The design of under reinforced reinforcement is based on the results of the analysis using Equation 8 because 0.20933 < 0.31875 so it is declared OK, because it meets the requirements.

3.2.4. Condition 4

The left exterior column, positive moment of support, sway to the left, the need for detailing the cross-section is the same as for condition 3, that is, it takes D16 = 3 to carry \( M_u = 42.7506 \text{ Kn-m} \).

3.2.5. Condition 5

Condition 5 is mid-span, positive moment, sway to the right and left. Based on SNI 03-2847-2013 Article 21.5.2.2 requires both the negative flexural strength and positive flexural strength at each section along the span should not be less than the largest flexural strength provided on the two faces of the column [21], while that provided Mn = 99.9 Kn-m. moment Mu is 30.2581 kN-m > \( 1/4 \) \( \alpha M_n \) = 24.9975 kN-m, determined using 30.2581 kN-m. Reinforcement steel required for bending, as an initial trial, use D19: \( d = 400-(40+10+19/2) = 340.5 \text{ mm} \). Initial assumptions: \( j = 0.85 \) (coefficient of moment arm); \( \alpha = 0.8 \) (moment reduction factor) and \( \beta_i = 0.85 \), then the reinforcing requirement control is calculated using Equation 5.

\[
A_s = \frac{m_u}{\phi_f \cdot j \cdot d} = \frac{302581.10^6}{0.8 \cdot 400 \cdot 0.85 \cdot 340.5} = 326.704 \text{ mm}^2
\]  

(5)

It is enough to use 3 D16, \( A_s = 603 \text{ mm}^2 \), \( d_{\text{new}} = 342 \text{ mm} \). The actual equivalent compressive stress block height is calculated in Equation 6:

\[
\alpha = \frac{A_s f_y}{0.85 f'_c b} = \frac{603 \cdot 400}{0.85 \cdot 25300} = 37.835 \text{ mm}
\]  

(6)

Check the actual nominal moment using Equation 7.

\[
\omega M_z = \omega A_s f_y (d - \alpha / 2) = 0.8 \cdot 603 \cdot 400 (342 - 37.835/2).10^6
\]  

\[
\omega M_z = 62.34 \text{ Kn-m} \text{ declared Ok.}
\]

Check \( A_s \) minimum using Equation 8 and Equation 9.

\[
A_{s, \text{min}} = \frac{\sqrt{f_c}}{4f_y} \cdot b \cdot d = \frac{\sqrt{25}}{4 \cdot 400} \cdot 300 \cdot 342 = 320.625 \text{ mm}^2
\]  

(8)
The provision that $A_{s_{\text{min}}}$ must not be less than the result of the analysis using Equation 9 as calculated as follows:

\[
\frac{1.4}{4f_y} \cdot b \cdot w \cdot d = \frac{1.4}{400} \cdot 300 \cdot 342 = 359.1 \text{ mm}^2
\]  

(9)
Minimum reinforcement requirements are met and can be declared OK.

Check the reinforcement ratio using Equation 10 and continue using Equation 11.

\[
\rho = \frac{A_s}{b w d} = \frac{603}{300 \cdot 342} = 0.00587 \quad (10)
\]
\[
\rho_b = \beta_1 \frac{1.05 f_c}{f_y} = \beta_1 \left(\frac{600}{600 + f_y} + \frac{25}{400}\right) = 0.02709 \quad (11)
\]

Because 0.75 \(\rho_b\) then is 0.75 x 0.02709 = 0.0203

The results of the analysis stated Ok because \(\rho < 0.75 \rho_b \) and \(\rho < 0.025\) so that the maximum reinforcement requirements were met. Next check whether the tension-controlled cross-section uses Equation 12.

\[
\frac{a}{\frac{37.835}{342}} = 0.1106 < \frac{\alpha_{\text{cl}}}{\frac{357.1}{2}} = 0.375, \beta_1 = 0.375 x 0.85 = 0.31875 \quad (12)
\]

Reinforcement design based on analysis is declared OK.

### 3.3 Minimum Capacity of Positive Moments and Negative Moments.

Based on SNI 03-2847-2013 Articles 21.5.2.1 and 21.5.2.2 requires at least two upper reinforcement bars and two lower reinforcement bars to be installed continuously, and a minimum positive and negative moment capacity in the distribution of cross-sections along the beam span, SRPMK shall not be less than ¼ times the maximum moment capacity provided at both faces of the beam column [20].

The largest negative-positive moment strength in the span = 85.5012 kN-m ¼ the largest negative-positive moment strength = 21.3753 kN-m. The minimum capacity of positive and negative moments requires analysis of flexural reinforcement requirements, checking actual moments, checking \(A_s\)-min, checking reinforcement ratios, and checking tension-controlled cross-sections. The results of this analysis are presented in Table 3.

#### Table 3. Structural Reinforcement Steel Analysis

| Analysis                          | Equation Formula                   | Calculation                  | Count result | Analysis status |
|-----------------------------------|------------------------------------|------------------------------|--------------|-----------------|
| Flexible reinforcing steel        | \(A_s = \frac{m_t}{\phi \beta j d}\) | (5)                          | 21,3753.10^6 | 0.8400.0.853405| 230,794 mm^2    |
| Actual equivalent compressive stress beam height | \(\alpha = \frac{A_s f_y}{0.85 f_c b}\) | (6)                          | 402,400      | 0.8525,300      | 25,223mm        |
| Cek momen nominal aktual          | \(\alpha_{\text{Mn}} = \alpha_{A_s f_y (d - \alpha / 2)}\) | (7)                          | 0.8402,400.342 - 25,223 /2.106 | 42.37 K - m  | OK              |
| Check As minimum                  | \(\text{As}_{\text{min}} = \frac{f_y}{\sqrt{\beta}} \frac{\beta f_y}{400}\) | (8)                          | > 300,342     | > 359,1mm^2     | OK              |
| Check reinforcement ratio         | \(\rho = \frac{A_s}{b w d} \frac{0.85 f_c}{f_y}\) | (10)                         | 402          | 213753         | OK              |
| Check tension-controlled section  | \(\frac{a}{\frac{37.835}{342}} = \frac{\alpha_{\text{cl}}}{357.1}\) | (12)                         | \(\beta_1 = 0.375 x 0.85\) | 0.31875         | OK              |

#### 3.3.1 Calculate Probable Moment Capacities (Mpr)

Analysis of Probable Moment Capacities referring to SNI 03-2847-2013 Article 21.5.4.1 implies that the design shear due to the earthquake in the beam is calculated by assuming plastic hinges are formed at the ends of the beam with the beam flexural reinforcement stress reaching 1.25 \(f_y\) and the flexural strength reduction factor is \(\beta = 1 [20]\).

- The moment capacity of the ends of the beam when the structure sways to the right. The review in Condition 1 is calculated using Equation 14 and Equation 15 as follows:
\[ a_{pr,1} = \frac{1.25A_s f_y}{0.85 f'c_h} = \frac{1.25 \times 1136 \times 400}{0.85 \times 25.30} = 80.188 \text{ mm} \] (14)

\[ M_{pr,1} = 1.25A_s f_y \left( d - \frac{a_{pr,1}}{2} \right) = 1.25 \times 1136 \times 400 \times (340.5 - (80.188/2)) \times 10^{-6} \] (15)

\[ M_{pr,1} = 170.63 \text{ mm} \]

The review in Condition 3 is calculated using Equation 14 and Equation 15 as follows:

\[ a_{pr,3} = \frac{1.25A_s f_y}{0.85 f'c_h} = \frac{1.25 \times 603 \times 400}{0.85 \times 25.30} = 47,294 \text{ mm} \] (14)

\[ M_{pr,3} = 1.25A_s f_y \left( d - \frac{a_{pr,3}}{2} \right) = 1.25 \times 603 \times 400 \times (340.5 - (47,294/2)) \times 10^{-6} \] (15)

\[ M_{pr,3} = 95,53 \text{ mm} \]

The moment capacity of the ends of the beam when the structure sways to the left. Overview Condition 2 is calculated using Equation 14 and Equation 15 as follows:

\[ a_{pr,2} = \frac{1.25A_s f_y}{0.85 f'c_h} = \frac{1.25 \times 1136 \times 400}{0.85 \times 25.30} = 80.188 \text{ mm} \] (14)

\[ M_{pr,2} = 1.25A_s f_y \left( d - \frac{a_{pr,2}}{2} \right) = 1.25 \times 1136 \times 400 \times (340.5 - (80.188/2)) \times 10^{-6} \] (15)

\[ M_{pr,2} = 170.63 \text{ mm} \]

Overview of Condition 4 is calculated using Equation 14 and Equation 15 as follows:

\[ a_{pr,4} = \frac{1.25A_s f_y}{0.85 f'c_h} = \frac{1.25 \times 603 \times 400}{0.85 \times 25.30} = 47,294 \text{ mm} \] (14)

\[ M_{pr,4} = 1.25A_s f_y \left( d - \frac{a_{pr,4}}{2} \right) = 1.25 \times 603 \times 400 \times (340.5 - (47,294/2)) \times 10^{-6} \] (15)

\[ M_{pr,4} = 95,53 \text{ mm} \]

### 3.3.2. Gravity Shear Force

The shear reaction at the right and left ends of the beam due to the gravitational force acting on the structure is calculated using Equation 16 and Equation 17 as follows:

\[ W_s = 1.2DL + 1.0LL = 1.2 \times 20.1 + 1.0 \times 2.5 = 26.62 \text{ kN-m} \] (16)

\[ V_g = \frac{WuL}{2} = \frac{26.62 \times 4.6}{2} = 54.602 \text{ kN} \] (17)

Based on the analysis, it is known that the shear force at the right and left ends of the beam due to gravity is 54.602 kN, then it is necessary to review the direction of the upward and downward shearing force. The structure swaying to the right with gravity is analyzed using Equation 18.

\[ V_{sway_{ka}} = \frac{M_{pr,1} + M_{pr,3}}{ln} = \frac{170.63 + 95.53}{4.6} = 57.86 \text{ kN} \] (18)

Based on the analysis of Equation 18, it is known that the total shear reaction at the right end of the beam is 54.602 kN, then the direction of the downward shearing force can be calculated from 54.602 to 57.86, then the downward shearing force is 3.25, while the direction of the upward shearing force is 54.602 + 57.86 so that the upward shearing force is 112.462 kN.

In the condition of the structure swaying to the left, the gravitational force can be determined by analysis using Equation 18 as follows:

\[ V_{sway_{ka}} = \frac{M_{pr,1} + M_{pr,3}}{ln} = \frac{170.63 + 95.53}{4.6} = 57.86 \text{ kN} \] (18)

It is known that the total shear reaction at the left end of the beam is 54.602, so the direction of the downward shearing force can be calculated from 54.602 to 57.86, then the downward shearing force is 3.25. While the direction of the upward shearing force is 54.602 + 57.86 so that the upward shearing force is 112.462 kN.
3.3.3. Stirrup Reinforcement For Shear Style

Based on the provisions of SNI 03-2847-2002 Article 21.5.4.2 that the contribution of concrete in resisting the shear force, namely \( V_c \), must be taken = 0 in the shear design in the plastic hinge area [22], if:

- The shear force \( V_{sway} \) due to plastic hinges at the ends of the beam exceeds \( \frac{1}{2} \) (or more) of the maximum required shear strength, \( V_u \), throughout the span; and
- Factored axial compression forces, including those due to earthquake loading, are less than \( Agf_c'/20 \).

Before determining the stirrup reinforcement, it is necessary to know in advance the shear forces in front of the interior and exterior columns. The identification of these styles is presented in Table 4.

| Earthquake vibration direction | \( V_{sway} \) (kN) | Exterior sup. reaction | \( \frac{1}{2} Vu(kN) \) | Interior sup. reaction | \( \frac{1}{2} Vu(kN) \) |
|-------------------------------|---------------------|-----------------------|-------------------|----------------------|-------------------|
| Right                         | 57.86               | -3.258                | 1.629             | 112.462              | 56.231            |
| Left                          | 57.86               | 112.462               | 56.231            | -3.258               | 1.629             |

Based on the results of structural analysis, the factored axial compressive force due to earthquake and gravity forces is 19,143 kN < \( Agf_c' = (300 \times 400 \times 25 \text{ N/mm}^2) = 150 \text{ kN} \). The condition of \( V_{sway} > \frac{1}{2} Vu \) only occurs in front of the exterior column due to sway to the left (while due to sway to the right, \( V_{sway} \) still exceeds \( \frac{1}{2} Vu \)). The factored axial compressive force due to earthquake and gravity < \( Agf_c'/20 \), then the shear reinforcement design is carried out without taking into account the contribution of the concrete \( V_c = 0 \) along the plastic hinge zone at each column face.

3.3.3.1. Maximum Sliding Force Control on Exterior Face

Furthermore, it is necessary to calculate the maximum shear strength at the exterior and interior column faces. Analysis of the maximum shear strength on the exterior face is calculated using Equation 19. It is known that the maximum shear force at the face of the exterior column is, \( V_u = 112.462 \text{ kN} \), then the analysis of the maximum shear force refers to SNI 03-2847-2013 Article 11.4.7.9 [23], calculated as follows:

\[
V_s = \frac{V_u}{b} - V_c = \frac{112.462}{0.75} - 0 = 149.94
\]

Maximum \( V_s = V_{s,max} = 2 \sqrt{\frac{f_{ck}}{3}} b_s d = 2 \sqrt{\frac{226}{3}} \times 300 \times 340 \times 10^{-3} = 340 \text{ kN} \) (20)

Based on the analysis using Equation 19 and Equation 20, it is known that \( V_s = 149.94 \text{ kN} < 340 \text{ kN} \), then the maximum \( V_s \) requirement is fulfilled and can be declared capable of withstanding the maximum shear force (OK).

The next step is to control the diameter of the stirrup reinforcement, it is known that the diameter of the stirrup reinforcement is D12 with 2 feet \((Av = 226 \text{ mm}^2)\), then this control can be done using Equation 21 and continued using Equation 22.

\[
s = \frac{V_s f_y d}{226.400.340.5} = 205 \text{ mm (use 200 mm spacing)}
\]

\[
V_s = \frac{Av f_y d}{s} = \frac{226.400.340.5}{200.1000} = 153.906 \text{ kN}
\]

By using D12 stirrup reinforcement with 2 legs \((Av = 226 \text{ mm}^2)\), based on the results of the analysis using Equation 21 and Equation 22, it is known to have a strength of 153.906 > 149.94 then the stirrup reinforcement is declared OK.

3.3.3.1. Maximum Sliding Force Control on Interior Face

It is known that the maximum shear force on the interior face is \( V_u = 112.462 \text{ kN} \). The value of \( V_s \) in the interior column = exterior column, then 2 feet D12 stirrups are needed with a spacing of 200 mm. The maximum shear force \( V_s \) at the end of the plastic hinge is \( 2h = 2 \times 2400 = 800 \text{ mm} \) from the face of the column is 112.462 \(-(0.8 \times 26.62 \text{ kN-m}) = 91.266 \text{ kN} \). In this zone, the contribution of \( V_s \) can be calculated using Equation 23 followed by Equation 22 as follows:
\[ V_c = \frac{\sqrt{f_c}}{6} b_n d = \frac{\sqrt{300 \times 340.5 \times 300}}{6 \times 1000} = 85.125 \]  
(23)

Then \[ V_s = \frac{91.266}{0.75} - 85.125 = 36.563 \]  
(22)

By using 2-foot stirrups with a diameter of 12 mm, \( A_v = 226 \text{ mm}^2 \), the spacing of the reinforcing stirrups can be determined by analysis using Equation 21 and Equation 22 as follows:

\[ S = \frac{A_v f_y d}{V_s} = \frac{226 \times 400 \times 340.5}{36.563 \times 1000} = 218 \text{ mm}, \text{(200 mm spacing is used)} \]  
(21)

\[ V_s = \frac{A_v f_y d}{s} = \frac{226 \times 400 \times 340.5}{200 \times 1000} = 153.91 \text{ kN} \text{(200 mm spacing is used)} \]  
(22)

Based on the provisions of SNI Article 21.5.3.1, after analysis it is necessary to hoops (closed stirrups) along with a distance of \( 2h \) from the side (face) of the nearest column \( 2h = 2 \times 400 = 800 \text{ mm} \) [24]. The first hoop is installed at a distance of 50 mm from the nearest column face, and the next one is installed with the smallest spacing between \( d/4 = 340.5/4, d/4 = 85.125, 6 \times \text{reinforcement, smallest longitudinal} = 6 \times 16 = 96, \text{and 150 mm} \) . Thus, 2 foot D12 closed stirrups are used which are installed with a spacing of 85 mm.

### 3.3.3.2. Cut-off points

In the negative reinforcement in front of the interior column, the number of top reinforcement installed is 4 pieces of D19, therefore 2 pieces of reinforcement will be cut-off, so \( A_{s \text{- remaining}} = 567 \text{ mm}^2 \). The design negative flexural strength with this reinforcement configuration can be analyzed using Equation 6 and continued using Equation 7, as in the following analysis:

\[ d = \frac{A_s F_y}{0.85 f_y'c_h} = \frac{567 \times 400}{0.85 \times 25 \times 300} = 35.58 \text{ mm} \]  
(6)

\[ \alpha M_r = \alpha A_s F_y (d - a / 2) = 0.8 \times 567 \times 400 (340.5 - 35.58/2).10^{-6} \]  
(7)

\[ \alpha M_r = 58.55 \text{ kN} \cdot \text{m} \]

Based on the provisions of SNI 03-2847-2013 Article 12.10.3 and Article 12.10.4 requires [25]; Reinforcement is extended beyond the point where it is no longer required to resist bending, to the extent that the effective member height, \( d \), is not less than 12db, except in the region of simple beam supports and the free-end region of the cantilever. Continuous reinforcement shall have a long embedding length, not less than the extension length \( 1d \) measured from the location where the flexural reinforcement is cut. Based on this, the distribution length of the D19 reinforcement is as calculated in Equation 24.

\[ l_{d19} = \frac{F_y \psi_{tp} \psi_e}{21 A_f f_c} d_b = \frac{400 \times 1.3 \times 1}{2.1 \times 1 \times \sqrt{25}} \times 19 = 940.95 \text{ mm} \]  
(24)

The results of the analysis of the length of distribution of reinforcement 2 D19 must be planted along 1000 mm so that the number of upper reinforcement installed is 4 pieces, namely D19, and 2 pieces of reinforcement will be cut-off, so \( A_{s \text{- remaining}} = 567 \text{ mm}^2 \). Because the value of the installed reinforcement is the same, the distribution length of 2 D19 reinforcement must be planted with a length of 1000 mm.

Based on the analysis that has been exemplified in the previous description, it can be seen that the moments that occur in the structure, if the building is on hard soil, medium soil, and soft soil as in Table 5 below.
Table 5. Moment and Structural Reinforcing Steel

| Hard ground conditions | Medium soil conditions | Soft soil conditions |
|------------------------|------------------------|---------------------|
| Moment description     | Reinforcement Steel Need | Moment description     | Reinforcement Steel Need | Moment description     | Reinforcement Steel Need |
| Mu top support beam: 85.5012 kN | Reinforcement steel 4 D19 | Mu top support beam: 92.0741 kN | Reinforcement steel 4 D19 | Mu top support beam: 107.842 kN | Reinforcement steel 5 D19 |
| Mu of lower support beam: 42.7506 kN | Reinforcement steel 3 D16 | Mu of lower support beam: 46.03705 kN | Reinforcement steel 3 D16 | Mu of lower support beam: 53.921 kN | Reinforcement steel 4 D16 |
| Mu mid-span beam: 30.2581 kN | Reinforcement steel 3 D16 | Mu mid-span beam: 59.4276 kN | Reinforcement steel 3 D16+ 1 D13 | Mu mid-span beam: 63.4546 kN | Reinforcement steel 4 D16 |
| Plastic hinges in beams occur at 800mm from each end of the beam | hoops are installed with a spacing of 85 mm | Plastic hinges in beams occur at 800mm from each end of the beam | mounted hoops with a spacing of 85mm | Plastic hinges in beams occur at 800mm from each end of the beam | mounted hoops with a spacing of 85mm |
| K1 column reinforcement 500 x 500 mm | Reinforcement steel 12 D22 | K1 column reinforcement 500 x 500 mm | Reinforcement steel 12 D22 | K1 column reinforcement 500 x 500 mm | Reinforcement steel 12 D22 |
| Reinforcement K2 150 x 250 mm | Reinforcement steel 6 D16 | Reinforcement K2 150 x 250 mm | Reinforcement steel 6 D16 | Reinforcement K2 150 x 250 mm | Reinforcement steel 6 D16 |
| The plastic hinge of the column occurs at 600mm from the face of the support | hoops are installed with 125mm spacing | The plastic hinge of the column occurs at 600mm from the face of the support | hoops are installed with 125mm spacing | The plastic hinge of the column occurs at 600mm | hoops are installed with 125mm spacing |

4. Conclusion

From the results of this study, it can be concluded that, in planning the Earthquake Resistant Truss Structure, the most important thing is the effective placement of reinforcing steel on the three elements, namely columns, beams, and joints in each plastic joint that arises. The response of the structure on a beam of 300 mm x 400 mm produced by gravity and earthquake loads on hard soil conditions of $Mu = -85.5012$ kN (right and left top supports) used 4D19; $Mu = 42.7506$ kN (right and left bottom supports) used 3D16, and $Mu = 30.2581$ kN (at the middle of the span) used 3D16. The response of the structure on a 300 mm x 400 mm beam produced by gravity and earthquake loads on moderate soil conditions is $Mu = -92.0741$ kN (right and left top supports) used 4D19; $Mu = 46.03705$ kN (right and left lower supports) used 3D16, and $Mu = 59.4276$ kN (at the middle of the span) used 3D16 + 1D13. The response of the structure on a 300 mm x 400 mm beam produced by gravity and earthquake loads on soft soil conditions of $Mu = -107.842$ kN (right and left top supports) is used 5D19; $Mu = 53.921$ kN (right and left lower supports) used 4D16, and $Mu = 63.4546$ kN (at the middle of the span) reinforcement formation used 4 D16. The plastic hinges that occur in the beam, are 800mm long from the face of the support and must be installed with steel hoops, with a maximum spacing of 85mm. There is no significant difference in the axial forces that occur in the main column due to the combination of the three types of soil. On hard soil = 337,949 kN, medium soil = 339,785 kN, soft soil = 342,954 kN, so that the main column reinforcement in all three uses 12D22. The plastic joints that occur in the column are 600 mm long on each face of the column and must be installed with hoops with a maximum spacing of 125 mm. The results of this study can contribute to determining the reinforcing steel of the building structure to be able to withstand earthquake loads. The diameter and amount of structural reinforcing steel, depending on the type of soil on which a building is built.

Declarations

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