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-2847-2013 and 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
Stirrup 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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of earthquake response [9]. When viewed from the various types of soil that exist in general consist of hard soil, medium soil, and soft soil [10].

According to Gideon H. Kusuma, the concept of an earthquake-resistant structure is to ensure that a structure is not damaged by small or medium earthquakes, but when a strong earthquake occurs, which rarely occurs, the structure is capable of ductile behavior by dispersing earthquake energy and at the same time limiting the earthquake load that enters the body structure [11]. To apply the concept so that the building meets the planned criteria, it is necessary to have a Capacity Design concept, namely the structural elements of the building are not made equally strong against the planned internal forces, but there are structural elements that are made weak compared to others by using the concept of (Strong Column and Weak Beam) [12]. The Moment Resistant Frame System is one of the systems that is often used in terms of designing earthquake-resistant building structures [13]. In its implementation, the use of the Moment Bearing Frame System must comply with the requirements that must be met in the application of the system [14]. So that the resulting structure can withstand the designed earthquake.

This study aims to determine the dimensions and reinforcement of the structure with a moment-bearing frame system in a building located in an earthquake-prone area so that the structure is able to withstand earthquakes. The soil conditions reviewed in this study consisted of three types of soil, namely hard soil, medium soil, and soft soil. This research is expected to contribute to society in general in determining the dimensions of the structure, reinforcing steel, and stirrup reinforcement, according to the type of soil to be erected a building.

2. Method

2.1. Research Location and Time

In this study, the authors took the object of research as a case study on the Nusa Putra Islamic Boarding School building, which has 3 floors and is located on Jalan Nasional III, Cibolang Kaler, Sukabumi, West Java, where the building in the case study of this research is still in the planning stage.

2.2. Tools and Materials

The tool used in this research is to use a computer software tool that functions as a structural analysis to get a reaction on the planned structure in the form of the moment, shear, and axial. The materials used in this research are in the form of structural and architectural plans. As well as supporting data in planning in the form of loading data, namely dead load, live load, wind load, and earthquake load in three conditions, namely hard soil, medium soil, soft soil.

2.3. Prosedur Penelitian Dan Analisis Data

Analysis of the data used to obtain the response of the planned structure is with the help of a computer structure application using a combination of loading under SNI-1726-2012 [15], namely as follows:

- 1,4 DL; (1)
- 1,2 DL + 1,6 LL; (2)
- 1,2 DL + 1,0 LL + 1,0 EX + 0,3 EY; and (3)
- 1,2 DL + 1,0 LL + 0,3 EX + 1,0 EY. (4)

In calculating the search for the forces acting on the structure, the author uses the help of computer software, to obtain earthquake load responses in the three soil conditions that occur, the author uses the seismic analysis method of Dynamic Response Spectrum. The soil conditions under consideration are:

- Hard soil type
- Medium middle type; and
- Soft soil type

After analyzing the moments on structural elements that have the same dimensions, the largest moment is chosen, while the smaller moment is considered to have been represented. Meanwhile, the calculation of reinforcement is done manually by taking into account the rules contained in the Special

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Moment Bearing Frame System (SRPMK). Briefly, the stages of this research are as described in Figure 1.

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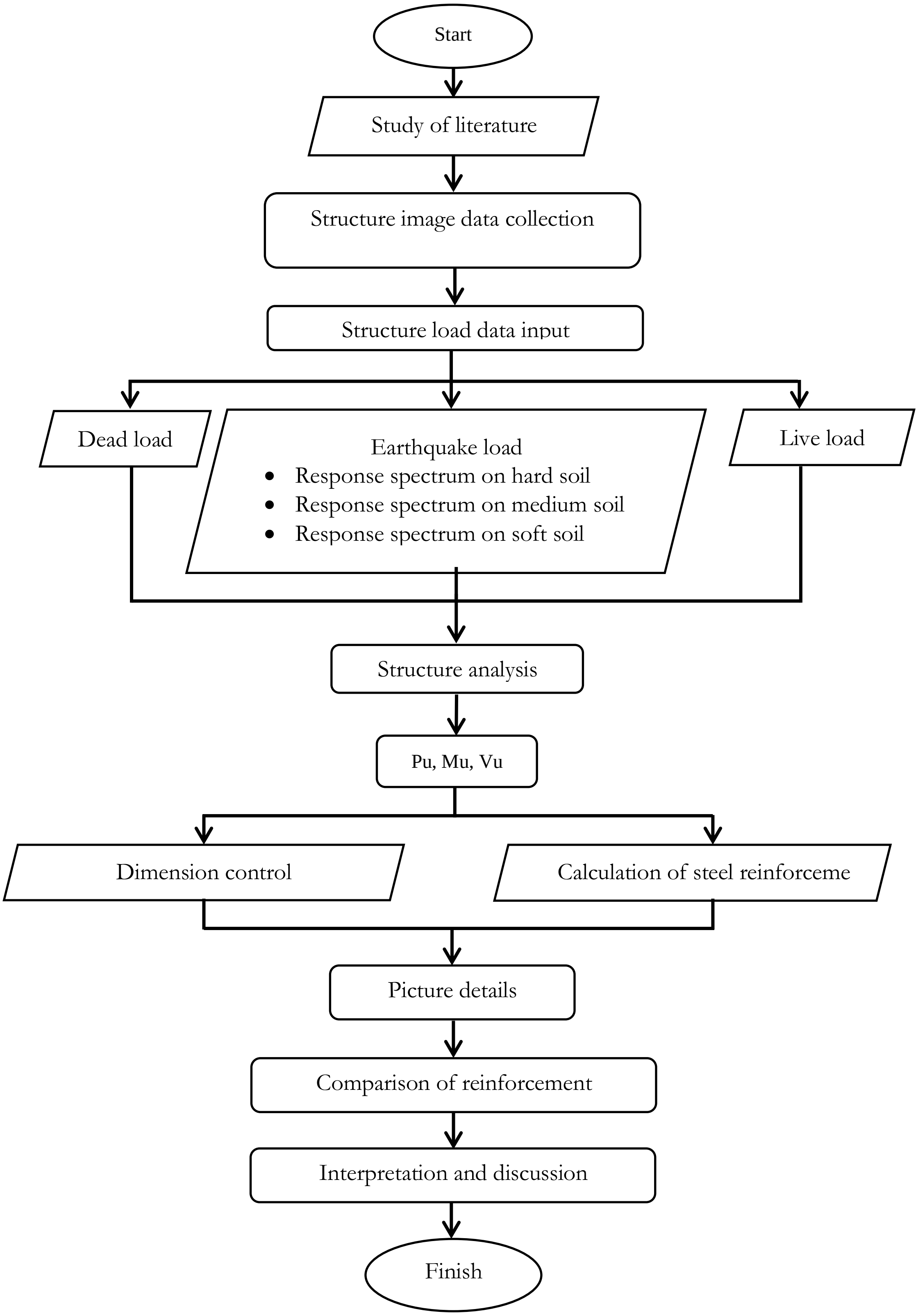

Fig. 1. Research flow

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.

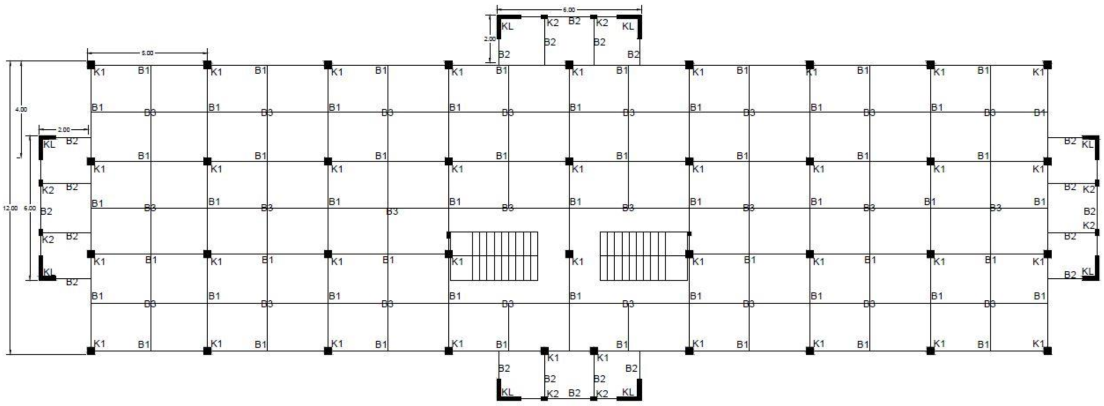

Fig. 2. Structural plans

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 and Material Specifications	
Building Function	Islamic Boarding School Building
Building area	600 m^2
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/m^2
Dead load on floor slab	1722 $kg/\ m^2$
Wind load on the roof	6,72 kg/m^2 (tekan), -13,44 kg/m^2 (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

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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.

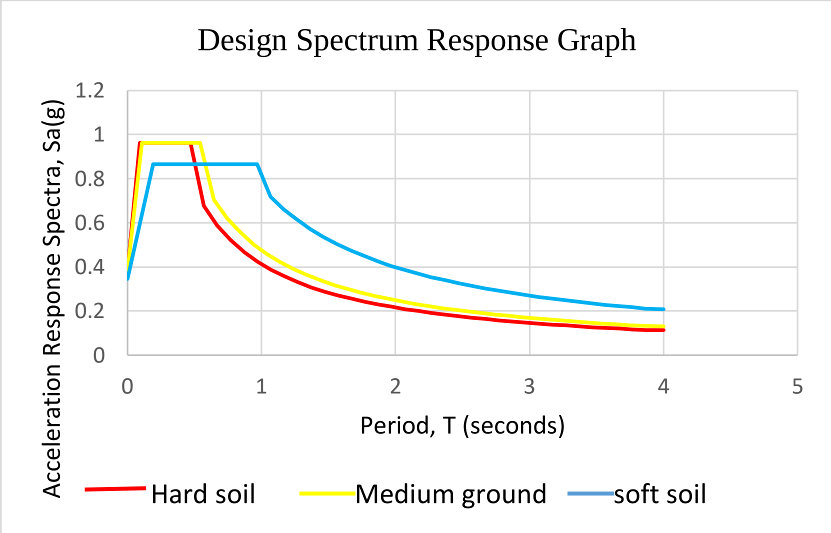

Fig. 3. Response Spectrum Design

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.

Table 2. Output Analisa Struktur Menggunakan aplikasi komputer

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 Mu= -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); Ø = 0,8 (moment reduction factor), and β_1 = 0,85. Further analysis of the diameter of the reinforcement using Equation 5 to determine the height of the beam is as follows:

$$As = \frac{m_u}{\phi f_y jd} = \frac{85{,}5012.10^6}{0{,}8.400.0{,}85.311} = 1010{,}75 \text{ mm}^2 \quad (5)$$

Since the reinforcement used is D19 with a total of 4 bars, and As = 1134 mm², d-new = 311 mm, the actual equivalent compression stress block height is calculated in Equation 6 below:

$$a = \frac{As.Fy}{0{,}85.f'c.b} = \frac{1134.400}{0{,}85.25.300} = 71{,}28 \text{ mm} \quad (6)$$

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:

$$\varnothing M_n = \varnothing As.Fy\ (d - a\ /\ 2) = 0{,}8\ .\ 1134\ .\ 400\ (311 - 71{,}28/2).10^6 \quad (7)$$

Actual nominal moment control $\varnothing M_n$ = 99,9 Kn-m > -85,5012 Kn-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:

$$\frac{1{,}4}{4fy} bw.d = \frac{1{,}4}{400}\ 300.311 = 326{,}55\ mm^2 \quad (8)$$

$$As_{-min} = \frac{\sqrt{f'c}}{4fy} bw.d = \frac{\sqrt{25}}{4.400}\ 300.311 = 291{,}56\ mm^2 \quad (9)$$

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:

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

$$\rho_b = \beta 1 \frac{0{,}85 f'c}{fy} (600\ /\ 600 + fy) = 0{,}85 \frac{0{,}85.25}{400} (600\ /\ 600 + 400) \quad (11)$$

$$\rho_b = 0{,}02709$$

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 ρ < 0,75 ρ_b and ρ < 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:

$$\frac{a}{dt} = \frac{71{,}28}{340{,}5} = 0{,}20933 < \frac{atcl}{dt} = 0{,}375.\ \beta 1 = 0{,}375 \text{ x } 0{,}85 = 0{,}31875 \quad (12)$$

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 Kn-m.

3.2.3. Condition 3

Right exterior column, positive moment of support, left sway with moment Mu is 42.7506 Kn-m. 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); Ø = 0,8 (moment reduction factor) and β_1 = 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:

$$As = \frac{m_u}{\phi f_y jd} = \frac{42{,}7506.10^6}{0{,}8.400.0{,}85.340.5} = 434{,}437\ mm^2 \quad (5)$$

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:

$$a = \frac{As.Fy}{0{,}85.f'c.b} = \frac{603.400}{0{,}85.25.300} = 37{,}835\ mm \quad (6)$$

Check the actual nominal moment using Equation 7 as follows:

$$\varnothing M_n = \varnothing As.Fy\ (d - a\ /\ 2) = 0{,}8\ .\ 603\ .\ 400\ (342 - 37{,}835/2).10^6 \quad (7)$$

$\varnothing M_n$ = 62,34 Kn-m -**Ok**

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Check As using Equation 8 provided that it cannot be less than the minimum As in Equation 9. This analysis is calculated as follows:

$$1{,}4/4fy\ bw.d = 1{,}4/400300.342 = 359{,}1\ mm^2 \tag{8}$$

$$As_{-min} = \frac{\sqrt{f'c}}{4fy}\ bw.d = \frac{\sqrt{25}}{4.400}\ 300.342 = 320{,}625\ mm^2 \tag{9}$$

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:

$$\rho = \frac{As}{bw.d} = \frac{603}{300.342} = 0.00587 \tag{10}$$

$$\rho_b = \beta 1 \frac{0{,}85 f'c}{fy}\ (600\ /\ 600 + fy) = 0.85\ \frac{0{,}85.25}{400}\ (600\ /\ 600 + 400) \tag{11}$$

$$\rho_b = 0.02709$$

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 ρ 0.00587 < 0,75 ρ_b and ρ < 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:

$$\frac{a}{dt} = \frac{71{,}28}{340{,}5} = 0.20933 < \frac{atcl}{dt} = 0.375.\ \beta 1 = 0{,}375 \text{ x } 0.85 = 0.31875 \tag{12}$$

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 Mu = 42.7506 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 > ¼ øMn = 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 mm. Initial assumptions: j = 0,85 (coefficient of moment arm); ø = 0,8 (moment reduction factor) and β_1 = 0,85, then the reinforcing requirement control is calculated using Equation 5.

$$As = \frac{m_u}{\phi fyjd} = \frac{30{,}2581.10^6}{0.8.\ 400.\ 0.85.340.5} = 326.704\ mm^2 \tag{5}$$

It is enough to use 3 D16, As = 603 mm2, d-new = 342 mm. The actual equivalent compressive stress block height is calculated in Equation 6:

$$a = \frac{As.Fy}{0.85.f'c.b} = \frac{603.400}{0.85.\ 25.300} = 37.835\ mm \tag{6}$$

Check the actual nominal moment using Equation 7.

$$øM_n = øAs.Fy\ (d - a\ /\ 2) = 0.8\ .\ 603\ .\ 400\ (342 - 37.835/2).10^{-6} \tag{7}$$

$øM_n$ = 62.34 Kn-m declared Ok.

Check As minimum using Equation 8 and Equation 9.

$$As_{-min} = \frac{\sqrt{f'c}}{4fy}\ bw.d = \frac{\sqrt{25}}{4.\ 400}\ 300\ .\ 342 = 320.625\ mm^2 \tag{8}$$

The provision that As_{-min} must not be less than the result of the analysis using Equation 9 as calculated as follows:

$$\frac{1.4}{4fy}\,bw.d = \frac{1.4}{400}300.\ 342 = 359.1\ mm^2 \qquad (9)$$

Minimum reinforcement requirements are met and can be declared OK.

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

$$\rho = \frac{As}{bw.d} = \frac{603}{300.\ 342} = 0.00587 \tag{10}$$

$$\rho_b = \beta 1\frac{0.85 f'c}{fy}\ (600\ /\ 600 + fy) = 0.85\frac{0.85.\ 25}{400}\ (600\ /\ 600 + 400) = 0{,}02709 \tag{11}$$

Because 0.75 ρ_b then = 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.

$$\frac{a}{dt} = \frac{37{,}835}{342} = 0{,}1106 < \frac{atcl}{dt} = 0{,}375.\ \beta_1 = 0{,}375 \times 0{,}85 = 0{,}31875 \tag{12}$$

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 As-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	Analys is status
Flexible reinforcing steel	$As = \frac{m_u}{\phi f_y jd}$	(5)	$\frac{21{,}3753 .10^6}{0{,}8.400.0{,}85.340.5}$	230,794 mm^2	
Actual equivalent compressive stress beam height	$a = \frac{As.Fy}{0{,}85.f'c.b}$	(6)	$\frac{402.400}{0{,}85.25.300}$	25,223mm	
Cek momen nominal aktual	øMn= øAs.Fy (d - a / 2)	(7)	08. 402. 400(342 – 25,223 /2).106	42,37 Kn-m	OK
Check As minimum	$a = \frac{As.Fy}{0{,}85.f'c.b}$ > $As\text{-}min = \frac{\sqrt{f'c}}{4fy} bw.d$	(8) (9)	$\frac{1{,}4}{400}$ 300.342 > $\frac{\sqrt{25}}{4.400}$ 300.342	359,1mm^2 > 359,1mm2	OK
Check reinforcement ratio	$\rho = \frac{As}{bw.d}$ $\rho b = \beta 1\frac{0{,}85 f'c}{fy}$ (600 / 600 + fy) Ø ρb	(10) (11) (13)	$\frac{402}{300.342}$ $0{,}85\frac{0{,}85.25}{400}$ (600 / 600 + 400) 0,75 x 0,02709	0,0203	OK
Check tension-controlled section	$\frac{a}{dt} < \frac{atcl}{dt}$	(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 fy and the flexural strength reduction factor is Ø = 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:

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$$a_{pr\_1} = \frac{1{,}25As.fy}{0{,}85f'c.b} = \frac{1{,}25.1136.400}{0{,}85.25.300} = 80.188 \text{ mm} \quad (14)$$

$$M_{pr\_1} = 1.25As.fy\ (d - (\ a_{pr\_1}\ /\ 2\ ) = 1.25 \text{ x } 1136 \text{ x } 400\ (340.5 - (80.188/2)10^{-6} \quad (15)$$

M_{pr_1} = 170.63 mm

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

$$a_{pr\_3} = \frac{1{,}25As.fy}{0{,}85f'c.b} = \frac{1{,}25.603.400}{0{,}85.25.300} = 47{,}294 \text{ mm} \quad (14)$$

$$M_{pr\_3} = 1{,}25As.fy\ (d - (\ apr\_3\ /\ 2\ ) = 1{,}25 \text{ x } 603 \text{ x } 400\ (340{,}5 - (47{,}294/2)10^{-6} \quad (15)$$

M_{pr_3} = 95,53 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:

$$a_{pr\_2} = \frac{1{,}25As.fy}{0{,}85f'c.b} = \frac{1{,}25.1136.400}{0{,}85.25.300} = 80.188 \text{ mm} \quad (14)$$

$$M_{pr\_2} = 1.25As.fy\ (d - (\ a_{pr\_1}\ /\ 2\ ) = 1.25 \text{ x } 1136 \text{ x } 400\ (340.5 - (80.188/2)10^{-6} \quad (15)$$

M_{pr_2} = 170.63 mm

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

$$a_{pr\_4} = \frac{1{,}25As.fy}{0{,}85f'c.b} = \frac{1{,}25.603.400}{0{,}85.25.300} = 47.294 \text{ mm} \quad (14)$$

$$M_{pr\_4} = 1.25As.fy\ (d - (\ a_{pr\_3}\ /\ 2\ ) = 1.25 \text{ x } 603 \text{ x } 400\ (340.5 - (47.294/2)10^{-6} \quad (15)$$

M_{pr_4}= 95.53 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:

$$W_u = 1.2DL + 1.0\ LL = 1.2\ .\ 20.1 + 1.0\ .\ 2.5 = 26.62 \text{ kN-m} \quad (16)$$

$$V_g = \frac{Wu.ln}{2} = \frac{26{,}62\ .\ 4{,}6}{2} = 54.602 \text{ kN} \quad (17)$$

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.

$$V_{sway\_ka} = \frac{Mpr\_1 + Mpr\_3}{ln} = \frac{170.63 + 95.53}{4.6} = 57.86 \text{ kN} \quad (18)$$

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, 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:

$$V_{sway\_ka} = \frac{Mpr\_1 + Mpr\_3}{ln} = \frac{170.63 + 95.53}{4.6} = 57.86 \text{ kN} \quad (18)$$

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 -3.258. 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 Vc must be taken = 0 in the shear design in the plastic hinge area [22], if:

- The sheer force V_{sway} due to plastic hinges at the ends of the beam exceeds ½ (or more) of the maximum required shear strength, Vu, throughout the span; and
- Factored axial compression forces, including those due to earthquake loading, are less than Agfc'/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.

Table 4. Sliding Style Upfront Column Interior And Exterior

Earthquake vibration direction	V_{sway}	Exterior sup. reaction		Interior sup. reaction	
	(kN)	*Vu (kN)*	*½ Vu(kN)*	*Vu(kN)*	*½ 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 < Agfc' = (300 x 400 x 25 N/mm2) = 150 kN. The condition of Vsway > ½ 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 ½ Vu). The factored axial compressive force due to earthquake and gravity < Agfc'/20, then the shear reinforcement design is carried out without taking into account the contribution of the concrete Vc = 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, Vu = 112.462 kN, then the analysis of the maximum shear force refers to SNI 03-2847-2013 Article 11.4.7.9 [23], calculated as follows:

$$\mathrm{Vs} = \frac{\mathrm{Vu}}{\emptyset} - \mathrm{Vc} = \frac{112{,}462}{0{,}75} - 0 = 149.94 \tag{19}$$

$$\text{Maximum } V_s = V_{s\text{-max}} = 2\frac{\sqrt{f'c}}{3}\, b_w d = 2\frac{\sqrt{25}}{3}\, 300 \times 340 \times 10^{-3} = 340 \text{ kN} \tag{20}$$

Based on the analysis using Equation 19 and Equation 20, it is known that V_s = 149.94 kN< 340 kN, then the maximum V_s requirement is fulfilled and can be declared capable of withstanding the maximum sheer 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 mm2), then this control can be done using Equation 21 and continued using Equation 22.

$$\mathrm{s} = \frac{\mathrm{Av.fy.d}}{\mathrm{Vs}} = \frac{226.\ 400.\ 340.5}{149.94.\ 1000} = 205 \text{ mm (use 200mm spacing)} \tag{21}$$

$$V_s = \frac{\mathrm{Av.fy.d}}{\mathrm{s}} = \frac{226.\ 400.\ 340.5}{200.\ 1000} = 153.906 \text{ kN} \tag{22}$$

By using D12 stirrup reinforcement with 2 legs (Av = 226 mm2), 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 Vu = 112.462 kN . The value of V_u in the interior column = exterior column, then 2 feet D12 stirrups are needed with a spacing of 200 mm. The maximum shear force V_u, at the end of the plastic hinge is 2h= 2x400= 800 mm from the face of the column is 112.462 -(0.8 x 26.62 kN-m) = 91.266 kN. In this zone, the contribution of V_c can be calculated using Equation 23 followed by Equation 22 as follows:

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$$V_c = \frac{\sqrt{f'c}}{6}\, b_w.d = \frac{\sqrt{25}.\ 300.\ 340.5}{6.\ 1000} = 85.125 \quad (23)$$

$$\text{Then } V_s = \frac{91.266}{0.75} - 85.125 = 36.563 \quad (22)$$

By using 2-foot stirrups with a diameter of 12 mm, Av = 226 mm, the spacing of the reinforcing stirrups can be determined by analysis using Equation 21 and Equation 22 as follows:

$$s = \frac{Av.fy.d}{Vs} = \frac{226.\ 400.\ 340.5}{36.563.\ 1000} = 218 \text{ mm, (200 mm spacing is used)} \quad (21)$$

$$V_s = \frac{Av.fy.d}{s} = \frac{226.\ 400.\ 340.5}{200.\ 1000} = 153{,}91 \text{ kN (200 mm spacing is used)} \quad (22)$$

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 x 400 = 800 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 x reinforcement, smallest longitudinal = 6 x 16 = 96, and 150 mm. Thus, 2 foot D12 closed stirrups are used which are installed with a spacing of 85 mm.

3.3.3.2. Cutt-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 $As\text{-}_{remaining}$ = 567 mm2. 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:

$$a = \frac{As.Fy}{0{,}85.f'c.b} = \frac{567.\ 400}{0.85.\ 25.\ 300} = 35.58 \text{ mm} \quad (6)$$

$$øM_n = øAs.Fy\,(d - a/2) = 0.8 \times 567 \times 400\,(340.5 - 35.58/2).10^{-6} \quad (7)$$

$$øM_n = 58.55 \text{ kN-m}$$

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.

$$l_{d\text{-}19} = \frac{Fy.\psi t\psi e}{2{,}1\lambda\sqrt{f'c}}\, d_b = \frac{400 \times 1.3 \times 1}{2.1 \times 1 \times \sqrt{25}} \times 19 = 940.95 \text{ mm} \quad (24)$$

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 $As\text{-}_{remaining}$ = 567 mm2. 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.

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