CHAPTER 2
DESIGN OF THE UPPER STRUCTURE

2.1. Preliminary Design

The planning work for The West Papua People's Assembly building especially for structural work components using steel material for the portal structure (plates, beams, and columns). The lower structure of the building uses a reinforced concrete sloof.

2.1.1. Planning Standards and Regulations

Some of the planning regulations and standards used in this work are as follows:

1. Minimum Load for Design of Buildings and Other Structures (SNI 1727:2013)
2. Earthquake Resistance Planning Standards for Building Structures (SNI 1726:2012)
3. Procedures for Planning Steel Structures for Buildings (SNI 1729:2015) P
4. Procedure for Calculation of Concrete Structures for Buildings (SNI 2847:2013).

2.1.2. Structural Material Specifications

Material Specifications that are used in this work are as follows:

1. Steel Profile. The steel profile used is BJ 37 with yield stress, fy = 240 MPa, and ultimate tensile stress, fu = 370 MPa. Modulus of Elasticity of Steel, Es = 200.000 MPa.
2. Concrete. Concrete Compression Strength after 28 Days, fc' = 30 MPa (Bottom Structure) Modulus of Elasticity of Concrete, Ec = 4700, fc' = 23500 MPa.
3. Steel Reinforcement. Reinforcement Steel with D > 12 mm used threaded steel (deform) with yield stress, fy = 240 MPa. Reinforcement Steel with D ≤ 12 mm used plain steel with a yield stress, fy = 240 MPa Modulus of Elasticity of Steel, Es = 200.000 MPa.

2.1.3. Structure Planning Method

1. Structure System. The building structure is designed using a particular moment-bearing frame system with columns and beams as moment gauges.
2. Structure Model. The structural design process is based on the internal forces in the structural elements due to the ultimate working load that modelling is done in three-dimensional models (3D Models). In structural modelling, frame elements are used to idealize beams and columns. In contrast, the floor slab is idealized as a deck.

3. Service Limit Performance. The service limit performance is evaluated based on the combination of service loads. The service limit parameters will be evaluated due to the influence of the design earthquake of the structure. The deviation between floors must be less than the allowable deviation between floors to limit the excessive melting of steel and concrete cracking, preventing non-structural damage and excessive drift that causes inconvenience to build occupants.
4. Ultimate Limit Performance. At the ultimate limit performance, a combination of ultimate loads is used to analyze the internal forces that occur in the structural elements. These internal forces are used in structural elements such as slabs, beams, columns, foundations, Etc. The ultimate limit performance will determine the structure's safety in supporting the ultimate design load acting on the structure.

2.2. Roof Structural Design

The roof is the covering that covers the top of a building and protects it from rain, snow, sunlight, wind, and temperature extremes. As a result of technical, financial, or aesthetic considerations, roofs have been constructed in a wide range of configuration flat, pitched, vaulted, domed, or in combinations. Roof consists of several parts that will calculate step by step below.

2.2.1. Design of Purlin

Purlin Design Data

Thickness: 3.2 mm

Purlin Weight (Unit Weight): 9.27 kg/m

Geometrical Moment of Inertia:

Ix: 7160000 mm^4

Iy: 840000 mm^4

Modulus of Section:

Zx: 71600 mm^3

Zy: 15800 mm^3

Main Truss Angle (θ): 20°

Purlin Spacing (a): 1.4 m

Web Spacing: 1.4 m

Ø: 0.9

Main Truss Spacing (L1): 4 m

E (Young Modulus): 200000 Mpa

Steel Quality (Fy): 240 N/mm^2

Ceiling Weight: 5.1 kg/m^2 = 0.050013915 kN/ m^2

Purlin Load Calculation

Purlin Load: $\frac{Purlin\,Weight}{100} = \frac{9.27}{100}$ = 0.0927 knm

Roof Load: $\frac{Purlin\ Spacing}{\cos\theta \times 0.25} = \frac{1.4}{\cos 0.9 \times 0.25}$ = 0.3720 knm

Ceiling Load: Purlin Spacing × 0.2 = 0.28 kNm

Dead Load (q): Purlin Load + Roof Load + Ceiling Load = 0.745 kNm

Live Load (p): = 1 kN

Purlin Moment Calculation

M2.D: $\frac{1}{8} q \times \sin\alpha \times \left(\frac{L}{3}\right)^2 : \frac{1}{8} \times 0.777 \times \sin 3\,0 \times \left(\frac{4}{3}\right)^2$ = 0.0566 kNm

M2.L: $\frac{1}{4} p \times \sin\alpha \times \frac{L}{3} : \frac{1}{4} \times 1 \times \sin 3\,0 \ \times \frac{4}{3}$ = 0.1140 kNm

M3.D: $\frac{1}{8} q \times \cos\alpha \times L^2 : \frac{1}{8} \times 0.777 \times \cos 3\,0 \ \times \ 4^2$ = 1.4004 kNm

M3.L: $\frac{1}{4} p \times \cos\alpha \times L^2 : \frac{1}{4} \times 1 \times \cos 3\,0 \times 4$ = 0.940 kNm

M2.U: $1.4 \ \times M_{2.D}: 1.4 \ \times 0.0566$ = 0.121 kNm

: $1.2 \ \times M_{2.D} \ + \ 1.6 \ \times M_{2.L}: 1.2 \ \times 0.0863 \ + 1.6 \times 0.1667$ = 0.370 kNm

Choose M2.U: Choose the bigger one: 0.370 kNm

M3.U: $1.4 \ \times M_{3.D}: 1.4 \ \times 1.4004$ = 1.961 kNm

: $1.2 \ \times M_{3.D} \ + \ 1.6 \ \times M_{3.L}: 1.2 \ \times 1.4004 + 1.6 \times 0.940$ = 3.184 kNm

Choose M3.U: Choose the bigger one:3.184 kNm

2.2.2. Tension Check at Profile C

fb = $\frac{M3.U}{ØW3} + \frac{M2.U}{ØWw}$ ≤ Fy with Ø = 0.9 (If not fulfill choose other profile)

fb = $\frac{3184}{0.9 \times 71600} + \frac{0.370}{0.9 \times 15800}$ = 67.018 MPa

67.02 ≤ 250 MPa, C Profile Canal is Safe

2.2.3. Purlin Deflection Check

$\delta 2 : \frac{5}{384} \times \frac{q cos\alpha\, L^4}{EI} + \frac{1}{48} \times \frac{p cos\alpha\, L^3}{EI}$: $\quad : \frac{5}{384} \times \frac{0.745 \cos 30\ 4^4}{(200000 \times 7140000)} + \frac{1}{48} \times \frac{1 \cos 30\ 4^3}{(200000 \times 7140000)}$

= 0.5161

$\delta 3: \frac{5}{384} \times \frac{qsin\alpha}{EI} \times \left(\frac{L}{3}\right)^4 + \frac{1}{48} \times \frac{psin\alpha}{EI} \times \left(\frac{L}{3}\right)^3 = \frac{5}{384} \times \frac{0.745 \sin 30}{(200000 \times 7140000)} \times \left(\frac{4}{3}\right)^4 + \frac{1}{48} \times \frac{1 \sin 30}{(200000 \times 7140000)} \times \left(\frac{4}{3}\right)^3 =$ 0.1001

$\delta : \sqrt{\delta 3^2 + \delta 2^2} \leq \frac{1}{240} L : \sqrt{0.1001^2 + 0.5161^2} \leq \frac{1}{240} \times 4 = 0.5257 \leq 16$ (SAFE)

2.2.4. Truss Connection (Sag Rod)

Ft,D = $n \left(\frac{L}{3 \times q \times \sin\alpha}\right)$

Ft,L = $\frac{n}{2}$ x p x sinα

Sum of Purlin (n)

Ft,D = $n \left(\frac{n}{3 \times 0.777 \times \sin(rad(30))}\right)$ = 1.359 kN

Ft,L = $\frac{n}{2}\left(1 \sin(rad(30))\right)$ = 1 kN

2.2.5. Load Combination

Ft,U = 1.4 Ft,D

Ft,U = 1.2 Ft,D + 1.6 Ft,L = 1.903 kN

Ft,U = 1.4 x 2.072 = 2.9002 kN

Ft,U = (1.2 x 2.072) + (1.6 x 1) = 2.726 kN

Choosen Ft,U = 2.726 kN

Sag Rod Width

Sag Rod Amount Needed:

$A_{sr} : \frac{Ft \times 10^3}{\phi \times Fy} : \frac{2.726 \times 10^3}{0.9 \times 250} = 12.1137$ kN

2.2.6. Main Truss Load

Truss load is a Figure for weight, and consequent pressure, on a truss. A truss is a type of structure, often made of one or more triangular pieces, commonly used in residential, commercial, or public works construction design. A common form of a truss is a standard roof truss that forms the load-bearing basis of the standard roof of a building.

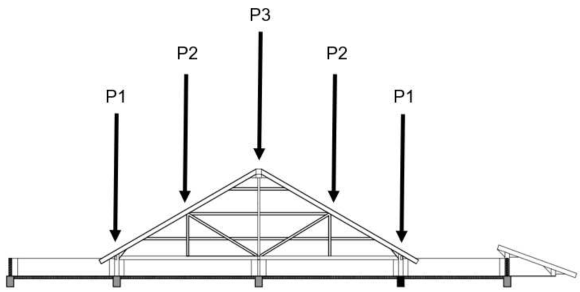

Figure 2.1. Roof Load Design

Figure 2.1 shows the design of our roof, and the location of each Main Truss Loading. We divided our Main Truss Loading into 3 parts, which are P1, P2, and P3, depends on the location of our Main Trusses. Here, it shows that the P1 Load formula will be used for the Main Truss that is located above the support. P3 Load formula will be used for the Main Truss that is located in the center, and P2 Load will be used for the Main Truss that is located in between.

P1 Load

Main Truss Self Weight $= \left(\frac{\alpha}{2}\right) \times \text{Main Truss Weight}$

Purlin Weight = L x Purlin Weight per m'

Roof Weight $= \frac{\left(\frac{\alpha}{2}\right)+b}{\cos\alpha} \times L \times \text{Roof Weight}$

Celling Weight $= \left(\left(\frac{a}{2}\right) + b\right) \times L \times \text{Celling Weight}$

Main Truss Self Weight $= \left(\frac{1.4}{2}\right) \times 0.25$ = 0.175 kN

Purlin Weight = 4x0.93 = 0.37 kN

Roof Weight $= \frac{\left(\frac{1.4}{2}\right)+0.82}{\cos(\text{RADIANS}(30))} \times 4 \times 0.404$ = 2.4099 kN

Celling Weight $= \left(\left(\frac{1.4}{2}\right) + 0.82\right) \times 4 \times 0.28$ = 1.702 kN

P1 Load = 0.25 + 0.37 + 2.8373 + 1.702 = 4.6851 kN

P2 Load

Main Truss Self Weight $= \alpha \times \text{Main Truss Weight}$

Purlin Weight = L x Purlin Weight per m'

Roof Weight $= \frac{\alpha}{\cos\alpha} \times L \times \text{Roof Weight}$

Celling Weight $= \alpha \times L \times \text{Celling Weight}$

Main Truss Self Weight	= 1.4 × 0.25	= 0.35 kN
Purlin Weight	= 4 × 0.093	= 0.37 kN
Roof Weight	$= \left(\frac{1.4+0.82}{\cos(\text{RADIANS}(30))}\right) \times 4 \times 0.404$	= 3.5197 kN
Celling Weight	= 1.4 × 4 × 0.28	= 1.568 kN
P2 Weight	= 0.35 + 0.37 + 4.144 + 1.568	= 5.8085 kN

P3 Load

Main Truss Self Weight = α × Main Truss Weight

Purlin Weight = 2 × L × Purlin Weight per m’

Roof Weight $= \frac{\alpha}{\cos\alpha} \times L \times$ Roof Weight

Celling Weight = α × L × Celling Weight

Main Truss Self Weight	= 1.4 × 0.25	= 0.35 kN
Purlin Weight	= 2 × 4 × 0.093	= 0.74 kN
Roof Weight	$= \left(\frac{1.4+0.82}{\cos(\text{RADIANS}(30))}\right) \times 4 \times 0.404$	= 4.318 kN
Celling Weight	= 1.4 × 4 × 0.28	= 1.568 kN
P3 Load	= 0.35 + 0.74 + 4.685 + 1.568	= 6.9772 kN

2.2.7. Wind Load

Wind Load is used to refer to any pressures or forces that the wind exerts on a building or structure.

W1 Load $= \frac{\frac{a}{2}+b}{\cos\ \alpha} \times Cti \times L \times Qw$

W2 Load $= \frac{a}{\cos\ \alpha} \times Cti \times L \times Qw$

W3 Load $= \frac{a}{2\cos\ \alpha} \times Cti \times L \times Qw$

W4 Load $= \frac{a}{2\cos\ \alpha} \times Csi \times L \times Qw$

W5 Load $= \frac{a}{\cos\ \alpha} \times Csi \times L \times Qw$

W6 Load $= \frac{\frac{a}{2}+b}{\cos\ \alpha} \times Csi \times L\ \times\ Qw$

W1 Load

We calculate the vertical and horizontal value of the wind load in order to input to our roof model in SAP 2000 to check the safety.

Left: $\frac{\left(\frac{1.4}{2}+0.82\right)}{\cos 34} \times 0.4 \times 4 \times 0.25$ = 0.724

Right: $\frac{\left(\frac{1.4}{2}+0.82\right)}{\cos 34} \times (-0.6) \times 4 \times 0.25$ = -1.086

W2 Load

Left: $\frac{1.4}{\cos 30} \times 0.4 \times 4 \times 0.25$ = 0.596

Right: $\frac{1.4}{\cos 30} \times (-0.6) \times 4 \times 0.25$ = -0.894

W3 Load

Left: $\frac{1}{2} \times \frac{1.4}{\cos 38} \times 0.4 \times 4 \times 0.25$ = 0.355

Right: $\frac{1}{2} \times \frac{1.4}{\cos 38} \times (-0.6) \times 4 \times 0.25$ = -0.533

W4 Load

Left: $\frac{1}{2} \times \frac{1.4}{\cos 38} \times (-0.6) \times 4 \times 0.25$ = -0.533

Right: $\frac{1}{2} \times \frac{1.4}{\cos 38} \times (0.4) \times 4 \times 0.25$ = 0.355

W5 Load

Left: $\frac{1.4}{\cos 30} \times (-0.6) \times 4 \times 0.25$ = -0.894

Right: $\frac{1.4}{\cos 30} \times 0.4 \times 4 \times 0.25$ = 0.596

W6 Load

Left: $\frac{\left(\frac{1.4}{2}+0.82\right)}{\cos 34} \times (-0.6) \times 4 \times 0.25$ = -1.086

Right: $\frac{\left(\frac{1.4}{2}+0.82\right)}{\cos 34} \times 0.4 \times 4 \times 0.25$ = 0.724

Wind Load 1

W1: 0.724

z: $W1 \times \sin 34$: $0.724 \times \sin 34$ = 0.4047

x: $W1 \times \cos 34$: $0.724 \times \cos 34$ = 0.6000

W2: 0.596

z: $W2 \times \sin 34 + W2 \times \sin 38 = 0.596 \times \sin 34 + 0.596 \times \sin 38$ = 1.3785

x: $W2 \times \cos 34 + W2 \times \cos 38 = 0.596 \times \cos 34 + 0.596 \times \cos 38$ = 1.8973

W3: 0.3553

z: $W3 \times \sin 38 = 0.3553 \times \sin 38$ = 0.2188 x: $W3 \times \cos 38 = 0.3553 \times \cos 38$ = 0.2800

W4: -0.5330

z: $W4 \times \sin 38 = -0.5330 \times \sin 38$ = -0.3281

x:$W4 \times \cos 38 = -0.5330 \times \cos 38$ = -0.4200

W5: -0.9699

z: $W5 \times \sin 38 + W5 \times \sin 34 = -0.894 \times \sin 38 + -0.894 \times \sin 34$ = -1.0502

x: $W5 \times \cos 38 + W5 \times \cos 34 = -0.894 \times \cos 38 + -0.894 \times \cos 34$ = -1.4455

W6: -1.1001

z: $W6 \times \sin 34 = -1.086 \times \sin 34$ = -0.6071

x: $W6 \times \cos 34 = -1.086 \times \cos 34$ = -0.9000

Wind Load 2

W1: -1.1001

z: $W1 \times \sin 34 = -1.1001 \times \sin 34$ = -0.6071

x: $W1 \times \cos 34 = -1.1001 \times \cos 34$ = -0.9000

W2: -0.9699

z: $W2 \times \sin 38 + W2 \times \sin 34 = -0.9699 \times \sin 38 + -0.9699 \times \sin 34$ = -1.0502

x: $W2 \times \cos 38 + W2 \times \cos 34 = -0.9699 \times \cos 38 + -0.9699 \times \cos 34$

= -1.4455

W3: -0.5330

z: $W3 \times \sin 38 = -0.5330 \times \sin 38$ = -0.3281

x: $W3 \times \cos 38 = -0.5330 \times \cos 38$ = -0.4200

W4: 0.3553

z: $W4 \times \sin 38 = 0.3553 \times \sin 38$ = 0.2188

x: $W4 \times \cos 38 = 0.3553 \times \cos 38$ = 0.2800

W5: 0.6466

z: $W5 \times \sin 34 + W5 \times \sin 38 = 0.6466 \times \sin 34 + 0.6466 \times \sin 38$ = 1.3785

x: $W5 \times \cos 34 + W5 \times \cos 38 = 0.6466 \times \cos 34 + 0.6466 \times \cos 38$ = 1.8973

W6: 0.7334

z: $W6 \times \sin 34 = 0.7334 \times \sin 34$ = 0.4047

x: $W6 \times \cos 34 = 0.7334 \times \cos 34$ = 0.6000

2.3. Stair Design

2.3.1. Stair Design Data

Stairs are a set of steps that connect one floor of a building to another. Information about stair:

1. Height of each floor = 4 m
2. Op = 160 mm
3. An = 300 mm

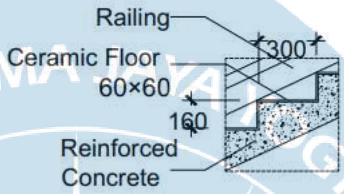

Figure 2.2. Stair Detail

4. Stair width: 4000/2 = 2000 mm
5. Minimum borders width: 4000/2 = 2000 mm
6. Number of staircases: 4000/160 = 25
7. Stair Length: ½ (No. Staircases/Op - 1) An = ½ ((4000/160)-1).330 = 3960 mm ≈ 4 m
8. α: tan^-1(Op/An) = tan^-1(160/300) = 28.07° ≈ 28°

2.3.2. Stair Calculation

Number of Staircases: 25 Staircases

Height Between Floors: 4 m

Antrede (An): 30 cm = 300 mm

Optrede (Op): 153.8 mm ≈ 160 mm

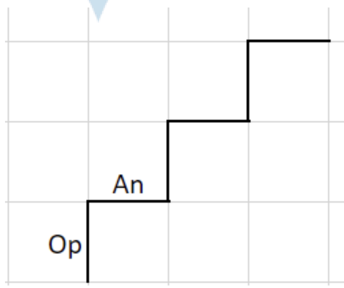

Figure 2.3. Stair Design Illustration

Figure 2.3. shows the location of Optrade (Op) and Antrade (An) of the stair. Antrade is the width of the staircases in stair and Optrade is the height of the staircases in

stair.

a. Control

$600 \leq 2 \times Op + An \leq 650$

$600 \leq 2 \times 160 + 300 \leq 650$

$600 \leq 620 \leq 650$ => OK!

b. Stair Slope

$\alpha = \tan^{-1}\left(\frac{160}{300}\right) = 28.07°$

c. Thickness of Stairs

Tt: $\frac{\frac{1}{2} \times An \times Op}{\sqrt{An^2 + Op^2}}$: $\frac{\frac{1}{2} \times 300 \times 160}{\sqrt{300^2 + 160^2}}$: 70.59 mm

t: 150 mm

t' = $\frac{(t+tt)}{\cos\alpha} = \frac{(150+70.59)}{\cos 28.07}$ = 249.99 ≈ 250 mm

2.3.3. Stair Load

a. Load every 1 meter wide of stair:

Dead Load

Plate: $\frac{250}{1000} \times 24 = 6$ kN/m

Tiles: $0.03 \times 24 = 0.72$ kN/m

Mortars: $0.02 \times 21 = 0.42$ kN/m

Railing: 1 kN/m

Dead Load: 8.14 kN/m ≈ 8.5 kN/m

Live Load

Office: 5 kN/m

b. Load every 1 meter wide of bordes:

Dead Load

Plate: $0.15 \times 24 = 3.6$ kN/m

Tiles: $0.03 \times 24 = 0.72$ kN/m

Mortar: $0.02 \times 21 = 0.42$ kN/m

Railing: 1 kN/m

Dead Load: 5.74 kN/m ≈ 6 kN/m

Live Load

Office: 5 kN/m

2.3.4. Stair Modelling

From our stair design in SAP2000, we got the value of our stair and bordes Moment Field and Moment Support. We calculate the moments and use it to design the safest reinforcement, so that our stair and bordes will be safe from the load combinations given.

Stair

Mu field: 28.097 kN/m

Mu support: - 22.253 kN/m

Bordes

Mu field: 0 kN/m

Mu support: - 22.253 kN/m

Fc' = 25 Mpa

Fy = 240 Mpa

a. Determine the minimum thickness of the plate

Stair Plate

H_{min}: $\frac{1}{24}L\left(0{,}4+\frac{fy}{700}\right)=\frac{1}{24}\left(\frac{24}{2}\times\frac{300}{\cos 28.07°}\right)\left(0{,}4+\frac{fy}{700}\right)$: 126.283 < 250 mm -> SAFE

Bordes Plate

H_{min}: $\frac{1}{24}L\left(0{,}4+\frac{fy}{700}\right)=\frac{1}{24}\left(6000-\frac{24}{2}\times 300\right)\left(0{,}4+\frac{240}{700}\right)$: 74.286 mm < 150mm -> SAFE

b. Determine the effective height of the plate

Stair Plate

$d = 150-\left(20+\frac{1}{2}\times 13\right) = 123.5$ mm

Bordes Plate

$d = 150-\left(20+\frac{1}{2}\times 13\right) = 123.5$ mm

2.3.5. Stair Reinforcement

Main Reinforcement: D 13 = 132.732 mm^2

Shrinkage Reinforcement: D 8 = 50.2655 mm^2

Stair field Reinforcement

$$Rn = \frac{Mu}{0.9\times Bw\times d^2} = \frac{28.097}{0.9\times 1000\times 123.5^2} = 2.047 \text{ Mpa}$$

$$\rho_{req} = \frac{0.85 \times fc'}{fy}\left(1 - \sqrt{1 - \frac{2Rn}{0.85 \times fc'}}\right)$$

$$= \frac{0.85\times25}{240}\left(1 - \sqrt{1 - \frac{2.047}{0.85\times25}}\right) = 8.9846 \times 10^3$$

$$As_{req} = \rho_{req} \times bw \times d = 8.9846 \times 10^3 \times 1000 \times 150 = 1109.598 \text{ mm}^2$$

$$As_{min} = 0.0018 \times bw \times h = 270 \text{ mm}^2$$

Use the biggest (Reinforcement Area) As = 1109.598 mm^2

$$S_{req} = \frac{As\times bw}{As\ Used} = \frac{132.732 \times 1000}{1109.598} = 119.62 \text{ mm} \approx 100 \text{ mm}$$

$$S_{shrinkage} = 150 \text{ mm}$$

Check $S_{shrinkage}$: $\frac{Bw}{S_{shrinkage}} \times As = \frac{1000}{150} \times 50.2655 = 335.103 \text{ mm}^2 > 270 \text{ mm}^2$

Stair Support Reinforcement

$$Rn = \frac{Mu}{0.9 \times bw \times d^2} = \frac{22.253}{0.9 \times 1000 \times 123.5^2} = 1.6211 \text{ Mpa}$$

$$\rho_{req} = \frac{0.85\times fc'}{fy}\left(1 - \sqrt{1 - \frac{2Rn}{0.85\times fc'}}\right)$$

$$\rho_{req} = \frac{0.85\times25}{240}\left(1 - \sqrt{1 - \frac{2\times1.621}{0.85\times25}}\right) = 7.034 \times 10^3$$

$$As_{req} = \rho_{req} \times bw \times d = 7.034 \times 10^3 \times 1000 \times 123.5 = 868.687 \text{ mm}^2$$

$$As_{min} = 0.0018 \times 1000 \times 150 = 270 \text{ mm}^2$$

As Used: 868.687 mm^2

$$S_{req} = \frac{As\times bw}{As\ Used} = \frac{132.732 \times 1000}{868.687} = 152.796 \text{ mm} \approx 150 \text{ mm}$$

$$S_{shrinkage} = 150 \text{ mm}$$

Check $S_{shrinkage}$: $\frac{Bw}{S_{Shrinkage}} \times As = \frac{1000}{150} \times 50.2655 = 335.103 \text{ mm}^2 > 270 \text{ mm}^2$ (As_{min})

Bordes Field Reinforcement

$$Rn = \frac{Mu}{0.9 \times bw \times d^2} = \frac{0}{0.9 \times 1000 \times 123.5^2} = 0$$

$$\rho_{req} = \frac{0.85\times fc'}{fy}\left(1 - \sqrt{1 - \frac{2Rn}{0.85\times fc'}}\right)$$

$$\rho_{req} = \frac{0.85\times25}{240}\left(1 - \sqrt{1 - \frac{2\times0}{0.85\times25}}\right) = 0$$

$$As_{req} = \rho_{req} \times bw \times d = 0 \times 1000 \times 123.5 = 0$$

$$As_{min} = 0.0018 \times 1000 \times 150 = 270 \text{ mm}^2$$

Use 270 mm^2

$$S_{req} = \frac{As \times bw}{As\ Used} = \frac{132.732 \times 1000}{270} = 491.6 \text{ mm} \approx 400 \text{ mm}$$

$$S_{shrinkage} = 150 \text{ mm}$$

Check $S_{shrinkage}$: $\frac{Bw}{S_{Shrinkage}} \times As = \frac{1000}{150} \times 50.2655 = 335.103 \text{ mm}^2 > 270 \text{ mm}^2 \ (As_{min})$

Bordes Support Reinforcement

$$Rn = \frac{Mu}{0.9 \times bw \times d^2} = \frac{22.253}{0.9 \times 1000 \times 123.5^2} = 1.6211 \text{ Mpa}$$

$$\rho_{req} = \frac{0.85 \times fc'}{fy}\left(1 - \sqrt{1 - \frac{2Rn}{0.85 \times fc'}}\right)$$

$$\rho_{req} = \frac{0.85 \times 25}{240}\left(1 - \sqrt{1 - \frac{2 \times 1.621}{0.85 \times 25}}\right) = 7.034 \times 10^3$$

$$As_{req} = \rho_{req} \times bw \times d = 7.034 \times 10^3 \times 1000 \times 123.5 = 868.687 \text{ mm}^2$$

$$As_{min} = 0.0018 \times 1000 \times 150 = 270 \text{ mm}^2$$

As Used: 868.687 mm^2

$$S_{req} = \frac{As \times bw}{As\ Used} = \frac{132.732 \times 1000}{868.687} = 152.796 \text{ mm} \approx 150 \text{ mm}$$

$$S_{shrinkage} = 150 \text{ mm}$$

Check $S_{shrinkage}$: $\frac{Bw}{S_{Shrinkage}} \times As = \frac{1000}{150} \times 50.2655 = 335.103 \text{ mm}^2 > 270 \text{ mm}^2 \ (As_{min})$

Conclusion:

1. Stair Field Reinforcement

   Main Reinforcement: D13 - 100

   Shrinkage Reinforcement: D8 - 150
2. Stair Support Reinforcement

   Main Reinforcement: D13 - 150

   Shrinkage Reinforcement: D8 - 150
3. Bordes Field Reinforcement

   Main Reinforcement: D13 - 400

   Shrinkage Reinforcement: D8 - 150
4. Bordes Support Reinforcement

   Main Reinforcement: D13 - 150

Shrinkage Reinforcement: D8 – 150

2.4. Building Design Structure

2.4.1. Structure General Description

This chapter presents the complete structural design process for the MRP building in West Papua. The building structure consists of 2 floors designed using a special moment resisting frame system (SRPMK). Structural elements such as floor plates, beams, and columns in the building structure are designed using steel materials. The sloof beam and foundation use reinforced concrete material. The plan is divided into one building with 3 levels, including the lower ground.

2.4.2. Structure General Specification

The specifications of the materials used in the structural design of the residence (office) are presented as follows:

Steel Profile

a. The steel Profile that is used is BJ 37 with yield stress, fy = 240 MPa, and ultimate tensile stress, fu = 370 MPa

b. Modulus of Elasticity of Steel, Es = 200.000 MPa

Concrete

a. Concrete Compression Strength after 28 Days, fc' = 30 MPa (Bottom Structure)

b. Modulus of Elasticity of Concrete, Ec = 4700, fc' = 23500 MPa

Reinforcement Steel

a. Reinforcement Steel with D > 12 mm, used threaded steel (*deform*) with yield stress, fy = 240 MPa

b. Reinforcement Steel with D ≤ 12 mm, used plain steel with a yield stress, fy = 240 MPa

c. Modulus of Elasticity of Steel, Es = 200.000 MPa

2.4.3. Load Plan

Gravity Load

Gravity load is determined based on SNI 1727:2013 Minimum Load for Design of Buildings and Other Structures. The gravity load in the structural design of a residential house includes the structure's own weight/dead load (DL), additional dead load (ADL), and live load (LL). These expenses are explained as follows:

a. Structure's own weight (Dead Load) (DL)

The structure's own weight/dead load is the weight of each structural element in the form of floor slabs, beams, columns, etc. which are part of the main structure. In structural modeling using software, the self-weight of the structure will be calculated automatically by the software based on the data on the density of the material and the dimensions of the structural elements entered in the software.

b. Additional Dead Load (ADL)

Additional Dead Load is an additional load due to the use of non-structural components (architectural and MEP) that are attached to and burden the main structure of the building. The additional dead load is explained as follows:

1. Additional Dead Load at Floor Slab

Dead Load	= 0,15 x 24 kN/m3	= 3.6 kN/m2
Reinforce	= 0,02 x 21 kN/m3	= 0,66 kN/m2
Ceramic (thick 3 cm)	= 0,03 x 24 kN/m3	= 0,72 kN/m2
Ceiling and hangers	= 0,18 kN/m2	
Total Additional Dead Load	= 5 kN/m2	

2. Additional Dead Load at Beam

Wall (effective height 4 m) = 4 x 7x 0.15 kN/m2 = 4.2 kN/m

c. Live Load (LL)

Live load is the load that occurs because of the use of the building structure. The living burden can come from people/goods that can move places. The building is included in the category of public space according to SNI 1727:2013 so that the living expenses are determined as follows: Live Load = 2.5 kN/m2

Earthquake Load

Earthquake loads are determined based on SNI 1726:2012 Earthquake Resistance Planning Standards for Building Structures. The steps for calculating the design earthquake load are presented as follows:

a. Determine the building risk category (I-IV)

The building risk category is determined based on the operational function/type of utilization of a building. In SNI 1726:2012, building risk categories are divided into 4 types, namely risk categories I, II, III, and IV (see Table 2.1). In this work, the MRP building structure is included in the office building category, so it is designated as building risk category III.

Table 2.1. Building Risk Category

Utilization Type	Category Risk
Buildings and non-buildings that have a low risk to human life in the event of a failure, including, but not limited to, among others: 1 Agricultural, plantation, livestock, and fishery facilities 2 Temporary facilities 3 Warehouse 4 Guardhouses and other small structures	I
All other buildings and structures, except those included in risk categories I, III, IV, including, but not limited to: 1 Housing area 2 Home shop and home office 3 Market 4 Office building 5 Apartment/flat building 6 Shopping centers/malls 7 Industrial building	II
Buildings and non-buildings that have a high risk to human life in the event of a failure, including, but not limited to: 1 Cinema 2 Meeting hall 3 Stadium 4 Health facilities that do not have surgical units and emergency units 5 Childcare facilities 6 Prison 7 Buildings for the elderly Buildings and non-buildings, not included in risk category IV, which have the potential to cause major economic impacts and/or mass disruption to people's daily lives in the event of a failure, including, but not limited to: 1 Typical power plant 2 Water handling facilities 3 Waste handling facilities 4 Telecommunications centre	III

Utilization Type	Category Risk
Buildings and non-buildings that are not included in risk category IV, (Including, but not limited to manufacturing facilities, processes, handling, storage, use or disposal of hazardous fuels, hazardous chemicals, hazardous waste, or explosive materials) contain toxic or explosive materials where the amount of material content exceeds the limit value required by the competent writersity and is sufficient to pose a danger to the public in the event of a leak.	
Buildings and non-buildings indicated as important facilities, including, but not limited to: 1 Monumental buildings 2 School buildings and educational facilities 3 Hospitals and other healthcare facilities with surgical facilities and emergency departments, emergency vehicle garages 4 Firefighting facilities, ambulances, and police stations, as well as monumental buildings 5 School buildings and educational facilities 6 Hospitals and other healthcare facilities with surgical facilities and emergency departments, emergency vehicle garages 7 Fire, ambulance, and police station facilities, as well as an emergency vehicle garage	IV

Source: SNI 1726:2012

b. Determining earthquake priority factors (Ie)

The earthquake priority factor is determined based on the building risk category. In Table 2.2, the earthquake priority factor (Ie) is presented in accordance with SNI 1726:2012. In this work, the residential structure is included in the building risk category III so that the earthquake priority factor (Ie) is set at 1.25.

Table 2.2. Earthquake Priority Factor (Ie)

Risk Category	Earthquake Priority Factor (Ie)
I atau II	1
III	1,25
IV	1,5

Source: SNI 1726:2012

c. Determine the ground acceleration parameter (Ss dan S1)

The ground acceleration parameters (Ss and S1) are influenced by the soil properties at the project site. The values of Ss and S1 are used to determine the spectral response to the acceleration of the MCER earthquake at ground level, where Ss and S1 are the parameters of the spectral response to the acceleration of the MCER earthquake, respectively, which are mapped for a short period and a period of 1.0 second. The values of Ss and S1 (see Figure 2.4 and Figure 2.5) for the risk-targeted maximum considered earthquake (MCER) are presented in bedrock. In this work, the location of the building is in the city of Manokwari, so the values of Ss = 1500g and S1 = 0.600g are used.

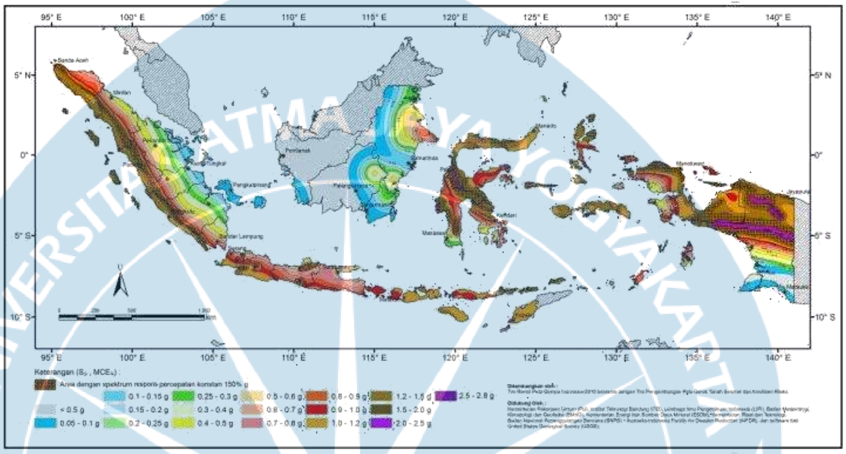

Figure 2.4. Ss Risk-Targeted Maximum Considered Earthquake (MCER) On Bedrock for A Short Period (0,2 Seconds)

Source: SNI 1726:2012

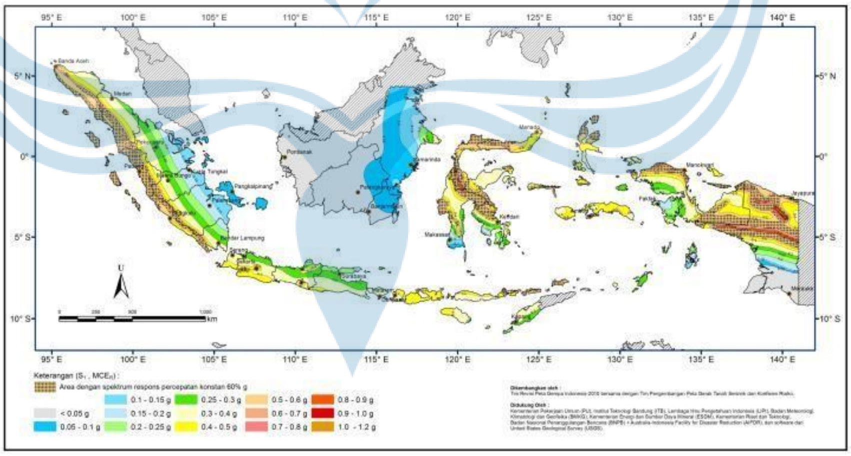

Figure 2.5. S1 Risk-Targeted Maximum Earthquake Considered (MCER) On Bedrock for A Period Of 1 Second

Source: SNI 1726:2012

d. Define site classification (SA - SF)

The characteristics of the project site, especially those related to geotechnical aspects, must be identified properly in the planning process through site investigation activities. The project site investigation activities can be in the form of soil investigations in the field and in the laboratory. Furthermore, the results of the investigation of the project site will be used as a basis for determining site classification. In SNI 1726:2012 the site classification is divided into 6 types, namely SA (hard rock), SB (rock), SC (hard soil), SD (medium soil), SE (soft soil), and SF (special soil) (see Table 2.3).

Table 2.3. Site Classification

Site class	(m/sec)	or	(kPa)
SA (hard rock)	>1500	N/A	N/A
SB (rock)	750 to 1500	N/A	N/A
SC (hard soil, very dense and soft rock)	350 to 750	>50	≥100
SD (medium soil)	175 to 350	15 to 50	50 to 100
SE (soft soil)	<175	<15	<50
	Or any soil profile containing more than 3 m of soil with the following characteristics: 1. Plasticity index, *PI*>20 2. Moisture content, *w*≥ 40% 3. Non-linear shear strength, *su*< 25 kPa		
SF (special soil, requiring specific geotechnical investigation and site-specific response analysis following Article 6.10.1)	Any soil profile that has one or more of the following characteristics: 1. Prone and has the potential to fail or collapse due to earthquake loads such as easy liquefaction, very sensitive clay, weak cemented soil 2. Very organic loam and/or peat (H thickness > 3 m) 3. Very high plasticity clay (thickness *H*> 7.5m with Plasticity Index *PI*> 75) 4. Soft/semi-firm clay layer with a thickness of H > 35 m with su < 50 kPa		
Note: N/A = cannot be used			

Source: SNI 1726:2012

Based on the results of the N-SPT test conducted in the field, the project site is included in the SD site classification (medium soil). The complete N-SPT data can be seen in the soil investigation report.

e. Determine the site coefficients (Fa and Fv)

To determine the spectral response of the MCER earthquake acceleration mapped on the ground surface, an amplification factor is needed for a period of 0.2 seconds (Fa) and 1 second (Fv). The amplification factor is determined based on the site class and ground acceleration parameters. The amplification factor in the 0.2 second (Fa) period was determined by the site class and the MCER earthquake acceleration spectral response parameter was mapped for the 0.2 second (Ss) period. Meanwhile, the amplification factor in the 1 second period (Fv) is determined by the site class and the MCER earthquake acceleration spectral response parameter is mapped for a 1 second period (S1). Determination of site coefficients (Fa and Fv) is based on Tables 2.4. and 2.5.

Table 2.4. Site Coefficient, Fa

Cite Class	The parameter of the spectral response of the MCER earthquake acceleration is mapped over a short period, T = 0.2 seconds (Ss)				
	Ss ≤ 0,25	Ss = 0,5	Ss = 0,75	Ss = 1,0	Ss ≥ 1,25
SA	0,8	0,8	0,8	0,8	0,8
SB	1,0	1,0	1,0	1,0	1,0
SC	1,2	1,2	1,1	1,0	1,0
SD	1,6	1,4	1,2	1,1	1,0
SE	2,5	1,7	1,2	0,9	0,9
SF	SS[b]				

Source: SNI 1726:2012

Notes:

a. For values between Ss, linear interpolation can be performed.

b. SS = site that requires a specific geotechnical investigation and site-specific response analysis, see Section 6.10.1.

Table 2.5. Site Coefficient, Fv

Cite Class	The parameter of the spectral response of the MCER earthquake acceleration is mapped over a short period, T = 0.2 seconds (Ss)				
	S1 ≤ 0,25	S1 = 0,5	S1 = 0,75	S1 = 1,0	S1 ≥ 1,25
SA	0,8	0,8	0,8	0,8	0,8
SB	1,0	1,0	1,0	1,0	1,0
SC	1,7	1,6	1,5	1,4	1,3

Cite Class	The parameter of the spectral response of the MCER earthquake acceleration is mapped over a short period, T = 0.2 seconds (Ss)				
	S1 ≤ 0,25	S1 = 0,5	S1 = 0,75	S1 = 1,0	S1 ≥ 1,25
SD	2,4	2,0	1,8	1,6	1,5
SE	3,5	3,2	2,8	2,4	2,4
SF	SS[b]				

Source: SNI 1726:2012

Notes:

a. For values between S1, linear interpolation can be performed.

b. SS = site that requires a specific geotechnical investigation and site-specific response analysis.

Based on Table 2.4 and Table 2.5, for the SD site class (medium soil) the Fa and Fv values were 1.2 and 1.8, respectively. Furthermore, the values of Fa and Fv are used to determine the parameters of the acceleration response spectrum in the short period (SMS) and 1 second period (SM1) which can be calculated using the following equation:

a. SMS = Fa x Ss = 1.800 g

b. SM1 = Fv x S1 = 1.08 g

f. Calculating design acceleration parameters (SDS and SD1)

In the previous step, the SMS and SM1 values have been obtained. Furthermore, based on the SMS and SM1 values, the design spectral acceleration parameters for a short period of 0.2 seconds (SDS) and a period of 1 second (SD1) need to be determined to construct the response spectral curve. SDS and SD1 values are calculated using the following equation:

a. SDS = 2/3 x SMS = 1.2 g

b. SD1 = 2/3 x SM1 = 0.72g

g. Define seismic design category (KDS: A - F)

The designed structure must be determined to be included in the seismic design category (KDS) in accordance with Article 6.5 of SNI 1726:2012. In Table 2.6 and Table 2.7 are presented seismic design categories based on the relationship between SDS and SD1 with KDS.

Table 2.6. Seismic design categories based on SDS values

SDS Value	Risk Category	
	I or II or III	IV
SDS < 0.167	A	A
0.167 ≤ SDS < 0.330	B	B
0.33 ≤ SDS < 0.500	C	C
0.5 ≤ SDS	D	D

Source: SNI 1726:2012

Table 2.7. Seismic design categories based on SD1 values

SDS Value	Risk Category	
	I or II or III	IV
SD1 < 0,167	A	A
0,067≤ SD1< 0,133	B	B
0,133≤ SD1< 0,200	C	C
0,200≤ SD1	D	D

Source: SNI 1726:2012

2.4.4. Combined Planned Load

The ultimate load combination is determined based on Article 4.2.2 of SNI 1726:2012 Procedure for Calculation of Concrete Structures for Buildings, which are as follows:

a. 1,4DL
b. 1,2DL + 1,6LL + 0,5(Lr or R)
c. 1,2DL + 1,6(Lr atau R) + (1,0L or 0,5W)
d. 1,2DL + 1,0W + 1,0L + 0,5(Lr or R)
e. 1,2DL + 1,0E + 1,0LL
f. 0,9DL + 1,0W
g. 0,9DL + 1,0E

For load combinations number 5 and 7 which is a combination of earthquake loads, it is specifically regulated in Article 7.4 SNI 1726:2012 Earthquake Resistance Planning Standards for Building Structures, which are as follows:

a. (1,2+0,2SDS)DL + 1,0LL ± 0,3ρEx ± 1,0ρEy
b. (1,2+0,2SDS)DL + 1,0LL ± 1,0ρEx ± 0,3ρEy
c. (0,9-0,2SDS)DL ± 0,3ρEx ± 1,0ρEy
d. (0,9-0,2SDS)DL ± 1,0ρEx ± 0,3ρEy

While the combination of service loads is determined based on Article 4.2.3 SNI

1726:2012 Procedures for Calculation of Concrete Structures for Buildings, which are as follows:

a. DL
b. DL + LL
c. DL + (Lr or R)
d. DL + 0,75LL + 0,75(Lr or R)
e. DL + (0,6W or 0,7E)
f. DL + 0,75(0,6W or 0,7E) + 0,75LL + 0,75(Lr or R)
g. 0,6DL + 0,6W
h. 0,6DL + 0,7E

where,

DL = Dead load (self-weight of structure and additional dead load)
LL = Live load
Lr = Live load on the roof structure
R = Rain load W = Wind load
Ex = earthquake load direction x Ey = earthquake load direction y
ρ = Redundancy factor
SDS = Design spectral acceleration parameter for a short period of 0.2 seconds.

The ultimate load combinations used in this work are presented in Table 2.8.

Table 2.8. Combination of service loads

Load Combination	DL	ADL	LL	Ex	Ey
COMB1	1,4	1,4	-	-	-
COMB2	1,2	1,2	1,6	-	-
COMB3	1,357	1.357	1	-1,3	-0,39
COMB4	1,357	1.357	1	-1,3	0,39
COMB5	1,357	1.357	1	1,3	-0,39
COMB6	1,357	1.357	1	1,3	0,39
COMB7	1,357	1.357	1	-0,39	-1,3
COMB8	1,357	1.357	1	-0,39	1,3
COMB9	1,357	1.357	1	0,39	-1,3
COMB10	1,357	1.357	1	0,39	1,3
COMB11	0,743	0,743	-	-1,3	-0,39
COMB12	0,743	0,743	-	-1,3	0,39
COMB13	0,743	0,743	-	1,3	-0,39
COMB14	0,743	0,743	-	1,3	0,39

Load Combination	DL	ADL	LL	Ex	Ey
COMB15	0,743	0,743	-	-0,39	-1,3
COMB16	0,743	0,743	-	-0,39	1,3
COMB17	0,743	0,743	-	0,39	-1,3
COMB18	0,743	0,743	-	0,39	1,3

2.4.5. Structure Modeling

Structural modeling is carried out to determine the internal forces that occur in structural elements and the behavior of the structure due to the working load. For the model structure of the MRP Building can be seen in Figure 2.6.

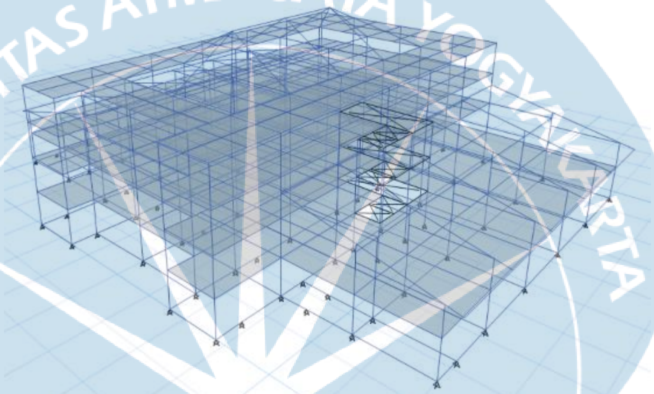

Figure 2.6. Model structure of the MRP Building

The materials used in the structural analysis are as follows:

a. Steel: fy 240Mpa fu 370Mpa
b. Plain Reinforcing Steel: fy 240 Mpa
c. Threaded Reinforcing Steel: fy 400 Mpa
d. Concrete: fc' 30 Mpa / K30

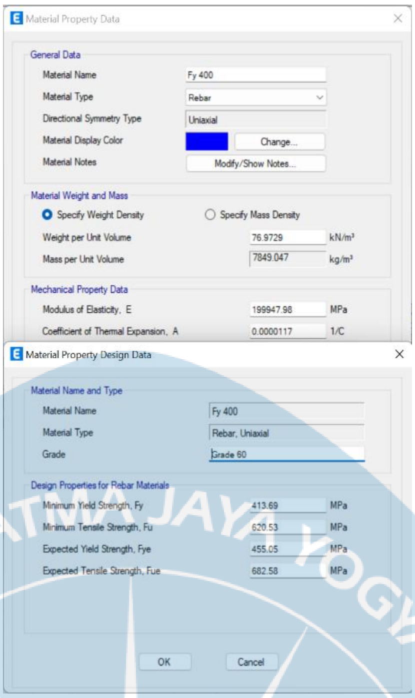

Figure 2.7. Material Property Data

Figure 2.7 shows the material property data, which are general data, material weight and mass, and mechanical property data. Then for material property design data are material name and type and design properties for rebar materials.

Beam and Column Profile. The beam and column cross sections are defined as in Figure 2.8 and Figure 2.9:

Beam profile in ETABS:

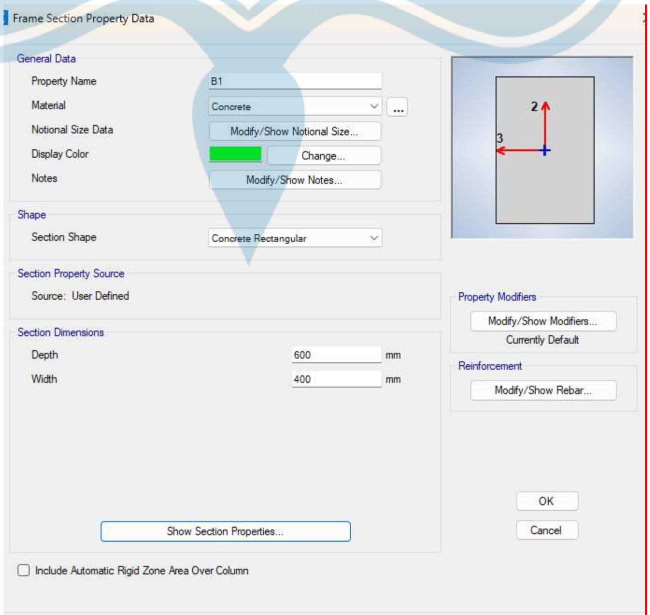

Figure 2.8. Frame Section Property Data of Main Beam

Figure 2.9. Frame Section Property Data of Secondary Beam

Column profile in ETABS

Figure 2.10. Frame Section Property Data of Column

2.4.6. Interpretation 3D Structure Modeling

After the material and section properties are complete, the next step is to create a 3D model. The model accommodates all sizes of beams and columns, along with the reinforcement planned to be installed as shown in Figure 2.11.

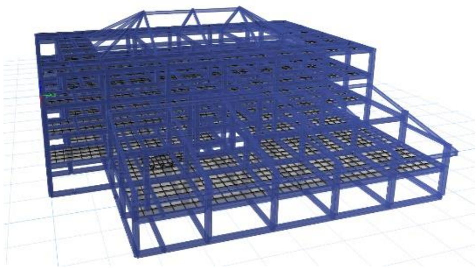

Figure 2.11. 3D Model of MRP Building

Giving Workload

Broadly speaking, the loads acting on this structure are divided into:

a. dead load (self-weight of steel, wall load),

b. additional dead load (ceramic load, pipe, ceiling etc.)

c. live load (working load), and

d. earthquake load. (Response spectrum)

A complete explanation of the loading is found in the Loading section of this report.

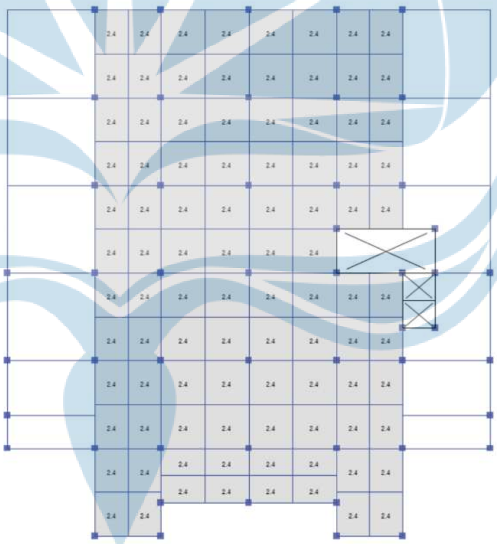

Figure 2.12. Loading of Floor Slab from Top View Model

Figure 2.12 shows the load on the floor slab from the top view designed using ETABS.

Giving Earthquake Load

Earthquake load is modeled in the program with the response spectrum function. Calculations and quantities can be seen in the Input data – response spectrum section. After obtaining the spectrum response graph, the graph is then input into the program, as shown in Figure 2.13.

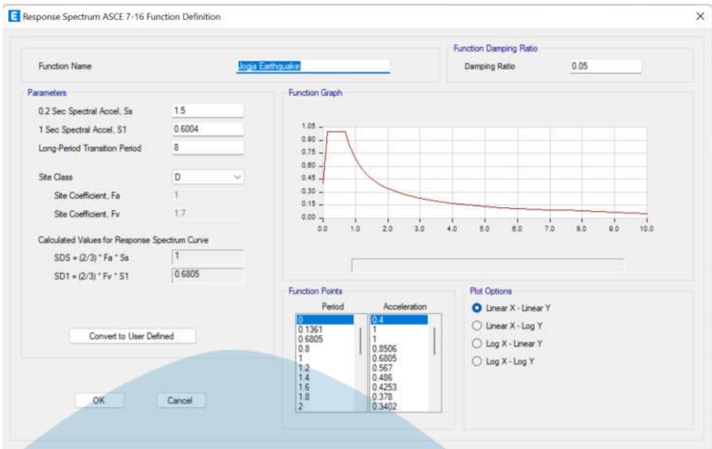

Figure 2.13. Input Earthquake Load

After the earthquake load has entered, a combination of loading is carried out which allows several extreme loads to work together.

Giving Combinations and Loading Factors

The load combination used refers to the 2012 Earthquake SNI, in this report the discussion of the load combination is carried out in the Data Input – Loading Combination section (see Figure 2.14).

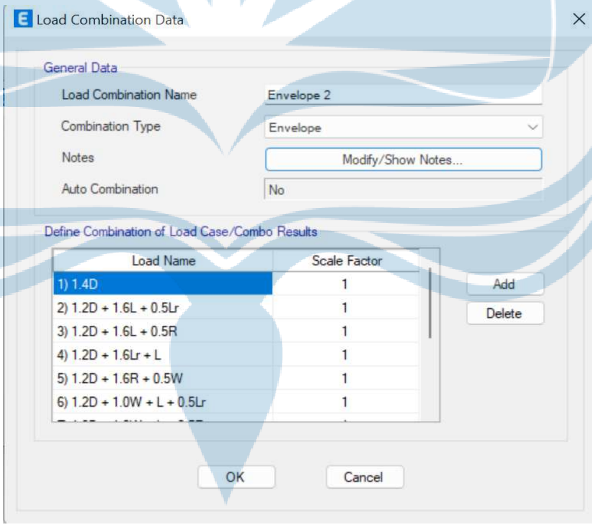

Figure 2.14. Input Load Combination

Running Program

After all the forces are installed, several treatments on the structure are carried out such as giving a mass source and a diaphragm, after which the program is run. The result of running the program is in the form of internal forces acting on the beams and columns of the structure. This force is the key in analyzing the strength of the structure itself. The force obtained in the running results can be seen in Figure 2.15.

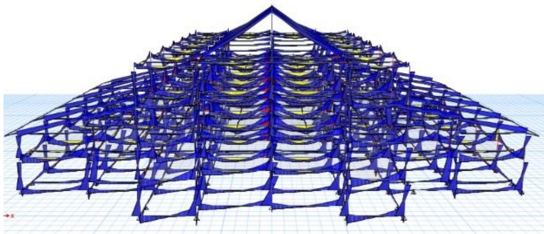

Figure 2.15. Working Force, Program Running Result

Internal Force Result

The forces that have been obtained are then exported into excel and analyzed. Each structural element is checked for safety values. The checking results are also displayed in excel as in the attachment.

2.4.7. Structure Behavior Check

1. Structural Fundamental Period Check

In this work, the type of structure used is a moment-bearing steel frame so that the values of Ct = 0.0724 and x = 0.8 are obtained. Furthermore, based on the value of SD1 = 0.479 g, the coefficient of Cu = 1.4 is obtained. So that the value of Ta = 0.382 seconds and Cu.Ta = 0.534 seconds is obtained. The period value of the structure modeling results, Tc = 0.713 seconds (Ta < Tc < Cu.Ta). So that the period of the structure used is T = 0.534 seconds.

2. Checking Capital Participation Mass Ratio

Based on the results of structural modeling, the capital participation mass ratio is presented in Table 2.9. The number of modes (modes) required to determine the natural vibrational variation for the structure must be sufficient to obtain a combined mass participation of at least 90% of the actual mass of each orthogonal horizontal direction of the response considered by the model, in accordance with Article 7.9.1 of SNI 1726:2012.

Based on the results of structural modeling, it is found that in both directions involving 582 modes of vibration, it is sufficient to produce more than 90% of the actual mass in both X and Y directions (see Table 2.9).

Table 2.9. Capital Participation Mass Ratio (Building A & Building B)

Case	Mode	Sum UX	Sum UY
Modal	1	0.4367	0.1665
Modal	2	0.5661	0.8349

Case	Mode	Sum UX	Sum UY
Modal	3	0.8631	0.837
Modal	4	0.8788	0.8731
Modal	5	0.8957	0.9127
Modal	6	0.9285	0.9128
Modal	7	0.9285	0.967
Modal	8	0.9458	0.967
Modal	9	0.9564	0.9674
Modal	10	0.9769	0.9679
Modal	11	0.9984	0.9679
Modal	12	0.9989	0.97
Modal	13	0.9991	0.9751
Modal	14	0.9992	0.9841
Modal	15	0.9998	0.9933
Modal	16	0.9999	0.9967
Modal	17	1	0.9996
Modal	18	1	0.9999
Modal	19	1	0.9999

3. Base Shear Check

In the seismic load analysis procedure of the variance response spectrum (RS), the base shear obtained must be compared with the base shear resulting from the equivalent lateral force (ELF) seismic load analysis procedure. The base shear of the variance response spectrum (RS) shall be not less than 85% of the equivalent lateral force (ELF) base shear. If this is not met, then the force scale factor on the variance response spectrum (RS) must be recalculated. In the following, the results of the calculation and checking of the base shear are presented to determine whether it is necessary to recalculate the force-scale factor of the variance response spectrum (RS).

4. Element Structure Design

The floor slab design is carried out based on SNI 2847:2013 Procedures for Calculation of Concrete Structures for Buildings. In this chapter, the procedure for calculating/designing floor slabs of type P1 (150 mm thick) is presented. Furthermore, the design of other types of floor slabs is carried out with the same calculation procedure. The floor slab design is carried out based on SNI 2847:2013 Procedures for Calculation of Concrete Structures for Buildings. In this chapter, the procedure for calculating/designing floor slabs of type P1 (150 mm thick) is presented. Furthermore, the design of other types of floor slabs is carried out with the same calculation procedure.

2.5. Slab Design

The floor slab design is carried out based on SNI 2847:2013 Procedures for Calculation of Concrete Structures for Buildings. In this chapter, the procedure for calculating/designing floor slabs of type P1 (150 mm thick) is presented. Furthermore, the design of other types of floor slabs is carried out with the same calculation procedure. The floor slab design is carried out based on SNI 2847:2013 Procedures for Calculation of Concrete Structures for Buildings. In this chapter, the procedure for calculating/designing floor slabs of type P1 (150 mm thick) is presented. Furthermore, the design of other types of floor slabs is carried out with the same calculation procedure.

2.5.1. Calculation of Slab

Determine The Load on Slab According to The Function

Semi-Basement, 1st, and 2nd Floor

Dead Load

Self-Weight = 3.6 kN/m2

Tiles Weight = 0,03 x 24 = 0.72 kN/m2

Reinforced Weight = 0,02 x 21 = 0.42 kN/m2

Ceiling Weight = 0.18 kN/m2

Sum = 4.92 kN/m2 = 5 kN/m2

Live Load

Live Load = 2500 kg/m2 = 2.5 kN/m2

Combination Load

Qu = 9.904 kN/m2 = 10 kN/m^2

Determine The Minimum Thickness of The Slab

Assume Dimension

<u>Beam 1 (Primary Beam)</u>

Bw: 600 mm

H: 400 mm

<u>Beam 2 (Secondary Beam)</u>

Bw: 350 mm

H: 400 mm

Slab Design

Bw:1400 mm

H: 800 mm

Hmin : $\frac{\ln\left(0.8+\left(\frac{fy}{1500}\right)\right)}{36+5\beta(\alpha-0.2)}$ $->$ αfm $<$ 2.0 Hmin: $\frac{\ln\left(0.8+\left(\frac{fy}{1500}\right)\right)}{36+9\beta}$ $>$ αfm $\geq$ 2.0

α: $\frac{Ecbx1cb}{Ecpx1cp}$

αfm : avg of α from every side of slab

Ecb = Ecp = 200000Mph

Ln = longest span of slab: 1400 mm

$$\alpha 1: \left(\frac{200000 \times \left(\frac{1}{12}\right) \times 600 \times 400^3}{200000 \times \left(\frac{1}{12}\right) \times 1400 * 800^3}\right) = 0.0536$$

$$\alpha 2: \left(\frac{200000 \times \left(\frac{1}{12}\right) \times 350 \times 400^3}{200000 \times \left(\frac{1}{12}\right) \times 1400 \times 800^3}\right) = 0.0313$$

$$\alpha fm: \frac{(3 \times \alpha 1) + \alpha 2}{4} = 0.0480 < 2$$

Hence , we use Hmin formula : $\frac{\ln\left(0.8+\left(\frac{fy}{1500}\right)\right)}{36+5\beta(\alpha-0.2)}$ = 150 mm

Calculation of Slab Thickness

Slab A

$$H: \frac{(Ly\times\left(0.8+\frac{fy}{1500}\right))}{(36+9\times\frac{Ly}{Lx})}$$

$$H: \frac{(4000\times\left(0.8+\frac{240}{1500}\right))}{(36+9\times\frac{4000}{3500})}$$

H : 82.9630 mm

Hmin : 150 mm

Hused : 150 mm

Slab B

$$H: \frac{(Ly\times\left(0.8+\frac{fy}{1500}\right))}{(36+9\times\frac{Ly}{Lx})}$$

$$H: \frac{(4000\times\left(0.8+\frac{240}{1500}\right))}{(36+9\times\frac{4000}{4000})}$$

H : 85.3333 mm

Hmin : 150 mm

Hused : 150 mm

Slab C

$$H: \frac{(Ly\times\left(0.8+\frac{fy}{1500}\right))}{(36+9\times\frac{Ly}{Lx})}$$

$$H: \frac{(4000\times\left(0.8+\frac{240}{1500}\right))}{(36+9\times\frac{4000}{3000})}$$

H : 80 mm

Hmin : 150 mm

Hused : 150 mm

Slab D

$$H: \frac{(Ly\times\left(0.8+\frac{fy}{1500}\right))}{(36+9\times\frac{Ly}{Lx})}$$

$$H: \frac{(3000\times\left(0.8+\frac{240}{1500}\right))}{(36+9\times\frac{3000}{2500})}$$

H : 61.5385 mm

Hmin : 150 mm

Hused : 150 mm

Slab E

$$H: \frac{(Ly\times\left(0.8+\frac{fy}{1500}\right))}{(36+9\times\frac{Ly}{Lx})}$$

$$H: \frac{(4000\times\left(0.8+\frac{240}{1500}\right))}{(36+9\times\frac{4000}{2500})}$$

H : 76.1905 mm

Hmin : 150 mm

Hused : 150 mm

	1,0	1,1	1,2	1,3	1,4	1,5	1,6	1,7	1,8	1,9	2,0	2,1	2,2	2,3	2,4	2,5	>2,5
(Mlx) = 0,001 qlx^2 x	44	52	59	66	73	78	84	88	93	97	100	103	106	108	110	112	125
(Mly) = 0,001 qlx^2 x	44	45	45	44	44	43	41	40	39	38	37	36	35	34	32	32	25
(Mlx) = − (Mtx) = 0,001 qlx^2 x	36	42	46	50	53	56	58	59	60	61	62	62	62	63	63	63	63
(Mly) = 0,001 qlx^2 x	36	37	38	38	38	37	36	36	35	35	35	34	34	34	34	34	13
− (Mty) = 0,001 qlx^2 x	36	37	38	38	38	37	36	36	35	35	35	34	34	34	34	34	38
(Mlx) = − (Mtx) = 0,001 qlx^2 x	48	55	61	67	71	76	79	82	84	86	88	89	90	91	92	92	94
(Mly) = 0,001 qlx^2 x	48	50	51	51	51	51	51	50	50	49	49	49	48	48	47	47	19
− (Mty) = 0,001 qlx^2 x	48	50	51	51	51	51	51	50	50	49	49	49	48	48	47	47	56
(Mlx) = 0,001 qlx^2 x	22	28	34	41	48	55	62	68	74	80	85	89	93	97	100	103	125
(Mly) = 0,001 qlx^2 x	51	57	62	67	70	73	75	77	78	79	79	79	79	79	79	79	25
− (Mty) = 0,001 qlx^2 x	51	57	62	67	70	73	75	77	78	79	79	79	79	79	79	79	75
(Mlx) = − (Mtx) = 0,001 qlx^2 x	51	54	57	59	60	61	62	62	63	63	63	63	63	63	63	63	63
(Mly) = 0,001 qlx^2 x	22	20	18	17	15	14	13	12	11	10	10	10	9	9	9	9	13
(Mlx) = 0,001 qlx^2 x	31	38	45	53	59	66	72	78	83	88	92	96	99	102	105	108	125
(Mly) = 0,001 qlx^2 x	60	65	69	73	75	77	78	79	79	80	80	80	79	79	79	79	25
− (Mty) = 0,001 qlx^2 x	60	65	69	73	75	77	78	79	79	80	80	80	79	79	79	79	75
(Mlx) = − (Mtx) = 0,001 qlx^2 x	60	66	71	76	79	82	85	87	88	89	90	91	91	92	92	93	94
(Mly) = 0,001 qlx^2 x	31	30	28	27	25	24	22	21	20	19	18	17	17	16	16	15	12
(Mlx) = − (Mtx) = 0,001 qlx^2 x	38	46	53	59	65	69	73	77	80	83	85	86	87	88	89	90	54
(Mly) = 0,001 qlx^2 x	43	46	48	50	51	51	51	51	50	50	50	49	49	48	48	48	19
− (Mty) = 0,001 qlx^2 x	43	46	48	50	51	51	51	51	50	50	50	49	49	48	48	48	56
(Mlx) = − (Mtx) = 0,001 qlx^2 x	13	48	51	55	57	58	60	61	62	62	62	63	63	63	63	63	63
(Mly) = 0,001 qlx^2 x	38	39	38	38	37	36	36	35	35	34	34	34	33	33	33	33	13
− (Mty) = 0,001 qlx^2 x	38	39	38	38	37	36	36	35	35	34	34	34	33	33	33	33	38

Catatan: ——— = Terletak bebas
═══ = Menerus atau terjepit elastis

Figure 2.16. Moment Table

In Figure 2.16, we are using the Moment Table from PBI 1971 in order to get the Clx, Cly, Ctx, Cty value from our slab dimension ratio so that we can calculate the Moments that will occur on our Slab designs. We will get the value of Mlx, Mly, Mtx, Mty.

Mlx (X Direction Field Moment)

Mly (Y Direction Field Moment)

Mtx (X Direction Support Moment)

Mty (Y Direction Support Moment)

Ly: Slab Length

Lx: Slab Width

Calculation of Slab Moments

Slab A

Ly: 4

Lx: 3.5

Ly/Lx = 4/3.5 = 1.1

From Moment Table we got:

Clx: 42

Cly: 37

Ctx: 42

Cty: 37

Required Moments:

Mlx: $= 0.001 \times qu \times Lx^2 \times Clx = 5.11$

Mly: $= 0.001 \times qu \times Ly^2 \times Cly = 5.863$

Mtx: $= 0.001 \times qu \times Lx^2 \times Ctx = 5.11$

Mty: $= 0.001 \times qu \times Ly^2 \times Cty = 5.863$

Required Reinforcement for Slab A

Reinforcement in X direction:

$K = \frac{Mlx}{\omega bd^2} = \frac{5.11 \times 10^6}{1 \times 300 \times 375^2} = 0.121\ MPa$

A $= \left(1 - \sqrt{1 - \frac{2K}{0.85fc'}}\right) = 9.49$ mm

Main Reinforcement

As = $\frac{0.85a \times b \times fc}{fy}$ = 302.49 mm2

Because fc' < 31.36 MPa, then:

As.u =$\frac{1.4 \times b \times d}{fy}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\ Mn$ = 52.79

Mlx = 5.11

Mlx < Md (SAFE)

Field Reinforcement in Y direction:

$K = \frac{Mly}{\omega bd^2} = \frac{5.863 \times 10^6}{1 \times 300 \times 375^2} = 0.139\ MPa$

A= $\left(1 - \sqrt{1 - \frac{2K}{0.85fc'}}\right)$ = 10.01 mm

Main Reinforcement

As = $\frac{0.85a \times b \times fc}{fy}$ = 319.07 mm2

Because fc' < 31.36 MPa, then:

As.u = $\frac{1.4 \times b \times d}{fy}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1:$\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\ Mn$ = 52.79

Mly = 5.83

Mly < Md (SAFE)

Support Reinforcement in X direction:

$K = \frac{Mtx}{\omega bd^2} = \frac{5.11 \times 10^6}{1 \times 300 \times 375^2} = 0.121\ MPa$

A = $\left(1 - \sqrt{1 - \frac{2K}{0.85fc'}}\right)$ = 9.49 mm

Main Reinforcement

As = $\frac{0.85a \times b \times fc}{fy}$ = 302.49 mm2

Because fc' < 31.36 MPa, then:

As.u = $\frac{1.4 \times b \times d}{fy}$= 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4\times 656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4\times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\ Mn$ = 52.79

Mtx = 5.11

Mtx < Md (SAFE)

Support Reinforcement in Y Direction:

$K = \frac{Mly}{\omega b d^2} = \frac{5.863 \times 10^6}{1\times 300\times 375^2} = 0.139\ MPa$

A = $\left(1 - \sqrt{1 - \frac{2K}{0.85 fc'}}\right)$= 10.01 mm

Main Reinforcement

As = $\frac{0.85 a\times b\times fc}{fy}$ = 319.07 mm2

Because fc' < 31.36 MPa, then:

As.u = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4\times 478}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4\times 656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4\times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md =$\omega\ Mn$ = 52.79

Mty = 5.83

Mly < Md (SAFE)

Slab B

Ly: 4

Lx: 4

Ly/Lx = 4/4 = 1

From Moment Table we got:

Clx: 36

Cly: 36

Ctx: 36

Cty: 36

Required Moments:

Mlx: $0.001 \times qu \times Lx^2 \times Clx = 0.001 \times 9.904 \times 4^2 \times 36 = 5.704$

Mly:$0.001 \times qu \times Ly^2 \times Cly = 0.001 \times 9.904 \times 4^2 \times 36 = 5.704$

Mtx: $0.001 \times qu \times Lx^2 \times Ctx = 0.001 \times 9.904 \times 4^2 \times 36 = 5.704$

Mty: $0.001 \times qu \times Ly^2 \times Cty = 0.001 \times 9.904 \times 4^2 \times 36 = 5.704$

Required Reinforcement for Slab B

Field Reinforcement in X direction:

$$K = \frac{Mlx}{\omega bd^2} = \frac{5.704 \times 10^6}{1 \times 300 \times 375^2} = 0.135\ MPa$$

$$A = \left(1 - \sqrt{1 - \frac{2K}{0.85fc'}}\right) = 5.308 \text{ mm}$$

Main Reinforcement

$$As = \frac{0.85a \times b \times fc}{fy} = 169.19 \text{ mm2}$$

Because fc' < 31.36 MPa, then:

$$As.u = \frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478} = 656.25 \text{ mm2}$$

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25}$= 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\ Mn$ =52.79

Mlx = 5.704 kN

Mlx < Md (SAFE)

Fvald Reinforcement in Y direction:

$K = \frac{Mly}{\omega b d^2} = \frac{5.704 \times 10^6}{1 \times 300 \times 375^2} = 0.135\ MPa$

A $= \left(1 - \sqrt{1 - \frac{2K}{0.85fc'}}\right)$ = 5.308 mm

Main Reinforcement

As = $\frac{0.85a \times b \times fc}{fy}$ = 169.19 mm2

Because fc' < 31.36 MPa, then:

As.u = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1:$\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$= 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\, Mn$ =52.79

Mly = 5.704 kN

Mly < Md (SAFE)

Support Reinforcement in X Direction:

$$K = \frac{Mtx}{\omega bd^2} = \frac{5.704 \times 10^6}{1{\times}300{\times}375^2} = 0.135\ MPa$$

A $= \left(1 - \sqrt{1 - \frac{2K}{0.85fc'}}\right)$ = 5.308 mm

Main Reinforcement

As = $\frac{0.85a{\times}b{\times}fc}{fy}$ = 169.19 mm2

Because fc' < 31.36 MPa, then:

As.u = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 {\times} \frac{1000}{656.25}}{4{\times}478}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 {\times} \frac{1000}{656.25}}{4{\times}656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 {\times} \frac{1000}{656.25}}{4{\times}478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\, Mn$ =52.79

Mtx = 5.704 kN

Mtx < Md (SAFE)

Support Reinforcement in Y Direction:

$$K = \frac{Mty}{\omega bd^2} = \frac{5.704 \times 10^6}{1{\times}300{\times}375^2} = 0.135\ MPa$$

A $= \left(1 - \sqrt{1 - \frac{2K}{0.85fc'}}\right)$ = 5.308 mm

Main Reinforcement

As = $\frac{0.85a \times b \times fc}{fy}$ = 169.19 mm2

Because fc’ < 31.36 MPa, then:

As.u = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As =$\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 163.93 mm2

Main renforcement that’ll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\ Mn$ =52.79

Mty = 5.704 kN

Mty < Md (SAFE)

Slab C

Ly: 4

Lx: 3

Ly/Lx = 4/3 = 1.3

From Moment Table we got:

Clx: 50

Cly: 38

Ctx: 50

Cty: 38

Required Moments:

Mlx: $0.001 \times qu \times Lx^2 \times Clx$ = $0.001 \times 9.904 \times 3^2 \times 50$ = 4.4568

Mly: $0.001 \times qu \times Ly^2 \times Cly$ = $0.001 \times 9.904 \times 4^2 \times 38$ = 6.021632

Mtx: $0.001 \times qu \times Lx^2 \times Ctx$ = $0.001 \times 9.904 \times 3^2 \times 50$ = 4.4568

Mty: $0.001 \times qu \times Ly^2 \times Cty = 0.001 \times 9.904 \times 4^2 \times 38 = 6.021632$

Required Reinforcement for Slab C

Field Reinforcement in X direction:

$$K = \frac{Mlx}{\omega bd^2} = \frac{4.4568 \times 10^6}{1 \times 300 \times 375^2} = 0.106\ MPa$$

$$A = \left(1 - \sqrt{1 - \frac{2K}{0.85fc'}}\right) = \left(1 - \sqrt{1 - \frac{2 \times 0.106}{0.85 \times 30}}\right) = 4.165 \text{ mm}$$

Main Reinforcement

$$As = \frac{0.85a \times b \times fc}{fy} = \frac{0.85 \times 4.165 \times 300 \times 30}{240} = 132.76 \text{ mm2}$$

Because fc' < 31.36 MPa, then:

$$As.u = \frac{1.4 \times b \times d}{fy} = \frac{1.4 \times 300 \times 375}{240} = 656.25 \text{ mm2}$$

Biggest As = 656.25 mm2

Reinforcement Spacing:

$$S1: \frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25} = 478.476 \text{ mm}$$

S2: 2 h = 2 × 400 = 800 mm

Smallest S = 478 mm

$$As = \frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478} = 163.93 \text{ mm2}$$

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\ Mn$ =52.79

Mlx = 4.4568 kN

Mlx < Md (SAFE)

Field Reinforcement in Y direction:

$$K = \frac{Mly}{\omega bd^2} = \frac{6.021632 \times 10^6}{1 \times 300 \times 375^2} = 0.143\ MPa$$

$$A = \left(1 - \sqrt{1 - \frac{2K}{0.85fc'}}\right) = \left(1 - \sqrt{1 - \frac{2 \times 0.143}{0.85 \times 30}}\right) = 5.623 \text{ mm}$$

Main Reinforcement

$$As = \frac{0.85a \times b \times fc}{fy} = \frac{0.85 \times 5.623 \times 300 \times 30}{240} = 179.23 \text{ mm2}$$

Because fc' < 31.36 MPa, then:

As.u = $\frac{1.4 \times b \times d}{fy} = \frac{1.4 \times 300 \times 375}{240}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25}$ = 478.476 mm

S2: 2 h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\ Mn$ = 0.9 x Mn = 52.79

Mly = 4.4568 kN

Mly< Md (SAFE)

Support Reinforcement in X Direction

$$K = \frac{Mlx}{\omega b d^2} = \frac{4.4568 \times 10^6}{1 \times 300 \times 375^2} = 0.106\ MPa$$

$$A = \left(1 - \sqrt{1 - \frac{2K}{0.85 fc'}}\right) = 4.165 \text{ mm}$$

Main Reinforcement

As = $\frac{0.85 a \times b \times fc}{fy} = \frac{0.85 \times 4.165 \times 300 \times 30}{240}$ = 132.76 mm2

Because fc' < 31.36 MPa, then:

As.u = $\frac{1.4 \times b \times d}{fy}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $= \frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega \, Mn$

Mtx = 6.021632 kN

Mtx < Md (SAFE)

Support Reinforcement in Y Direction

$K = \frac{Mly}{\omega bd^2} = \frac{6.021632 \times 10^6}{1 \times 300 \times 375^2} = 0.143 \, MPa$

A $= \left(1 - \sqrt{1 - \frac{2K}{0.85fc'}}\right)$ = 5.623 mm

Main Reinforcement

As $= \frac{0.85a \times b \times fc}{fy} = 179.23$ mm2

Because fc' < 31.36 MPa, then:

As.u $= \frac{1.4 \times b \times d}{fy}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25}$ = 478.476 mm

S2: 2 h = 2 x 400 = 800 mm

Smallest S = 478 mm

As $= \frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega \, Mn$ = 0.9 x Mn = 52.79

Mty = 6.021632 kN

Mty< Md (SAFE)

Slab D

Ly: 3

Lx: 2.5

Ly/Lx = 3/2.5 = 1.2

From Moment Table we got:

Clx: 46

Cly: 38

Ctx: 46

Cty: 38

Required Moments:

Mlx: $0.001 \times qu \times Lx^2 \times Clx$ = $0.001 \times 9.904 \times 2.5^2 \times 46$ = 2.8474

Mly: $0.001 \times qu \times Ly^2 \times Cly$ = $0.001 \times 9.904 \times 3^2 \times 38$ = 3.387168

Mtx: $0.001 \times qu \times Lx^2 \times Ctx$ = $0.001 \times 9.904 \times 2.5^2 \times 46$ = 2.8474

Mty: $0.001 \times qu \times Ly^2 \times Cty$ = $0.001 \times 9.904 \times 3^2 \times 38$ = 3.387168

Required Reinforcement for Slab D

Field Reinforcement in X Direction:

$$K = \frac{Mlx}{\omega bd^2} = \frac{2.8474 \times 10^6}{1 \times 300 \times 375^2} = 0.067\ MPa$$

$$A = \left(1 - \sqrt{1 - \frac{2K}{0.85fc'}}\right) = \left(1 - \sqrt{1 - \frac{2 \times 0.067}{0.85 \times 30}}\right) = 2.63 \text{ mm}$$

Main Reinforcement

As ; $\frac{0.85a \times b \times fc}{fy} = \frac{0.85 \times 2.63 \times 300 \times 30}{240}$ = 83.83 mm2

Because fc' < 31.36 MPa, then:

As.u = $\frac{1.4 \times b \times d}{fy}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\ Mn$ = 52.79

Mlx = 2.8474 kN

Mlx < Md (SAFE)

Field Reinforcement in Y Direction:

$K = \frac{Mly}{\omega b d^2} = \frac{3.387168 \times 10^6}{1 \times 300 \times 375^2} = 0.08\ MPa$

A $= \left(1 - \sqrt{1 - \frac{2K}{0.85 fc'}}\right) = \left(1 - \sqrt{1 - \frac{2 \times 0.08}{0.85 \times 30}}\right)$ = 3.142 mm

Main Reinforcement

As ; $\frac{0.85 a \times b \times fc}{fy} = \frac{0.85 \times 3.142 \times 300 \times 30}{240}$ = 100.15 mm2

Because fc' < 31.36 MPa, then:

As.u = $\frac{1.4 \times b \times d}{fy}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\ Mn$ = 52.79

Mly = 3.387168 kN

Mly < Md (SAFE)

Support Reinforcement in X Direction

$K = \frac{Mtx}{\omega b d^2} = \frac{2.8474 \times 10^6}{1 \times 300 \times 375^2} = 0.067\ MPa$

A $= \left(1 - \sqrt{1 - \frac{2K}{0.85 fc'}}\right)$ = 2.63 mm

Main Reinforcement

As ; $\frac{0.85a \times b \times fc}{fy} = \frac{0.85 \times 2.63 \times 300 \times 30}{240}$ = 83.83mm2

Because fc' < 31.36 MPa, then:

As.u = $\frac{1.4 \times b \times d}{fy}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\ Mn$ = 52.79

Mtx = 2.8474 kN

Mtx < Md (SAFE)

Support Reinforcement in Y Direction:

$K = \frac{Mty}{\omega b d^2} = \frac{3.387168 \times 10^6}{1 \times 300 \times 375^2} = 0.08\ MPa$

A $= \left(1 - \sqrt{1 - \frac{2K}{0.85fc'}}\right) = \left(1 - \sqrt{1 - \frac{2 \times 0.08}{0.85 \times 30}}\right)$ = 3.142 mm

Main Reinforcement

As ; $= \frac{0.85a \times b \times fc}{fy}$ = 100.15 mm2

Because fc' < 31.36 MPa, then:

As.u = $\frac{1.4 \times b \times d}{fy}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega \; Mn$ = 52.79

Mty = 3.387168 kN

Mty < Md (SAFE)

Slab E

Ly: 4

Lx: 2.5

Ly/Lx = 4/2.5 = 1.6

From Moment Table we got:

Clx: 58

Cly: 36

Ctx: 58

Cty: 36

Required Moments:

Mlx: $0.001 \times qu \times Lx^2 \times Clx = 0.001 \times 9.904 \times 2.5^2 \times 58 =$ 3.5902

Mly: $0.001 \times qu \times Ly^2 \times Cly = 0.001 \times 9.904 \times 4^2 \times 36 =$ 5.704704

Mtx: $0.001 \times qu \times Lx^2 \times Ctx = 0.001 \times 9.904 \times 2.5^2 \times 58 =$ 3.5902

Mty: $0.001 \times qu \times Ly^2 \times Cty = 0.001 \times 9.904 \times 4^2 \times 36 =$ 5.704704

Required Reinforcement for Slab E

Field Reinforcement in X Direction:

$K = \frac{Mlx}{\omega b d^2} = \frac{3.5902 \times 10^6}{1 \times 300 \times 375^2} = 0.085 \; MPa$

A $= \left(1 - \sqrt{1 - \frac{2K}{0.85 fc'}}\right) = \left(1 - \sqrt{1 - \frac{2 \times 0.085}{0.85 \times 30}}\right) =$ 3.34 mm

Main Reinforcement

As ; $= \frac{0.85 a \times b \times fc}{fy} =$ 106.46 mm2

Because fc' < 31.36 MPa, then:

As.u = $\frac{1.4 \times b \times d}{fy}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\ Mn$ = 52.79

Mlx = 5.704704 kN

Mlx < Md (SAFE)

Field Reinforcement in Y Direction:

$K = \frac{Mly}{\omega b d^2} = \frac{5.704704 \times 10^6}{1 \times 300 \times 375^2} = 0.135\ MPa$

A $= \left(1 - \sqrt{1 - \frac{2K}{0.85fc'}}\right) = \left(1 - \sqrt{1 - \frac{2 \times 0.135}{0.85 \times 30}}\right)$ = 5.308 mm

Main Reinforcement

As ; $= \frac{0.85a \times b \times fc}{fy}$ = 169.19 mm2

Because fc' < 31.36 MPa, then:

As.u = $\frac{1.4 \times b \times d}{fy}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\ Mn$ = 52.79

Mly = 5.704704 kN

Mly < Md (SAFE)

Support Reinforcement in X Direction:

$$K = \frac{Mtx}{\omega b d^2} = \frac{3.5902 \times 10^6}{1 \times 300 \times 375^2} = 0.085\ MPa$$

$$A = \left(1 - \sqrt{1 - \frac{2K}{0.85 fc'}}\right) = \left(1 - \sqrt{1 - \frac{2 \times 0.08}{0.85 \times 30}}\right) = 3.34 \text{ mm}$$

Main Reinforcement

As ; $\frac{0.85 a \times b \times fc}{fy}$ = 106.46 mm2

Because fc' < 31.36 MPa, then:

As.u = $\frac{1.4 \times b \times d}{fy}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 S}{4As} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 S}{4s} = \frac{\pi 10^2 \times \frac{1000}{656.25}}{4 \times 478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d - \left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\ Mn$ = 52.79

Mtx = 5.704704 kN

Mtx < Md (SAFE)

Field Reinforcement in Y Direction:

$$K = \frac{Mly}{\omega b d^2} = \frac{5.704704 \times 10^6}{1 \times 300 \times 375^2} = 0.135\ MPa$$

A $= \left(1-\sqrt{1-\frac{2K}{0.85fc'}}\right) = \left(1-\sqrt{1-\frac{2\times0.135}{0.85\times30}}\right)$ = 5.308 mm

Main Reinforcement

As ; $= \frac{0.85a\times b\times fc}{fy}$ = 169.19 mm2

Because fc' < 31.36 MPa, then:

As.u = $\frac{1.4\times b\times d}{fy}$ = 656.25 mm2

Biggest As = 656.25 mm2

Reinforcement Spacing:

S1: $\frac{\pi D^2 s}{4As} = \frac{\pi 10^2\times\frac{1000}{656.25}}{4\times656.25}$ = 478.476 mm

S2: 2 x h = 2 x 400 = 800 mm

Smallest S = 478 mm

As = $\frac{\pi D^2 s}{4s} = \frac{\pi 10^2\times\frac{1000}{656.25}}{4\times478}$ = 163.93 mm2

Main renforcement that'll be used is the one with specification d10-470 area 163.93 mm2

Design Moment Control

Minimal moment:

Mn: $As \times fy \times \left(d-\left(\frac{a}{2}\right)\right)$ = 656,25 x 240 x (375-(5.1/2)) = 58657169 kNm

Md = $\omega\ Mn$ = 52.79

Mty = 5.704704 kN

Mty < Md (SAFE)

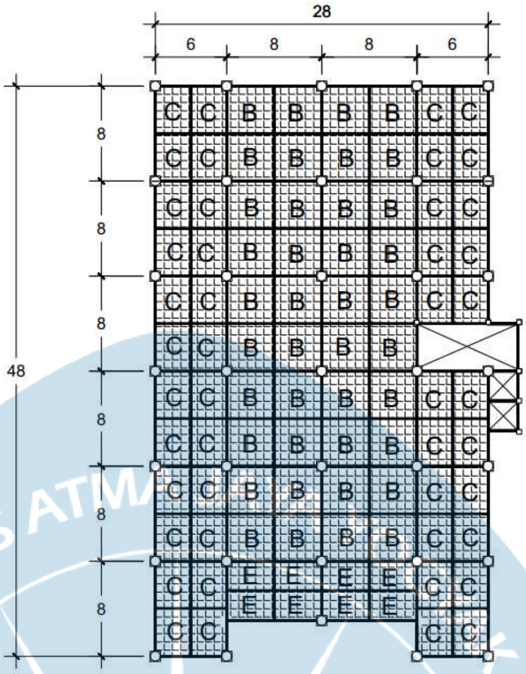

Figure 2.17. Slab Design First Floor

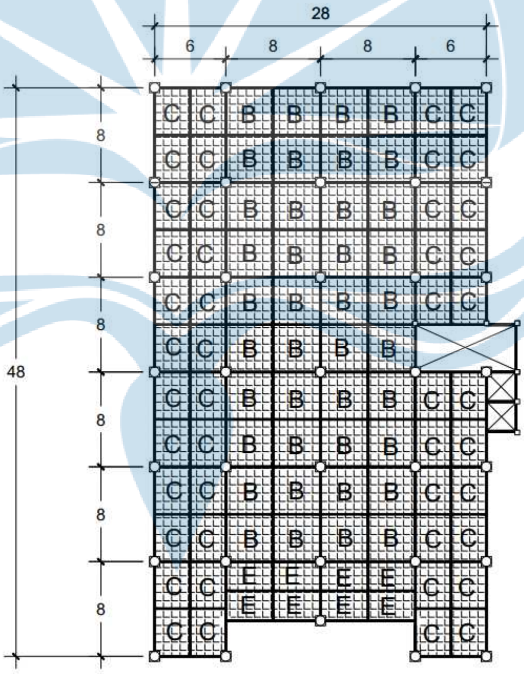

Figure 2.18. Slab Design Second Floor

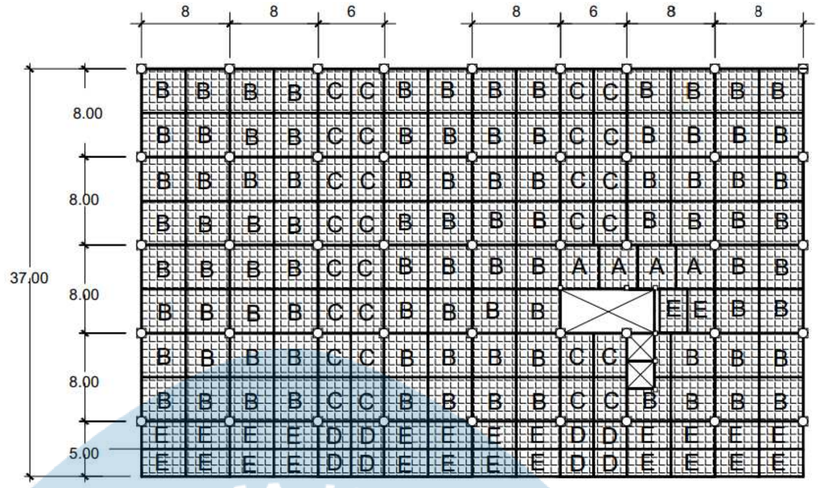

Figure 2.19. Slab Design Lower Ground

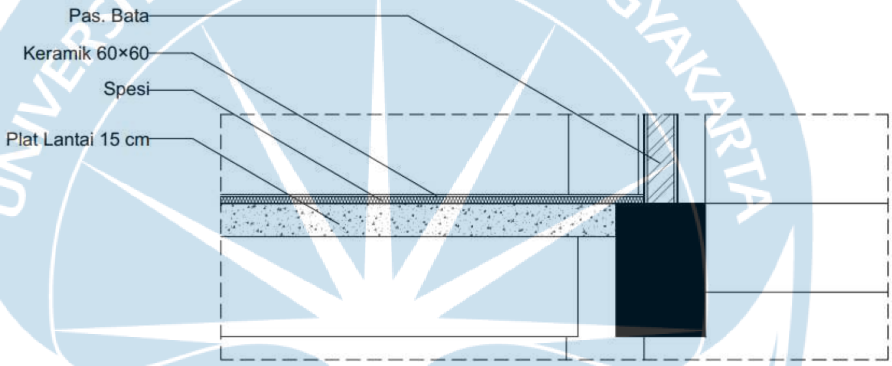

Figure 2.20. Slab Detailed Drawing

In Figure 2.17 – Figure 2.20, you can see the slab design on the lower ground, 1st floor, 2nd floor, and detailed slab drawings drawn using AutoCAD.

2.6. Beam Design

The beam design is based on SNI 1729-2015 Specifications for Structural Steel Buildings and SNI 2847-2013 Requirements for Structural Concrete for Buildings. This chapter presents the calculation/design procedures for WF 250 type beams. Furthermore, the design of other types of beams is carried out with the same calculation procedure (see Table 2.10 and Table 2.11).

Table 2.10. Flexible Design

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
Material Properties and Sections					
Beam Length, L			Inputs	mm	8000
Beam Width, b			Inputs	mm	400
Beam Height, h			Inputs	mm	600
Focus Length	21.5.3.1	18.6.4.1	2*p.s	mm	1200
Diameter of Longitudinal Reinforcement, d_b			Inputs	mm	19
Diameter of Transversal Reinforcement, d_{bt}			Inputs	mm	13
Diameter of stirrup Reinforcement, d_s			Inputs	mm	10
Clear Cover, c_c			Inputs	mm	30
Effective Height of Beams, d			$h - c_c - d_s - d_b/2$	mm	550,5
Concrete Compressive Strength, f_c'			Inputs	MPa	30
Yield Strength of Longitudinal Reinforcement, f_y			Inputs	MPa	420

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
Yield Strength of Transversal Reinforcement, f_y			Inputs	MPa	240
β_1	10.2.7.3	Table 22.2.2.4.3	0.65 <= 0.85 - 0.05 * (fc' - 28) / 7 <= 0.85		0,8357
Column Length, c_1			Input (Side perpendicular to the width of the beam)	mm	600
Column Width, c_2			Input (Affixed side of the beam/parallel to the width of the beam)	mm	600
L_n			L-c_1	mm	7400
λ			Assuming not using lightweight concrete		1
Internal Force					
$M_{u, focus}(-)$			Inputs	kN-m	-352,8871
$M_{u, focus}(+)$			Inputs	kN-m	253,2332
$M_{u, field}(-)$			Inputs	kN-m	-284,0134
$M_{u, field}(+)$			Inputs	kN-m	183,2446
pu			Inputs	kN	122,0987
Terms of Force and Geometry					

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
Axial Force Requirements	21.5.1.1	Not required. See R18.6.1 and 18.6.4.7	Pu <= 0.1 $A_g f_c'$?		OK
Effective High Terms	21.5.1.2	18.6.2.1	L_n>= 4d ?		OK
Width Requirements 1	21.5.1.3	18.6.2.1	b >= min(0.3h, 250 mm) ?		OK
Width Requirements 2	21.5.1.4	18.6.2.1	b <= c_2+2*min (c_2, 0.75c_1) ?		OK
Flexural Reinforcement					
Negative Support					
Total Support Negative Reinforcement, n			Inputs		7
d_b				mm	19
Clear Distance Between Rebars			(b - 2 c_c- 2 d_s- n * d_b) / (n - 1)	mm	31,167
Clear Distance Check	7.6.1	25.2.1	Clear Distance >= $d_b$and 25mm?		IYA
Number of Layers					1
As Pair			n*π/4 *d_b^2	mm^2	1984,70 1
$US_{min, 1}$	10.5.1	9.6.1.2	$(f_c')^{0.5}$/ (4 * f_y) *b *d	mm^2	717,908
$US_{admin, 2}$	10.5.1, 21.5.2.1	9.6.1.2	1.4 / (4 * f_y) *b *d	mm^2	734,000
Check As min			As Pair >= As min ?		OK

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
ρ			As / (b * d)		0,90%
$\rho_{max,1}$	B.10.3	There isn't any	$0.75\rho_b = 0.75 * 0.85 * \beta_1 * f_c' / f_y * (600/(600 + f_y))$		2,24%
$\rho_{max,2}$	21.5.2.1	18.6.3.1	2,5%		2,50%
Check As max			ρ <=ρmax ?		OK
a	10.2.7.1	22.2.2.4.1	As * fy / (0.85 * fc' * b)	mm	81,723
M_n	10.2.7.1	22.2.2.4.1	As * fy * (d - a/2)	kN-m	424,822
c	10.2.7.1	22.2.2.4.1	a/β1	mm	97,788
ε_s	10.2.2, 10.2.3	22.2.1.2, 22.2.2.1	(d - c) / c * 0.003		0,014
ϕ	S9.3.2	Table 21.2.2	0.65 <= 0.65 + (εs - 0.002) / 0.003 * 0.25 <= 0.9		0,900
ϕM_n			ϕ * Mn	kN-m	382,339
$M_{u, focus}(-)$				kN-m	352,887
Check Capacity			$\phi M_n > M_u$?		OK
As necessary			Mu / [fy * (d - a/2)]	mm^2	1648,634
Positive Support					
n			Inputs		5
d_b				mm	19

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
Clear Distance Between Rebars			$(b - 2\ c_c - 2\ d_s - n * d_b) / (n - 1)$	mm	56,250
Clear Distance Check	7.6.1	25.2.1	Clear Distance >= d_b and 25mm?		IYA
Number of Layers					1
As Pair			$n*\pi/4 * d_b^2$	mm^2	1417,644
$US_{min,\ 1}$	10.5.1	9.6.1.2	$(f_c')^{0.5} / (4 * f_y) * b * d$	mm^2	717,908
$US_{admin,\ 2}$	10.5.1, 21.5.2.1	9.6.1.2	1.4 / (4 * fy) * b * d	mm^2	734,000
$US_{admin,\ 4}$	21.5.2.2	18.6.3.2	0.5 * Negative Support Axle	mm^2	992,351
Check As min			As Pair >= As min ?		OK
ρ			As / (b * d)		0,64%
$\rho_{max,1}$	B.10.3		$0.75\rho_b = 0.75 * 0.85 * \beta_1 * f_c' / f_y * (600/(600 + f_y))$		2,24%
$\rho_{max,2}$	21.5.2.1	18.6.3.1	2,5%		2,50%
Check As max			$\rho <= \rho max$?		OK
a	10.2.7.1	22.2.2.4.1	As * fy / (0.85 * fc' * b)	mm	58,374
M_n	10.2.7.1	22.2.2.4.1	As * fy * (d - a/2)	kN-m	310,395
c	10.2.7.1	22.2.2.4.1	$a/\beta 1$	mm	69,849
ε_s	10.2.2, 10.2.3	22.2.1.2, 22.2.2.1	(d - c) / c * 0.003		0,021

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
ϕ	S9.3.2	Table 21.2.2	0.65 <= 0.65 + (εs - 0.002) / 0.003 * 0.25 <= 0.9		0,900
ϕM_n			ϕ * Mn	kN-m	279,356
M_u				kN-m	253,233
Check $\phi M_n > M_u$			$\phi M_n > M_u$?		OK
As necessary			Mu / [fy * (d - a/2)]	mm^2	1156,572
Negative Field					
n			Inputs		6
d_b				mm	19
Clear Distance Between Rebars			$(b - 2 c_c - 2 d_s - n * d_b) / (n - 1)$	mm	41,200
Clear Distance Check	7.6.1	25.2.1	Clear Distance >= d_b and 25mm?		IYA
Number of Layers					1
As Pair			$n*\pi/4 *d_b^2$	mm^2	1701,172
$US_{min, 1}$	10.5.1	9.6.1.2	$(f_c')^{0.5} / (4 * f_y) *b *d$	mm^2	717,908
$US_{admin, 2}$	10.5.1, 21.5.2.1	9.6.1.2	1.4 / (4 * fy) * b * d	mm^2	734,000
$US_{admin, 4}$	21.5.2.2	18.6.3.2	0.25 * Negative Support Axle	mm^2	496,175
Check As min			As Pair >= As min ?		OK
ρ			As / (b * d)		0,77%

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
$\rho_{max,1}$	B.10.3		$0.75\rho_b = 0.75 * 0.85 * \beta_1 * f_c' / f_y * (600/(600 + f_y))$		2,24%
$\rho_{max,2}$	21.5.2.1	18.6.3.1	2,5%		2,50%
Check As max			$\rho <= \rho max$?		OK
a	10.2.7.1	22.2.2.4.1	As * fy / (0.85 * fc' * b)	mm	70,048
M_n	10.2.7.1	22.2.2.4.1	As * fy * (d - a/2)	kN-m	368,304
c	10.2.7.1	22.2.2.4.1	a/β1	mm	83,818
ε_s	10.2.2, 10.2.3	22.2.1.2, 22.2.2.1	(d - c) / c * 0.003		0,017
ϕ	S9.3.2	Table 21.2.2	0.65 <= 0.65 + (εs - 0.002) / 0.003 * 0.25 <= 0.9		0,900
ϕM_n			ϕ * Mn	kN-m	331,473
M_u				kN-m	284,013
Check $\phi M_n > M_u$			$\phi M_n > M_{u\,?}$		OK
As necessary			Mu / [fy * (d - a/2)]	mm^2	1311,841
Positive Field					
n			Inputs		6
d_b				mm	19
Clear Distance Between Rebars			$(b - 2 c_c - 2 d_s - n * d_b) / (n - 1)$	mm	41,200

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
Clear Distance Check	7.6.1	25.2.1	Clear Distance >= $d_b$and 25mm?		IYA
Number of Layers					1
As Pair			$n*\pi/4 *d_b^2$	mm^2	1701,17 2
$US_{min, 1}$	10.5.1	9.6.1.2	$(f_c')^{0.5}/ (4 * f_y) *b *d$	mm^2	717,908
$US_{admin, 2}$	10.5.1, 21.5.2.1	9.6.1.2	1.4 / (4 * fy) * b * d	mm^2	734,000
$US_{admin, 4}$	21.5.2.2	18.6.3.2	0.25 * Negative Support Axle	mm^2	496,175
Check As min			As Pair >= As min ?		OK
ρ			As / (b * d)		0,77%
$\rho_{max,1}$	B.10.3		$0.75\rho_b= 0.75 * 0.85 *\beta_1*f_c' /f_y* (600/(600 + f_y))$		2,24%
$\rho_{max,2}$	21.5.2.1	18.6.3.1	2,5%		2,50%
Check As max			ρ <=ρmax ?		OK
a	10.2.7.1	22.2.2.4.1	As * fy / (0.85 * fc' * b)	mm	70,048
M_n	10.2.7.1	22.2.2.4.1	As * fy * (d - a/2)	kN-m	368,304
c	10.2.7.1	22.2.2.4.1	a/β1	mm	83,818
ε_s	10.2.2, 10.2.3	22.2.1.2, 22.2.2.1	(d - c) / c * 0.003		0,017
ϕ	S9.3.2	Table 21.2.2	0.65 <= 0.65 + (εs - 0.002) / 0.003 * 0.25 <= 0.9		0,900

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
ϕM_n			ϕ * Mn	kN-m	331,473
M_u				kN-m	183,245
Check $\phi M_n > M_u$			$\phi M_n > M_u$?		OK
As necessary			Mu / [fy * (d - a/2)]	mm^2	846,396

Table 2.11. Shear Design

Parameter	Reference Article		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
Material Properties and Sections					
Beam Length, L			From the Flex Design Sheet	mm	8000
Beam Width, b			From the Flex Design Sheet	mm	400
Beam Height, h			From the Flex Design Sheet	mm	600
Focus Length	21.5.3.1	18.6.4.1	From the Flex Design Sheet	mm	1200
Diameter of Longitudinal Reinforcement, d_b			From the Flex Design Sheet	mm	19
Diameter of Transversal Reinforcement, d_{bt}			From the Flex Design Sheet	mm	13
Diameter of stirrup reinforcement, d_s			From the Flex Design Sheet	mm	10
Clear Cover, c_c			From the Flex Design Sheet	mm	30

Parameter	Reference Article		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
Effective Height of Beams, d			From the Flex Design Sheet	mm	550,5
Concrete Compressive Strength, f_c'			From the Flex Design Sheet	MPa	30
Yield Strength of Longitudinal Reinforcement, f_y			From the Flex Design Sheet	MPa	420
Yield Strength of Transversal Reinforcement, f_y			From the Flex Design Sheet	MPa	240
β_1	10.2.7.3	Table 22.2.2.4.3	From the Flex Design Sheet		0,8357
Column Length, c_1			From the Flex Design Sheet	mm	600
Column Width, c_2			From the Flex Design Sheet	mm	600
L_n			From the Flex Design Sheet	mm	7400
Internal Force					
$V_{u,\ focus}$			Inputs	kN	213,0711
$V_{u,\ field}$			Inputs	kN	208,667
Support					
Design Force					
V_g, pedestal	S21.5.4	R18.6.5	Input [1.2 D + L Combination]	kN	146,2835
A_s^+pedestal			From the Flex Design Sheet	mm^2	1417,644
US^-pedestal			From the Flex Design Sheet	mm^2	1984,701
a_{pr}^+			1.25 a (positive support)	mm	72,967
a_{pr}^-			1.25 a (negative support)	mm	102,154

Parameter	Reference Article		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
M_{pr}^{+}	S21.5.4	R18.6.5	$A_s^{+} * (1.25f_y) * (d - a_{pr}^{+}/2)$	N mm	382563446
M_{pr}^{-}	S21.5.4	R18.6.5	$A_s^{-} * (1.25f_y) * (d - a_{pr}^{-}/2)$	N mm	520382976
V_{sways} or V_{pr}	21.5.4.1	18.6.5.1	$(M_{pr}^{+} + M_{pr}^{-}) / l_n$	N	122020
V_e	21.5.4.1	18.6.5.1	$V_g + V_{pr}$	N	268303
Concrete Shear Resistance					
V_{pr}				N	122020
$1/2V_e$				N	134152
pu				N	122098,7
Ag fc' / 20				N	360000
V_c reckoned?	21.5.4.2	18.6.5.2	$V_c = 0$ if $V_{pr} >= 1/2V_e$ and $P_u < A_g f_c' / 20$		Iya
V_c				N	205034
Shear Reinforcement					
Number of Feet			Inputs		4
Av			$n*\pi/4 *d_s^2$	mm^2	314,159
Space			Inputs	mm	100
Max Spacing 1	21.5.3.2	18.6.4.4	d / 4	mm	137,63
Max Spacing 2	21.5.3.2	18.6.4.4	6 d_b	mm	114,00
Max 3 Spaces	21.5.3.2	18.6.4.4	150mm	mm	150,00

Parameter	Reference Article		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
Check Space					OK
V_s	11.4.7.2	22.5.10.5.3	A_v*f_y* d / s	N	415067
V $limit_s$	11.4.7.9	22.5.1.2	0.66 * $(f_c')^{0.5}$*b*d	N	796016
ϕ	9.3.2.3	12.5.3.2, 21.2.4			0,75
V_n			V_c+ V_s	N	620102
V_u				N	268303
$\phi V_n/V_u$					1,733
Check Capacity			$\phi V_n/V_u$>= 1 ?		OK
Field					
Shear Reinforcement					
Number of Feet			Inputs		3
Av			n*π/4 *d_s^2	mm^2	235,619
Space			Inputs	mm	150
Spacing Max	21.5.3.4	18.6.4.6	d / 2	mm	275,25
Check Space					OK
V_s	11.4.7.2	22.5.10.5.3	A_v*f_y* d / s	N	207534
V $limit_s$	11.4.7.9	22.5.1.2	0.66 * $(f_c')^{0.5}$*b*d	N	796016
V_c	11.2.1.1	22.5.5.1	0.17 * $(f_c')^{0.5}$*b*d	N	205034

Parameter	Reference Article		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
ϕ	9.3.2.3	12.5.3.2, 21.2.4			0,75
V_n			$V_c + V_s$		412568
V_u				N	208667
$\phi V_n/V_u$					1,483
Check Capacity			$\phi V_n/V_u >= 1$?		OK

Table 2.12. Torque Design

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
Material Properties and Sections					
Beam Length, L			From the Flex Design Sheet	mm	8000
Beam Width, b			From the Flex Design Sheet	mm	400
Beam Height, h			From the Flex Design Sheet	mm	600
Focus Length	21.5.3.1	18.6.4.1	From the Flex Design Sheet	mm	1200
Diameter of Longitudinal Reinforcement, d_b			From the Flex Design Sheet	mm	19
Diameter of Transversal Reinforcement, d_{bt}			From the Flex Design Sheet	mm	13
Diameter of stirrup reinforcement, d_s			From the Flex Design Sheet	mm	10
Clear Cover, c_c			From the Flex Design Sheet	mm	30

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
Effective Height of Beams, d			From the Flex Design Sheet	mm	550,5
Concrete Compressive Strength, f_c'			From the Flex Design Sheet	MPa	30
Yield Strength of Longitudinal Reinforcement, f_y			From the Flex Design Sheet	MPa	420
Yield Strength of Transversal Reinforcement, f_y			From the Flex Design Sheet	MPa	240
β_1	10.2.7.3	Table 22.2.2.4.3	From the Flex Design Sheet		0,8357
Column Length, c_1			From the Flex Design Sheet	mm	600
Column Width, c_2			From the Flex Design Sheet	mm	600
L_n			From the Flex Design Sheet	mm	7400
Parameters of Sectional Geometry for Torque Calculation					
A_{cp}			b*h	mm^2	240000
P_{cp}			2*(b+h)	mm	2000
x_o			$b - 2c_c - d_s$	mm	330
y_o			$h - 2c_c - d_s$	mm	530
A_{oh}		R22.7.6.1.1	x_o*y_o	mm^2	174900
A_o	11.5.3.6	22.7.6.1.1	$0.85\ A_{oh}$	mm^2	148665
P_h		22.7.6.1	$2 * (x_o+y_o)$	mm	1720
Internal Force					
Q_u			Inputs	kN m	60,6077

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
Checking the Need for Torsion Reinforcement					
Q_{cr}			$0.33 * (f_c')^{0.5} * A_{cp}^2/P_{cp}$	N mm	52055552
ϕ	9.3.2.3	Table 21.2.1			0,75
$\phi\ T_{cr}/ 4$				N mm	9760416
Need Torsion Reinforcement?	11.5.1	Table 22.7.4.1	$Q_u>\phi T_{cr}/ 4$?		OK
Checking Adequacy of Cross Section Dimensions					
Torque Type			Certain Static = Equilibrium, Indeterminate Static = Compatibility		Compatibili ty
Q_uUse	11.5.2.2	22.7.3.2, 22.7.5	$\phi\ T_{cr}$ or T_u	N mm	39041664
V_u			From the Slide Design Sheet	N	268303
V_c	11.2.1.1	22.5.5.1	$0.17 * (f_c')^{0.5} * b * d$	N	205034
Utimate Shear Stress+Torque	11.5.3.1	22.7.7.1	$\{ [V_u/b*d]^2+[T_uP_h/ (1.7\ A_{oh}^2)]^2\}^{0.5}$	MPa	1,775
Concrete Stress Capacity	11.5.3.1	22.7.7.1	$\phi * \{ [V_c/ (b * d)] + 0.66 * (f_c')^{0.5}\}$	MPa	3,410
Check the Dimensions of the Cross Section	11.5.3.1	22.7.7.1	Left Side <= Right Side ?		OK
Other Common Parameters					

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
f_y/ f_{yt}			Yield Strength of Torsion Reinforcing Steel = Yield Strength of Bending and Shear Reinforcing Steel		1
θ	11.5.3.6	22.7.6.1.2	θ is taken for beams of non-prestressed members	°	45
Torsional Transversal Reinforcement					
n foot pedestal			From the Slide Design Sheet		4
n Court feet			From the Slide Design Sheet		3
s Focus			From the Slide Design Sheet	mm	100
s Field			From the Slide Design Sheet	mm	150
smax 1	11.5.6.1	9.7.6.3.3	P_h/ 8	mm	215
s max 2	11.5.6.1	9.7.6.3.3	300mm	mm	300
Check Focus Spacing			s Focal >= s max ?		OK
Check Field Spacing			s Field >= s max ?		OK
A_{v+t}/ s Focus Mount			$n*\pi/4 *d_s^2/ s$	mm^2/mm	3,142
A_{v+t}/ s Focus Mount			$n*\pi/4 *d_s^2/ s$	mm^2/mm	1,571
A_t/ s	11.5.3.6	22.7.6.1	$Q_u/ (2 *\varphi* A_o*f_y)$	mm^2/mm	0,417

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
A_v/ s Focus Required			$(V_u pedestal / \varphi - V_c) / (f_y * d)$	mm^2/mm	1,156
A_v/s Required Field			$(V_u Field / \varphi - V_c) / (f_y * d)$	mm^2/mm	0,554
A_{v+t}/ s Focus Required	11.5.5.2	R9.5.4.3	$2 * A_t/s + A_v/ s$		1,989
A_{v+t}/s Required Field	11.5.5.2	R9.5.4.3	$2 * A_t/s + A_v/ s$		1,388
A_{v+t}/s min 1	11.5.5.2	9.6.4.2	$0.062 * (f_c')^{0.5} * b/f_y$		0,323
A_{v+t}/ s min 2	11.5.5.2	9.6.4.2	$0.35 * w/f_y$		0,333
Check Shear + Focus Torque			A_{v+t}/s Install >= A_{v+t}/ s Need and min ?		OK
Check Shear + Torque Pitch			A_{v+t}/s Install >= A_{v+t}/ s Need and min ?		OK
Torsional Longitudinal Reinforcement					
$d_b$or d_{bt}				mm	13
d_b, min	11.5.6.2	9.7.5.2	0.042 s	mm	6,3
Check d_b			d_b>= $d_b$min?		OK
Ace Needs Top Support			From the Slide Design Sheet	mm^2	1648,634
Ace Needs Bottom Support			From the Slide Design Sheet	mm^2	1156,572
Ace Needs Upper Court			From the Slide Design Sheet	mm^2	1311,841
Ace Needs Down Court			From the Slide Design Sheet	mm^2	846,396
A_l	11.5.3.7	22.7.6.1	$A_t/s * P_h$	mm^2	716,981

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
$A_l min$	11.5.5.3	9.6.4.3	$0.42 * (f_c')^{0.5} * A_{cp} / f_y - (A_t/s) * P_h$	mm^2	597,553
A_s+A_lNeed Focus				mm^2	3522,187
A_s+A_lNeed Field				mm^2	2875,218
n Top Pedestal			From the Flex Design Sheet		7
n Central fulcrum			Input (Suggested Multiples of 2)		2
n Base			From the Flex Design Sheet		5
n Vertical support			2 + n Middle / 2		3
n Upper Court			From the Flex Design Sheet		6
n Midfield			Input (Suggested Multiples of 2)		2
n Upper Court			From the Flex Design Sheet		6
n Vertical support			2 + n Middle / 2		3
Focus Horizontal Spacing			$(b - 2c_c - 2d_s - d_b)$ / [min(n up, n down) - 1]	mm	75
Focus Vertical Spacing			$(h - 2c_c - 2d_s - d_b)$ / (n Vertical - 1)	mm	251
Field Horizontal Spacing			$(b - 2c_c - 2d_s - d_b)$ / [min(n up, n down) - 1]	mm	60
Field Vertical Spacing			$(h - 2c_c - 2d_s - d_b)$ / (n Vertical - 1)	mm	251
Check the Support Longitudinal Reinforcement Spacing	11.5.6.2		Spacing >= 300 mm ?		OK

Parameter	Reference		Equality	Unit	Mark
	SNI 2847:2013	SNI 2847:2019			
Check Field Longitudinal Reinforcement Spacing	11.5.6.2		Spacing >= 300 mm ?		OK
A_s+A_lInstall Focus				mm^2	3667,809
A_s+A_lInstall Field				mm^2	3667,809
Bend + Torque Check			A_s+A_lInstall >= A_s+A_lNeed ?		OK
Check Flexure + Pitch Torsion			A_s+A_lInstall >= A_s+A_lNeed ?		OK

Beam Profile

Table 2.13 is a conclusion and the type of reinforcement of beam that will be used based on the calculation results.

Table 2.13. Beam Profile Summary

Conclusion	
Terms of Force and Geometry	OK
Flexural Capacity	OK
Sliding Capacity	OK
Torque Capacity	OK
Longitudinal Reinforcement	
Upper Support Longitudinal	8 D22
Centerline Longitudinal	8 D22
Lower Support Longitudinal	8 D22
Upper Court Longitudinal	8 D22
Midfield Longitudinal	8 D22
Lower Field Longitudinal	8 D22
Transversal reinforcement / stirrups	
Focus bracket	2D16-100
Field stirrup	2D16-100

Beam Detail Engineering Design

Figure 2.21 – Figure 2.23 are the design of beam from lower ground until second floor drawn using AutoCAD. In Figure 2.24 and Figure 2.25 are main beam and secondary beam detailed drawing and Figure 2.26 is WF beam table.

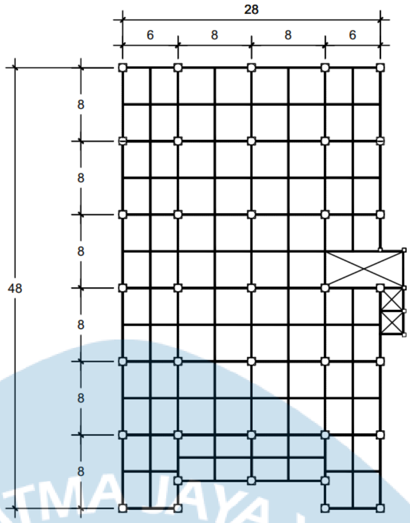

Figure 2.21. First Floor Beam Design

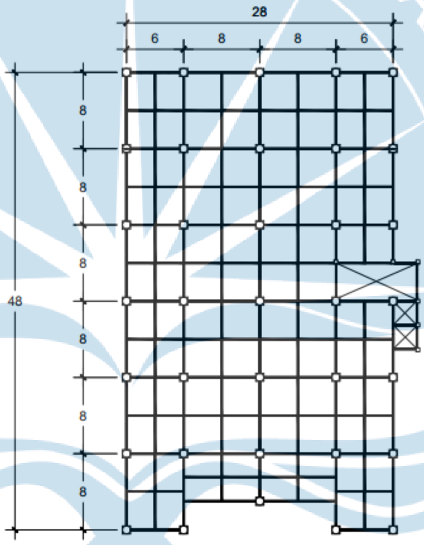

Figure 2.22. Second Floor Beam Design

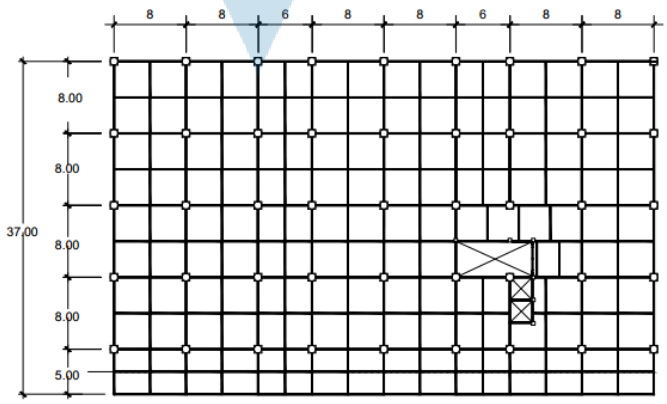

Figure 2.23. Lower Ground Beam Design

	B1 Tumpuan	B1 Lapangan
Sketch		
Dimension	600 x 400	650 x 400
Main Reinforcement	3D22	3D22
Stirrup	8D16	8D16

Figure 2.24. Main Beam Detail Drawing

	BA2 Tumpuan	BA2 Lapangan
Sketsa		
Dimensi	350 x 400	350 x 400
T.utama	3D16	2D16
Sengkang	8D16	8D16

Figure 2.25. Secondary Beam Detail Drawing

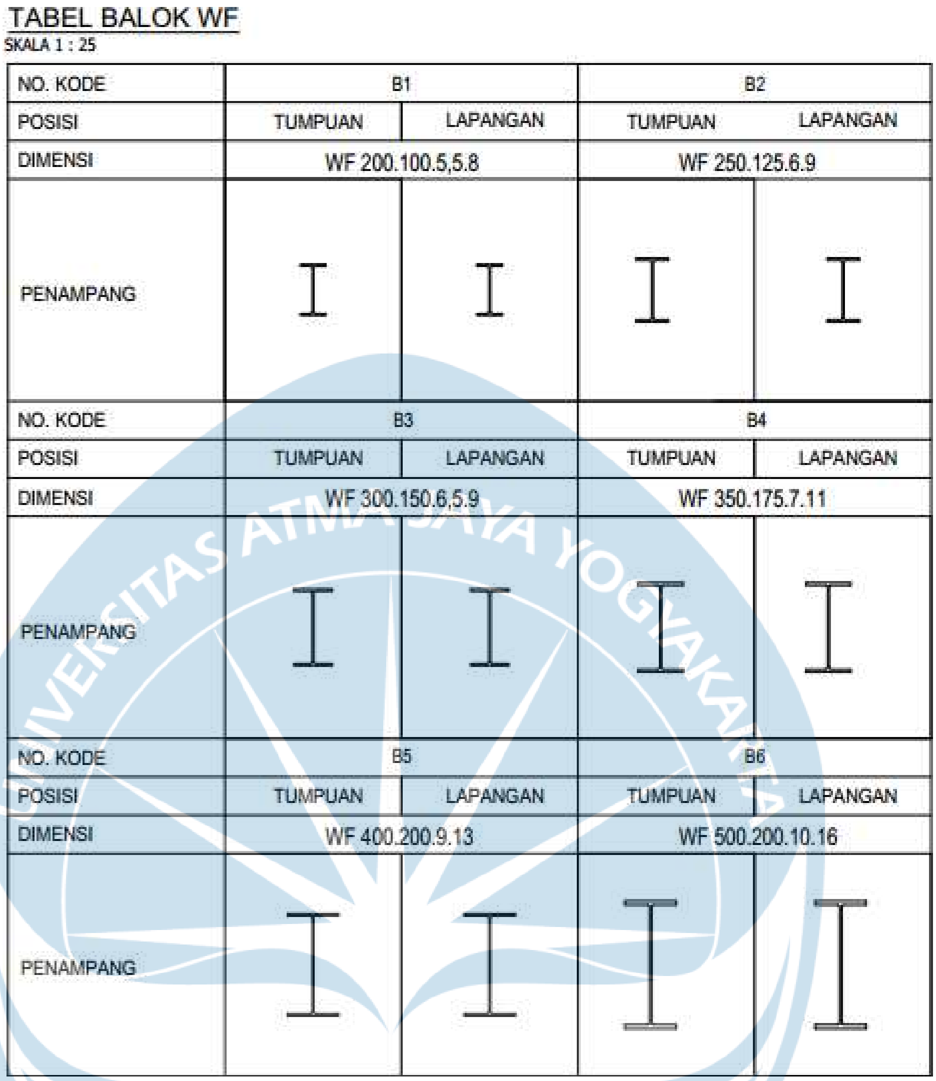

Figure 2.26. WF Beam Table

2.7. Column Design

Column design is carried out based on SNI 1729-2015 Specifications for Structural Steel Buildings. This chapter presents the calculation/design procedure for the H300 type column. Furthermore, the design of other types of columns is carried out with the same calculation procedure.

Column Internal Force

Table 2.14 shows the internal force column in which there is axial - flex, least compressive force, shear support and field.

Table 2.14. Column Internal Force

Axial - Flex			
Condition	P(kN)	M2 (kN-m)	M3 (kN-m)
P max	135,495	9,0303	0,744

P min	-2180,53	-0,2519	-2,897
M2 Max	-448,874	305,8844	76,6831
M2 Min	-1412,68	-236,3928	-155,3826
M3 Max	-181,438	44,0205	233,55
M3 Min	-366,334	-152,7743	-173,9557

Least Compressive Force			
Nu(kN)	-0,08		

Shear	
Support	
V2 (kN)	-258,722
V3 (kN)	-520,122
Field	
V2 (kN)	-258,722
V3 (kN)	-520,122

Longitudinal Design

Tables 2.15 - 2.16 sequentially are the results of calculations from the longitudinal design and transverse design.

Table 2.15. Longitudinal Design

Parameter	Reference		Quantity	Unit	Total
	SNI 2847:2013	SNI 2847:2019			
Material Properties and Sections					
Column Length/Height, L			Inputs	mm	4250
Short Side of Column, b			Inputs	mm	600
Long Side of Column, h			Inputs	mm	600
Diameter of Longitudinal Reinforcement, d_b			Inputs	mm	29
Diameter of stirrup reinforcement, d_s			Inputs	mm	13
Clear Cover, c_c			Inputs	mm	40
Concrete Compressive Strength, f_c'			Inputs	MPa	28
Yield Strength of Reinforcing Steel, f_y			Inputs	MPa	420
Beam Height, hb			Inputs	mm	600
L_n			L-hb	mm	3650
Terms of Force and Geometry					
Axial Force Requirements	21.6.1	Not required. See R18.7.1	Pu > 0.1 $A_g f_c'$?		OK
Shortest Side Terms	21.6.1.1	18.7.2.1	b >= 300mm ?		OK
Sectional Dimension Ratio Requirements	21.6.1.2	18.7.2.1	b/h >= 0.4 ?		OK
Checking the Force in Axial-Bending (Using PCA Column, or SP Column, or CSI Column, etc.)					
Number of Reinforcement, n			Inputs		20
Longitudinal Reinforcement Area, A_s			$n*\pi/4*d_b^2$	mm^2	13210,4
Rebar Ratio,ρ			As / (b * h)		2,06%

Parameter	Reference		Quantity	Unit	Total
	SNI 2847:2013	SNI 2847:2019			
Check ρ_{min} and ρ_{max}	21.6.3.1	18.7.4.1	1%<=ρ<= 6%		OK
Checking Strong Column - Weak Beam (SCWB)					
Column Nominal Moment, M_{nc}			Input (smallest value)	kN m	445,245
M_n^- Beam Focus			Inputs	kN m	424,822
M_n^+ Beam Focus			Inputs	kN m	310,395
Check SCWB	21.6.2.2	18.7.3.2	$2 * M_{nc} >= 1.2 * (M_n^- + M_n^+)$		OK

Transversal Design

Table 2.16. Transversal Design

Parameter	Reference		Quantity	Unit	Total
	SNI 2847:2013	SNI 2847:2019			
Material Properties and Sections					
Column Length/Height, L			From the Longitudinal Design Sheet	mm	4250
Short Side of Column, b			From the Longitudinal Design Sheet	mm	600
Long Side of Column, h			From the Longitudinal Design Sheet	mm	600
Diameter of Longitudinal Reinforcement, d_b			From the Longitudinal Design Sheet	mm	29
Diameter of Stirrup Reinforcement, d_s			From the Longitudinal Design Sheet	mm	13
Clear Cover, c_c			From the Longitudinal Design Sheet	mm	40
Concrete Compressive Strength, f_c'			From the Longitudinal Design Sheet	MPa	28
Yield Strength of Reinforcing Steel, f_y			From the Longitudinal Design Sheet	MPa	420
Beam Height, h_b			From the Longitudinal Design Sheet	mm	600
L_n			From the Longitudinal Design Sheet	mm	3650
Plastic Joint Zone Length					
l_{o1}	21.6.4.1	18.7.5.1	h	mm	800,0

Parameter	Reference		Quantity	Unit	Total
	SNI 2847:2013	SNI 2847:2019			
l_{o2}	21.6.4.1	18.7.5.1	$L_n/ 6$	mm	608,3
l_{o3}	21.6.4.1	18.7.5.1	450mm	mm	450
l_o	21.6.4.1	18.7.5.1	Max (l_{o1}; l_{o2}; l_{o3})	mm	800,0

Plastic Hinge/Support Zone Transversal Reinforcement					
Number of Short Side Legs, n1			Inputs		4
Number of Long Side Legs, n2			Inputs		4
Space, s			Inputs	mm	100
Largest Foot Spacing, $x_{i\ max}$	S21.6.4.2	R18.7.5.2	Inputs	mm	300
$A_{sh}1$			$n*\pi/4 *d_s^2$	mm^2	530,929
$A_{sh}2$			$n*\pi/4 *d_s^2$	mm^2	530,929
A_{sh}/ s, 1				mm^2/mm	5,309
A_{sh}/ s, 2				mm^2/mm	5,309
***Confinement*/Plastic Joint Zone Restraints**					
Concrete Core Cross-sectional Width, b_c	S21.6.4.2	R18.7.5.2	b - $2c_c$	mm	720
Length of Concrete Core Cross Section, h_c	S21.6.4.2	R18.7.5.2	h - $2c_c$	mm	720
Column Cross-sectional Area, A_g			b*h	mm^2	640000
Concrete Core Cross-sectional Area, A_{ch}			$b_c* h_c$	mm^2	518400
Short Side/Weak Axis					
A_{sh}/s min, 1	21.6.4.4	18.7.5.4	$0.3 (b_c*f_c' /f_y) * (A_g/ A_{ch}- 1)$	mm^2	3,378
A_{sh}/s min, 2	21.6.4.4	18.7.5.4	$0.09*b_c*f_c' /f_y$	mm^2	4,320
Check A_{sh}/s 1			A_{sh}/s 1 >= Ash/s min ?		OK
Long Side/Strong Axis					
A_{sh}/s min, 1	21.6.4.4	18.7.5.4	$0.3 (p_c*f_c' /f_y) * (A_g/ A_{ch}- 1)$	mm^2	3,378

Parameter	Reference		Quantity	Unit	Total
	SNI 2847:2013	SNI 2847:2019			
A_{sh}/s min, 2	21.6.4.4	18.7.5.4	$0.09*h.s_c*f_c' / f_y$	mm^2	4,320
Check A_{sh}/s 2			A_{sh}/s 2 >= Ash/s min ?		OK
Check Space					
$s_{max,1}$	21.6.4.3	18.7.5.3	b / 4	mm	200
$s_{max,2}$	21.6.4.3	18.7.5.3	6 * d_b	mm	174
h_x	21.6.4.3	18.7.5.3	$x_{i\ max}$	mm	300
$s_{max,3}$= s_o	21.6.4.3	18.7.5.3	100 <= 100 + (350 - hx) / 3 <= 150	mm	116,667
s_{max}	21.6.4.3	18.7.5.3	Min (s_{max1}, s_{max2}, s_{max3})	mm	116,667
Check Space					OK
Shear Strength of Plastic Joint Zone					
Design Shear Force (Requires input from PCA Column, or SP Column, or CSI Column, etc. with f_{pr}= 1.25f_y)					
M_{pr}Column			Input, (largest value)	kN m	410,322
$V_{u\ 1}$	S21.5.4	18.7.6.1	2 * M_{pr}Column / Ln	N	224834
Structural Analysis Results of Shear Forces					
$V_{u\ 2}$, Weak Axis			From the Inner Force Sheet	N	258722
$V_{u\ 2}$, Strong Axis			From the Inner Force Sheet	N	520122
Weak Axis Concrete Shear Resistance					
V_u			Max (V_{u1}, V_{u2})	N	258722
φ	9.3.2.3	Table 21.2.1			0,75
V_c	11.2.1.2	22.5.6.1	$0.17 (1 + N_u/(14 A_g)] (f_c')^{0.5}hd$; $d = b - c_c - d_s - d_b/ 2$	N	527144
V_sNeed	11.1.1	22.5.10.1	$V_u/φ - V_c$	N	-182181
A_s/s Required	11.4.7.2	22.5.10.5.3	$V_s/ (f_y*d)$; $d = b - c_c - d_s - d_b/ 2$	mm^2/mm	-0,5922
A_s/s Min 1	-	10.6.2.2	$0.062 (f_c')^{0.5}h/f_y$	mm^2/mm	0,6249
A_s/s Min 2	-	10.6.2.2	$0.35h/f_y$	mm^2/mm	0,6667

Parameter	Reference		Quantity	Unit	Total
	SNI 2847:2013	SNI 2847:2019			
Check A_s/s			A_{sh}/s 1 >= Max(A_s/s Necessary, A_s/s Min) ?		OK
Strong Axis Concrete Shear Resistance					
V_u			Max (V_{u1}, V_{u2})	N	520122
φ	9.3.2.3	Table 21.2.1			0,75
V_c	11.2.1.2	22.5.6.1	$0.17\ (1 + N_u/(14\ A_g))\ (f_c')^{0.5}bd$; $d = h - c_c - d_s - d_b/2$	N	527144
V_sNeed	11.1.1	22.5.10.1	$V_u/\varphi - V_c$	N	166351
A_s/s Required	11.4.7.2	22.5.10.5.3	$V_s/(f_y*d)$; $d = h - c_c - d_s - d_b/2$	mm^2/mm	0,5407
A_s/s Min 1	-	10.6.2.2	$0.062\ (f_c')^{0.5}b / f_y$	mm^2/mm	0,6249
A_s/s Min 2	-	10.6.2.2	$0.35w/f_y$	mm^2/mm	0,6667
Check A_s/s			A_{sh}/s 2 >= Max(A_s/s Necessary, A_s/s Min) ?		OK

Outer Transversal Reinforcement Zone of Plastic Joints/Abutments					
Number of Short Side Legs, n1			Inputs		2
Number of Long Side Legs, n2			Inputs		2
Space, s			Inputs	mm	150
A_vWeak Axis			$n*\pi/4*d_s^2$	mm^2	265,465
A_vStrong Axis			$n*\pi/4*d_s^2$	mm^2	265,465
***Confinement*/Outer Bridging of the Plastic Joint Zone**					
Max spacing 1	21.6.4.5	18.7.5.5	$6d_b$	mm	174,0
Max 2 spacing	21.6.4.5	18.7.5.5	150mm	mm	150,0
Check Space			Spacing <= Max Spacing ?		OK
Outer Shear Strength of Plastic Joint Zone					
Weak Axis Concrete Shear Resistance					
V_u			From the Inner Force Sheet	N	258722
φ	9.3.2.3	Table 21.2.1			0,75

Parameter	Reference		Quantity	Unit	Total
	SNI 2847:2013	SNI 2847:2019			
V_c	11.2.1.2	22.5.6.1	$0.17\ (1 + N_u/(14\ A_g))\ (f_c')^{0.5}hd$; $d = b - c_c - d_s - d_b/2$	N	527144
V_sNeed	11.1.1	22.5.10.1	Max $(V_u/\varphi - V_c; 0)$		0
A_v/s Required	11.4.7.2	22.5.10.5.3	$V_s/(f_y{*}d)$; $d = b - c_c - d_s - d_b/2$		0,0000
A_s/s Min 1	-	10.6.2.2	$0.062\ (f_c')^{0.5}b / f_y$	mm^2/mm	0,0000
A_s/s Min 2	-	10.6.2.2	$0.35w/f_y$	mm^2/mm	0,0000
Check A_s/s			$A_v/s >= A_v/s$ Need ?		OK
Strong Axis Concrete Shear Resistance					
V_u			From the Inner Force Sheet	N	520122
φ	9.3.2.3	Table 21.2.1			0,75
V_c	11.2.1.2	22.5.6.1	$0.17\ (1 + N_u/(14\ A_g))\ (f_c')^{0.5}bd$; $d = h - c_c - d_s - d_b/2$	N	527144
V_sNeed	11.1.1	22.5.10.1	Max $(V_u/\varphi - V_c; 0)$		166351
A_v/s Required	11.4.7.2	22.5.10.5.3	$V_s/(f_y{*}d)$; $d = h - c_c - d_s - d_b/2$		0,5407
A_s/s Min 1	-	10.6.2.2	$0.062\ (f_c')^{0.5}b / f_y$	mm^2/mm	0,6249
A_s/s Min 2	-	10.6.2.2	$0.35w/f_y$	mm^2/mm	0,6667
Check A_s/s			$A_v/s >= A_v/s$ Need ?		OK

Column Profile

Table 2.17 is a conclusion and the type of reinforcement of column that will be used based on the calculation results.

Table 2.17. Column Profile Summary

Conclusion	
Terms of Force and Geometry	OK
Flexural Capacity	OK
Sliding Capacity	OK
Longitudinal Reinforcement	
Longitudinal	16D22
Transversal Reinforcement/Support stirrups	
Weak Axis	4D13-100
Strong Axis	4D13-100
Transversal Reinforcement/Field Stirrup	
Weak Axis	4D13-100
Strong Axis	4D13-100