CHAPTER II
UPPSTRUCTURE DESIGN

2.1.Introduction

The Designing of the upper structure of the Nursing Home building will start with the Designing of the roof and will be followed by the Designing of the structure underneath. This process is carried out with the aim of knowing the load that occurs on the roof and the amount of load given to the structure underneath. In this project, known data related to the initial design or preliminary design. The initial design data is the result of the initial structural design determined by the architectural design which will then be analyzed to obtain an efficient and robust design. The data in the preliminary design will later be calculated and analyzed to check the efficiency and strength of the design and changes will be made if the structural capability does not meet the expected minimum capacity.

The superstructure Designing will be based on SNI (Indonesian National Standardization) which will serve as a guide in Designing each structure. The following is the SNI guide used.

a. SNI 2847:2019 concerning Structural Concrete Requirements for Buildings and Explanations
b. SNI 1727:2020 concerning Minimum Design Load and Related Criteria for Buildings and Other Structures
c. SNI 1729:2020 concerning Procedures for Designing Steel Structures for Buildings
d. SNI 1726:2019 concerning Procedures for Designing Earthquake Resistance for Building and Non-Building Structures
e. Steel Specifications from PT. Mount Garuda

2.2.Used Materials

2.2.1. Roof

The roof plan on the building is a very important part because of the function and aesthetics of the building in question and the load that will be borne by the structure underneath. In Designing the roof, it is necessary to consider the Designing of the Purlin and the Designing of the trusses. The roof plan for the Nursing Home building can be seen in the AutoCAD Drawing Appendix and analyzed using SAP2000 to look for reactions from the existing styles.

Table 1. Designing of Roof

Information	Purlin	Concrete	Beam Reinforcement	Column Reinforcement
Material	Steel	Concrete	Steel	Steel
Steel Tension (fy)	240 MPa	-	420 MPa	420 MPa
Tension ultimate (fu)	370 MPa	-	-	-
Concrete strength (fc')	-	25 MPa	-	-
Modulus elasticity (E)	200,000 MPa	4700 x fc' = 117,500 MPa	200,000 MPa	200,000 MPa

The Nursing Home building has 2 types of roofs, namely the Joglo roof which is used in the living room and the pyramid roof in the rooms. Both types of roofs have the same material plan, the only difference being the long span and slope of the roof. Next is the explanation below:

Roof for Rooms (Pyramid Roof)

The span on the roof of the room is 7.8m. The roof will use a profile of 2L 50x50x5. The upper structure used in the Nursing Home building in the rooms is a single WF profile.

Purlin Designing

Purlin or Purlin is a roof component that is located on the truss which functions to support the load that occurs on the roof. In Designing the curtains of this building, steel material was used with curtain specifications obtained from Steel Specifications from PT. Mount Garuda with roof data in Table

Table 2. Purlin Design (Pyramid Roof)

Know Data	Value	Unit	Information
Roof slope (α)	30	Degrees	-
Span Length roof (L1)	7,6	m	-
Curtain spacing (a)	120	mm	-
Purlin load	3.49	kN/m	Used material steel profile C100x50x20 (2 mm thickness)

Know Data	Value	Unit	Information
Roof load	4 kg/m2 x 9.8 x 1000 x 120 = 0.05133	kN/m	Used material light steel galvalume because it is strong withstand loads and do not rust easily
Ceiling load	19kg/m2 x 9.8 x 1000 x 120 = 0.22605	kN/m	PVC (Polyvinyl Chloride) material is used because it is more flexible, lighter and durable
Dead load (p)	(Calculation is below)	kN/m	Accumulation of all fixed loads on the building
live load (q)	1	kN	It's a workload

The following are the loads that occur and are imposed on the Purlin in the Designing of the roof structure which can be seen in :

a. Live loads (q) are loads caused by users and/or occupants of buildings which do not include construction loads and environmental loads, such as wind loads, rain loads, earthquake loads, flood loads or dead loads. In Purlin, the live load is the work load, which is 1 kN.

b. Dead load (p), is all the loads from all parts of the building that are fixed. In Purlin, the dead loads that work are self loads, ceiling loads, and roof loads.

Table 3. Profil c Spesification

METRIC SIZE

DIMENSION	THICKNESS	SECTION AREA	WEIGHT UNIT	GEOMETRICAL MOMENT OF INERTIA		MODULUS OF SECTION		RADIUS OF GYRATION		CENTER OF GRAVITY	SHEAR CENTER	TORSION CONSTANT	WARPING CONSTANT
H x B x C	t	A	kg/m	I_x	I_y	Z_x	Z_y	r_x	r_y	C_y	X_o	J	C_w
mm	mm	cm^2		cm^4	cm^4	cm^3	cm^3	cm	cm	cm	cm	cm^4	cm^6
C 100 x 50 x 20	2.0	4.54	3.56	71	17	14.3	5.4	3.97	1.93	1.87	4.48	605	444
	2.3	5.17	4.06	81	19	16.1	6.0	3.95	1.92	1.86	4.46	912	496
	2.5	5.59	4.39	87	20	17.3	6.5	3.94	1.90	1.86	4.45	1164	528
	2.8	6.20	4.87	95	22	19.1	7.1	3.92	1.89	1.86	4.42	1621	574
	3.0	6.61	5.19	101	23	20.2	7.4	3.91	1.88	1.86	4.41	1982	603
	3.2	7.01	5.50	106	24	21.3	7.8	3.90	1.87	1.86	4.40	2392	630

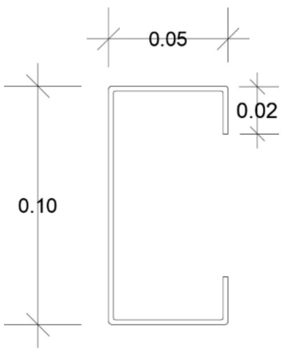

Figure 1. Purlin Mild Steel Profile C100x50x20

Purlin Designing will be carried out based on ULS and SLS parameters with the aim of minimizing expenses when Designing costs. In Designing Purlin, 2 pieces of checking so that the roof structure is safe are as follows.

a. Ultimate Limit State Design(ULS)

In this check, it will be reviewed based on the bending stress and shear stress where the stress that occurs (fb) must be less than the allowable stress (fy). In this check, the LRFD design is used where the stress is reduced by 0.9 (Φ) with the section modulus in the X and Y directions and the ultimate moment of the Purlin in the X and Y directions will depend on the type of Purlin used.

Through the loads previously obtained, the calculation of the ultimate moment of the Purlin in the X direction and Y direction is carried out with each combination. The ultimate Purlin moment used is the largest ultimate moment for the X and Y directions. The ultimate Purlin moment in the Y (M2,U) and X (M3,U) directions is used to obtain the resulting stress (fb) in the Purlin profile, which is then compared with the allowable steel stress (fy). The ultimate moment and steel stress use the formula in SNI 1729:2020. In Table 2.4, it is found that the applied stress, fb, is 216.785 MPa, while the allowable stress is 240 MPa so checking based on ULS is safe.

X-direction Purlin Moment Designing:

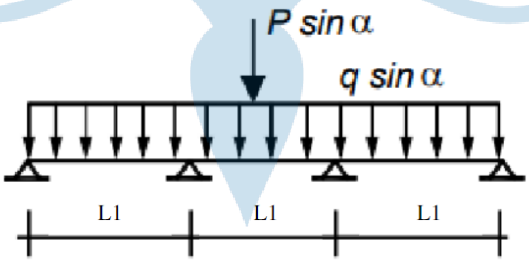

Figure 2. Moment Design in x Direction

$$M_{3,D} = \frac{1}{8} \times q \cos\alpha \, (L1)^2 \text{ (Moment Calculation in x direction due to Dead Load)}$$

$$M_{3,\,L} = +\frac{1}{4} \times p \cos\alpha \, L1 \text{ (Moment Calculation in x direction due to Live Load)}$$

$$M_{3,U} = 1{,}4M_{3,D} \text{ (Rumus 1 (Formula 1 x Direction Moment)}$$

$$M_{3,U} = 1{,}2M_{3,D} + 1{,}6M_{3,\,L} \text{ (Formula 2 x Direction Moment)}$$

*used the greatest moment between formula 1 and formula 2

Y-direction Purlin Moment Designing Formula:

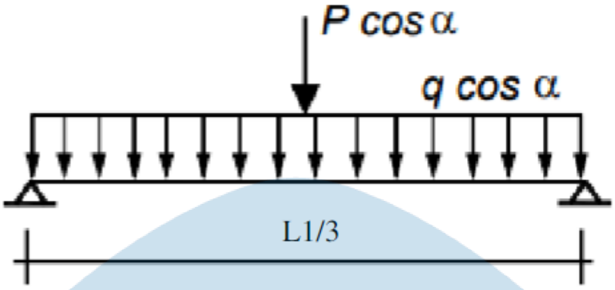

Figure 3. . Moment Design in y Direction

$M_{2.D} = \frac{1}{8} \times q \sin\alpha \left(\frac{L1}{3}\right)^2$ (Moment Calculation in y direction due to Dead Load)

$M_{2.L} = +\frac{1}{4} \times q \sin\alpha \frac{L1}{3}$ (Moment Calculation in y direction due to Live Load)

$M_{2.U} = 1.4M_{2.D}$ (Formula 1 y Direction Moment)

$M_{2.U} = 1.2M_{2.D} + 1.6M_{2.L}$ (Formula 2 y Direction Moment)

*used the greatest moment between formula 1 and formula 2

$f_b = \frac{M3.U}{\Phi W3} + \frac{M2.U}{\Phi W2}$(Stress Formula for Purlin Due to Moments x and y)

Table 4. ULS Purlin Check

Profile Purlin	Curtain Thickness (mm)	X Direction Ultimate Moment (kN)	Y Direction Ultimate Moment (kN)	Actual stress (MPa)	steel stress (fy) = 240 MPa	Safe: < fyi
C 100x50x20	2	0.52	0.23	162.51	240	Safe

ULS Purlin is considered safe if the stress on the steel (Fb) is less than the design steel strength (fy) in this case 162.51 MPa <240 MPa so it is safe.

b. Serviceability Limit State Design (SLS)

In this check, it will be reviewed from the deformation where the deformation that occurs must be smaller than the allowable deformation. In this plan, the deformation under consideration is based on the Purlin deflection (δ). Calculation of deflection using the following formula:

$$\delta x = \frac{5}{384}\frac{q\cos\ \alpha(L1)^4}{EI} + \frac{1}{48}\frac{p\cos\ \alpha(L1)^3}{EI} \text{ (Pyhtagoras Calculation x Direction)}$$

$$\delta y = \frac{5}{384}\frac{q\sin\ \alpha(L1)^4}{EI} + \frac{1}{48}\frac{p\sin\ \alpha(L1)^3}{EI} \text{ (Pyhtagoras Calculation y Direction)}$$

$$\delta = \sqrt{\delta_3^2 + \delta_2^2} \text{ (Deflection : SNI 1729: 2020)}$$

$$\delta = \frac{1}{240} x L1 \text{(Izi Deflection in Steel)}$$

Table 5. Purlin SLS Check

Purlin Profile	Curtain Thickness (mm)	Actual deflection(mm)	Allowable deflection(mm)	Safe:
C 100x50x20	2	5.9	12.5	Safe

The deflection that occurs is 5.9 mm smaller than the deflection limit of 12.5 mm so Steel profile C 100x50x20 with a thickness of 2 mm shows a safe number for the ULS and SLS tests can be used in the design of the Nursing Home building. The next step is to find the required sag-rod. Sag-rod is a connecting rod between one Purlin and another which serves to prevent the Purlin from curving. In the AutoCAD design drawing for the nursing home, it is found that the number of curtains under the roof is 8 lines (n) so that the area of the sag-rod can be found by combining the load of the sag-rod rods. The load combination will be used to find the minimum required sag-rod area after being reduced (Φ) by 0.9. The minimum required sag-rod area is 1.049 mm2.

$$F_{t,D} = n\left(\frac{L1}{3} q\sin\ \alpha\right) \text{ (Combination of Sagrod Capacity beacause of Dead Load)}$$

$$Ft, L = \left(\frac{n}{3} p\sin\ \alpha\right) \text{ (Combination of Sagrod Capacity beacause of Live Load)}$$

$$F_{t,U} = 1{,}4 Ft, D \text{ (Formula 1 Load Combination of Sagrod)}$$

$$F_{t,U} = 1{,}2\ F_{t,D} + 1{,}6\ F_{t,L} \text{ (Formula 2 Load Combination of Sagrod)}$$

$$A_{sr} = \frac{F_{t,U} \times 1000}{\Phi F_y} \text{ (Area of Sagrod Load)}$$

$$A_{sr} = \frac{1}{4}\pi d^2 \text{ (Capacity of Sagrod based on design materials)}$$

Based on the calculations, the smallest sag-rod diameter of 10 mm can be used to minimize the budget

Table 6. Sag-rod Designing

Sag rod diameter (mm)	Sag-rod area (1) (mm2)	Safe : (1) > 18.0313 mm2
10	104.88	Safe

For wind tie rods there is usually no detailed calculation, it is usually determined directly by considering the span and spacing of the trusses. In this case, the wind tie rod will use a diameter of 16 mm.

Designing of Main Truss

The main truss is the arrangement of the truss which functions to support the pressure on the roof truss and channel this pressure to the building structurethat's underneath. The following are the loads that occur and are imposed on the Main Trusss in the Designing of the roof structure.

Table 7. Main Truss Design

Known data	Value	Unit	Information
Roof slope (α)	30	o	-
span length roof (L1)	8	m	-
Roof height	3	m	Main Truss design
Structure height without roof	12,32	m	Structure design
Building height (h)	3 + 12.3 = 15.2	m	-
Roof length(L)	8	m	Main Truss Design
h/l	1,9	-	Ratio Height building and roof length
Distance between joints easel (a)	120	mm	-
The distance from the edge of the roof to the truss joint (b)	120	mm	-
The load of the Main Trusss	0.5	kN/m	It is assumed to be 0.5 kN/m from SNI 1729:2020
Purlin load	3.56 kg/m x 9.8 x 1000 = 0.03489	kN/m	Used material steel profile C100x50x20 with 2 mm thick

Roof load	4 kg/m2 x 9.8 x 1000 x 120 = 0.05133	kN/m	Galvalume mild steel material is used because it is strong enough to withstand loads and not easy to rust
Ceiling load	19kg/m2 x 9.8 x 1000 x 120 = 0.22605	kN/m	PVC (Polyvinyl Chloride) material is used because it is more flexible, lighter and durable
live load (q)	1	kN	It's a workload

a. Live loads are loads caused by users and/or occupants of buildings which do not include construction loads and environmental loads, such as wind loads, rain loads, earthquake loads, flood loads or dead loads. In the trusses, the live load is the working load, which is 1 kN.

b. Dead load, is all the loads from all parts of the building that are fixed. In the trusses, the dead loads that work are the trusses themselves, the curtain loads, the ceiling loads, and the roof loads.

Table 8. Calculation of Main Truss Dead Load

Dead Load Location	Formula	Yield (kN)
P1 load		
The weight of the Main Trusss	(a/2) x Weight	0.375
Purlin weight	L1 x Weight	0.104
roof weight	[(a/2 + b) / cosα] x L1 x Weight	0.305
Ceiling weight	(a/2 + b) x L1 x Weight	1,256
Total		2.107
P2 load		
The weight of the Main Trusss	ax Weight	0.750
Purlin weight	L1 x Weight	0.105
roof weight	[(a/2 + b) / cosα] x L1 x Weight	0.204
Ceiling weight	ax L1 x Weight	0.838
Total		1856
P3 load		
The weight of the Main Trusss	ax Weight	0.750
Purlin weight	2 x L1 x Weight	0.209
roof weight	(a/cosα) x L1 x Weight	0.204

Dead Load Location	Formula	Yield (kN)
Ceiling weight	ax L1 x Weight	0.838
Total		1996

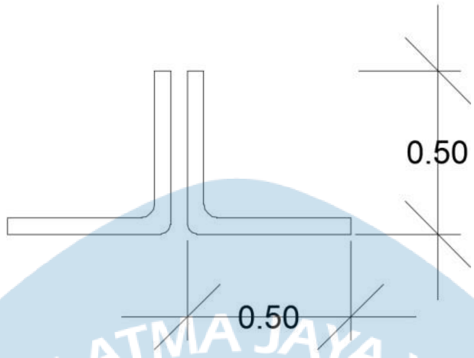

Figure 4. Main Truss Profiles 2L50x50x5

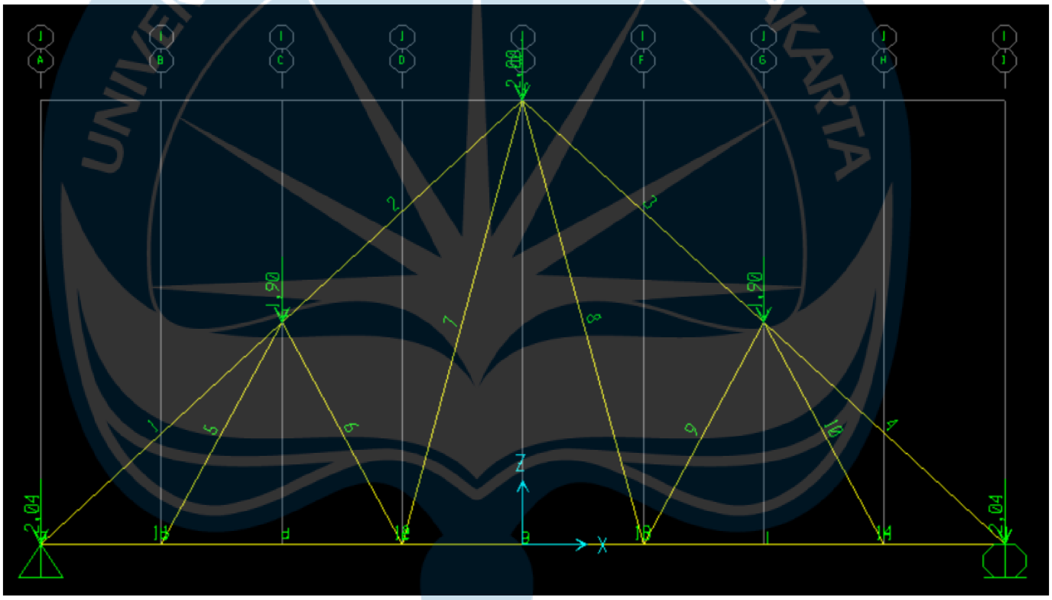

Figure 5. Projection of Main Truss Dead Load(SAP2000 Modeling)

Wind loads, are all loads acting on the building caused by air pressure. The wind load is determined by the amount of wind blowing (Qw), the wind coefficient (Cti), and the suction wind coefficient (Cis). Big gust of windassumed to be 0.25 based on SNI 1729:2002. Blowing winds are winds that go to the roof, while suction winds are winds that go away from the roof. There are six different wind loads according to their location on the truss joints, namely W1, W2, W3, W4, W5, and W6.

Table 9. Roof Pressure Coefficient

Koefisien tekanan atap, C_p, untuk digunakan dengan q_h

		Di sisi angin datang								Di sisi angin pergi		
		Sudut, θ (derajat)								Sudut, θ (derajat)		
Arah Angin	h/L	10	15	20	25	30	35	45	≥ 60[c]	10	15	≥ 20
Tegak lurus terhadap bubungan untuk $\theta \geq 10^0$	≤ 0,25	-0,7 -0,18	-0,5 $0,0^a$	-0,3 0,2	-0,2 0,3	-0,2 0,3	$0,0^a$ 0,4	0,4	0,01θ	- 0,3	- 0,5	- 0,6
	0,5	-0,9 -0,18	-0,7 -0,18	-0,4 $0,0^a$	-0,3 0,2	-0,2 0,2	-0,2 0,3	$0,0^a$ 0,4	0,01θ	- 0,5	- 0,5	- 0,6
	≥ 1,0	$-1,3^b$ -0,18	-1,0 -0,18	-0,7 -0,18	-0,5 $0,0^a$	-0,3 0,2	-0,2 0,2	$0,0^a$ 0,4	0,01θ	- 0,7	- 0,6	- 0,6
Arah Angin	h/L	Jarak horizontal dari tepi sisi angin datang				C_p						

Table 10. Calculation of the Wind Load of the Main Trusss

Wind Load Location	Formula	Yield (kN)
W1	$[(a/2 + b) / \cos\alpha] \times Cti \ x \ L1x \ Qw$	0.106
W2	$(a/\cos\alpha) \times L1 \times Qw$	0.071
W3	$(a/2 \cos\alpha) \times L1 \times Qw$	0.035
W4	$(a/2 \cos\alpha) \times L1 \times Qw$	-0.390
W5	$(a/2 \cos\alpha) \times L1 \times Qw$	-0.779
W6	$[(a/2 + b) / \cos\alpha] \times Cti \ x \ L1x \ Qw$	-1,169

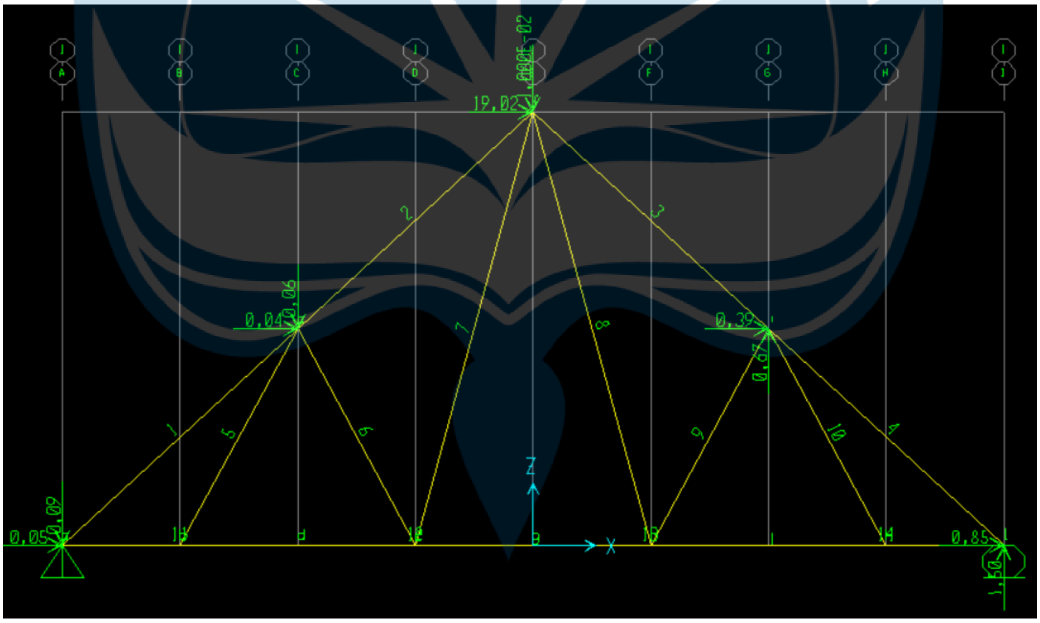

Figure 6. Projection of Wind Load from the Right at the Joint(SAP2000 Modeling)

From the shape of the truss and the load that has been obtained, then 2D modeling is carried out with SAP2000 software to determine the design force on the truss, the output in the form of reactions at the truss joints will be included as roof reaction data on buildings to facilitate building modeling in Etabs.

Table 11. L Steel Profile Specifications

STANDARD SECTIONAL DIMENSIONS				SECTION AREA	INFORMATIVE REFERENCE							
					CENTER OF GRAVITY	GEOMETRICAL MOMENT OF INERTIA			RADIUS OF GYRATION OF AREA			MODULUS OF SECTION
H x B	t	r_1	r_2	A	$C_x = C_y$	$I_x = I_y$	Max I_u	Min I_v	$i_x = i_y$	Max i_u	Min i_v	$Z_x = Z_y$
mm x mm	mm	mm	mm	cm^2	cm	cm^4	cm^4	cm^4	cm	cm	cm	cm^3
25 x 25	3	4	2	1.427	0.719	0.797	1.26	0.332	0.747	0.94	0.48	0.448
30 x 30	3	4	2	1.727	0.844	1.420	2.26	0.590	0.908	1.14	0.58	0.661
40 x 40	3	4.5	2	2.336	1.090	3.530	5.60	1.460	1.230	1.55	0.79	1.210
40 x 40	4	4.5	2	2.336	1.090	3.530	5.60	1.460	1.230	1.55	0.79	1.210
40 x 40	5	4.5	3	3.755	1.170	5.420	8.59	2.250	1.200	1.51	0.77	1.910
45 x 45	5	6.5	3	4.302	1.280	7.910	12.50	3.290	1.360	1.71	0.87	2.460
45 x 45	4	6.5	3	3.492	1.240	6.500	10.30	2.700	1.360	1.72	0.88	2.000
50 x 50	4	6.5	3	3.892	1.370	9.060	14.40	3.760	1.530	1.92	0.98	2.490
50 x 50	5	6.5	3	4.802	1.410	11.100	17.50	4.580	1.520	1.91	0.98	3.080
50 x 50	6	6.5	4.5	5.644	1.440	12.600	20.00	5.230	1.500	1.88	0.96	3.550

Based on calculations on Etabs, the maximum tensile force and maximum compressive force are selected. From the Designing of the stem force, 2 checks are carried out, namely as follows.

Serviceability Limit State Design (SLS)

In this check, it will be reviewed from the deformation where the deformation that occurs must be smaller than the allowable deformation. In this design, the deformation under consideration is based on the slenderness of the tensile and compressive rods (δ) with the profile size affecting the radius of gravitation (r).

$\delta = Lk/2r$ (Deflection Requirement; SNI 1729:2020)

Table 12. Terms of Slenderness of the Main Trusss

Profile 2L	Radius of Gyration (cm)	Pull Rod		Press rod	
		(1)	Safe: (1) < 240	(2)	Safe: (2) <200
2L50x50x5	1.52	86,449	Safe	86,449	Safe

Ultimate Limit State Design(ULS)

In this check, it will be reviewed based on the stress on the tensile and compressive rods where the stress that occurs must be less than the allowable stress (fy). In this check, an LRFD design is used in which the stress is reduced by 0.9 (Φ) for tension members and 0.85 (Φ) for compression members.

$f_t = \frac{Nu}{2 \times \Phi \times Ag}$, with (Tension of force for 2L Profile: SNI 1729: 2020)$\Phi = 0{,}9$

$f_c = \frac{\omega \times Nu}{2 \times \Phi \times Ag}$, with (Compression for 2L Profile: SNI 1729: 2020)$\Phi = 0{,}85$

$\omega = 1$, for$\delta_c \leq 0{,}25$

$\omega = \frac{1{,}43}{1{,}6 - 0{,}67 \times \delta_c}$, for$0{,}25 \leq \delta_c \leq 1{,}2$

$\omega = 1{,}25 \times {\delta_c}^2$, for$\delta_c \geq 1{,}2$

$\delta_c = \frac{Lk}{\pi r} \sqrt{\frac{f_y}{E}}$.

Table 13. Main Truss Stress

Profile 2L	Cross-sectional Area (cm2)	Pull Rod			Press rod		
		Style pull kN)	Tension (MPa)	Safe: < 240	Style Press (kN)	Tension (MPa)	Safe: < 240
2L50x50x5	4,802	18,584	21.5	Safe	21.19	38,602	Safe

Based on Table, it can be concluded that by using a 2L50x50x5 truss profile it will obtain safe stresses in the tension and compression rods, as well as safe slenderness requirements. Therefore, the truss profile used is 2L50x50x5.

Designing of Main Truss Connections

The truss connection usually uses bolts. The number of bolts used depends on the type and size of the bolts. In Designing the truss connection of this building, type A-325 bolts are used because they have a strength almost equivalent to grade 8.8 steel bolts. The bolts used are also threaded bolts with a bolt diameter of 12 mm to strengthen the gusset and truss Slab joints.

Table 14. Known Truss Connection Known Data

Known Data	Value	Unit	Information
Bolt diameter (d)	1,2	cm	-

Known Data	Value	Unit	Information
Hole diameter (h)	1,4	cm	Hole diameter based on SNI 1729:2020
Gross cross-sectional area of bolt non-threaded area (Ab)	(1/4π 1.22) = 1.13	cm2	The cross-sectional area of 1 bolt is 1.13 cm2, so for 2 bolts it is 2x1.13 = 2.26cm2
Thickness of gusset Slab (tp)	10	mm	-
tensile stress bolt break (fub)	825	MPa	Represents the tensile stress of a type A-325 bolt with thread
reduction factor invoice strength(Φf)	0.75	-	-
Tensile breaking stress between bolts (fu)	370	MPa	The tensile stress between bolt based on SNI 1729:2020
Steel stress(fy)	240	MPa	Allowable steel stress
shear stress nominal bolt (fnv)	372	MPa	Nominal shear stress of bolts type A-325 with thread
tensile stress nominal bolt (fnt)	620	MPa	Nominal tensile stress of bolts type A-325 with thread
r1	0.4 or 0.5	-	Based on SNI 1729:2020, 0.5 is used for threadless bolts, while 0.4 for threaded bolts
Easel Element Maximum Force (Nu)	18,584	kN	Got by using the maximum Main Truss-drawn design force obtained from the modeling SAP2000.
Gross Main Truss cross-sectional area Main Truss (Ag)	4,802	cm2	The cross-sectional area of the truss profile is 2L50x50x5

From the specifications of the bolts used, calculations are carried out to find the number of bolts used which is determined by comparing the smallest value between the shear strength and the bearing strength of the bolts. First of all, calculations are carried out using SNI 1729: 2020.

$$V_d = \frac{0{,}75 \times r1xFubxA_b}{1000} \text{ (Bolt Shear Strength)}$$

$$R_d = \frac{\Phi_f x2{,}4xdxt_p xA_b}{1000} \text{ (Bolt support Strength)}$$

The bolt shear strength is 28 kN, , the bolt bearing strength is 9042.38 kN. In this case there are two shear planes, the shear strength of the bolts becomes 2Vd, and the bearing strength of the Slab remains Rd, so the smallest values of 2Vd and Rd are chosen to find the number of bolts used with a minimum number of 2 bolts. The number of bolts obtained from is 0.879017 so the number of bolts used is 2 pieces. With 2 bolts, the gross cross-sectional area of the bolts is 2.26 cm2.

n = Nu/2Vd or Nu/Rd (Calculate required number of bolts)

*Choose the smallest amount of numbers

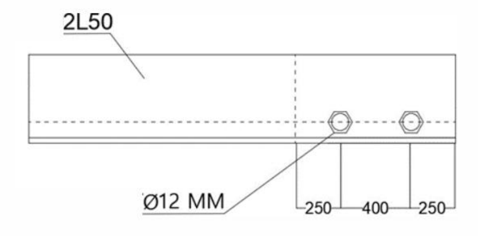

Figure 7. Easel Connection Designing Design

From the number and size of bolts used, ULS was checked by comparing the design strength with the largest design force of the truss rod elements. The reason for using the largest design style is to represent the design style of each truss element. ULS checking will be safe when all design strengths that occur are greater than the largest design force. The design strength obtained is the smallest value among the following styles:

a. Shear strength, namely the stress acting in the direction tangential to the cross section of the bolt caused by the shift between the gusset Slab and the steel profile.
b. Bearing Strength, namely the stress that occurs due to the action of the tensile force pressing the bolt in a fulcrum manner.
c. Tension on gross area,namely the stress that occurs in the gross cross-sectional area of the truss.
d. Tension on net area,namely the stress that occurs between the bolt and the cross-sectional area of the truss.
e. block shear strength,that is, the allowable limiting stress combining tensile failure in plane and shear failure in the plane perpendicular which in this case is a truss profile bolt.
f. slip critical strength,namely the allowable limit stress that occurs due to friction between two elements that are connected to join two structural elements which in this case are one truss profile with another truss profile.

Table 15. Calculation of Design Strength

Known Data	Formula	Value	Unit
Shear Strength			
ΦRn	Φ x [(Fnv x Ab,2 bolts)/10], with Φ = 0.75	106.56	kN
Bearing strength			
Rn closest hole	[1.2 x (2.5-h/2) x Fu] / 10	79,92	kN

Known Data	Formula	Value	Unit
Rn another hole	[1.2 x (4-h) x Fu] / 10	115,44	kN
Φ2Rn	Φ x 2 x Rn, with Φ = 0.75 and the smallest Rn	119.88	kN
Tension on gross area			
ΦRn	Φ x (Ag x fy /10), with Φ = 0.9	103,72	kN
Tension on net area			
ΦRn	Φ x Ag/2 x fu/10, with Φ = 0.75	66,63	kN
Block shear strength			
Gross area due to shear (Agv)	2 x 6.5 x Φ, with Φ = 0.7	9,1	cm2
Net area due to shear (Anv)	2 x (6.5 - 1.5 x 1.75) x Φ, with Φ = 0.7	5,425	cm2
Net area due to tension (Ant)	Φ x (4 - 1.75) with Φ = 0.7	1.1505	cm2
Rn1	(Fu/10) x (0.6 × $Anv + Ant$)	178.71	kN
Rn2	(Fy/10) x (0.6 × $Agv + Ant$)	168,84	kN
ΦRn	Φ × Rn, with Φ = 0.75 and Rn the smallest	168.84*	kN
Slip Critical Strength			
ΦRn	2 x 0.3 x Ab x 0.85 x 91	126.63	kN

*Choose smallest Rn

It can be seen that the largest design strength in Table 2.16 is 126.63 kN. By comparing the design strength which is greater than the largest design force on the rod which is 49.23 kN, it can be concluded that the design truss connection with threaded A-325 bolts with a diameter of 10 mm is safe for each element of the rod.

Roof for the Living Room (Joglo)

The roof structure system in the living room, a frame with a joglo form of double angled steel is used. The joglo-shaped roof frame is used in order to adjust the shape of the architectural design given.There is one frame profile in the middle that is longer than the other inner frames. so, roof truss design can use a profile that is more uniform and more economical. Following profile of Jolglo frame are:

Table 16. Living Room Roof Design

Data	Value	Unit
Lower roof slope	42	Degrees
Upper roof slope	20	Degrees
Roof span length	15	m

Data	Value	Unit
Purlin distance	1.5	m
Purlin type	ST-37	
Purlin load	0.034888	kN/m
roof load	0.0791232	kN/m
Ceiling load	0.2793	kN/m
Dead load	0.3933112	kN/m
live load	1	kN/m
Purlin profile	C 100 x 50 x 20 (t=2mm)	
Sagrood diameter	10	mm
Easel Designing		
roof height	3.5	m
Roofless structure height	12.32	m
Building height	15.82	m
The joints distance of the truss	2.2	m
Main Truss load (P1)	2.09	Kn
Main Truss load (P2)	1.93	Kn
Main Truss load (P3)	2.03	Kn
Easel profile	2L x 50 x 50 (h = 2mm)	
Wind load (W1)	0.106	Kn
Wind load (W2)	0.071	Kn
Wind load (W3)	0.035	Kn
Wind load (W4)	-0.390	Kn
Wind load (W5)	-0.779	Kn
Wind load (W6)	-1,169	Kn

For the calculation of the capacity of the materials and profiles used, such as the consideration on the Limas Roof and all the data, it shows that the joglo roof data has been able to support the given load.

2.2.2. Building Structure Designing

Building structure Designing is the second stage after Designing the roof structure. The Designing of the building structure includes the Designing of stairs, Slabs, beams and columns so that it can withstand the weight of the building structure.

Stair Designing

Stairs are one of the components in buildings found in multi-storey buildings that function as a means of connecting between floors. In Designing stairs, things that need to be considered are safety and comfort in using stairs. Therefore, in designing a Stair, it is necessary to pay attention to the slope of the stairs and the dimensions of each step. In a Stair, the components that compose it are the steps and the landing Slab which is located in the middle of the staircase design.

Table 17. Stair Design

Known Data	Value	Unit
Height Between Floors (hlt)	4	m
Stair Width	3.5	m
Border Width	1.75	m
Step Height (Optrede)	150	mm
Width of Steps (Antrede)	170	mm
Number of Steps (ntg)	hlt/Optire = 22	piece
Stair Elongated Width (Ltg)	(1/2 x hlt/Optrede - 1) = (1/2 x / -1) = 3.23	m
Staircase Corner	tan-1 (Optrede/Amtrede) = 29.54	degrees
Stair Slab Thickness (t)	150	mm
Equivalent Stair Thickness (tt)	(0.5 x Optred x Antrede) / [(Optrede2 + Antrede2)1/2] = 73.95	m
Total Equivalent Stair Slab (t')	[(t + tt) / cosα] = 257	mm

It is known that the height between floors 1 and 2 is 4 m high and the width of the stairs to be designed is 3.5 m. From these initial data, it can be planned in advance the size of the stairs to be used. In Designing the stairs, it is necessary to determine the dimensions for the 2 components, namely, the height of the steps (Optrede) and the width of the steps (Aptrede). The provisions for optrede sizes are between 150 and 200 mm, while aptrede sizes are limited to between 280 and 300 mm. For this plan, the planned optrede size is 150 mm and the aptrede size used is 170 mm as shown in Figure 2.14. after knowing the size of the steps, the next step is to determine the number of steps needed. The number of steps can be determined by dividing the height between the floors by the height of the steps. In this plan, the number of steps to be used is 20 which will be divided into 2 segments or parts.

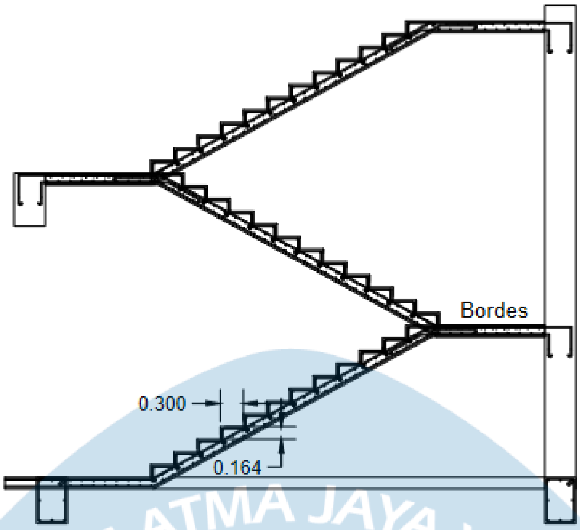

Figure 8. Staircase Details

From these results, it can be seen that the elongated width of the stairs to be built later is 3.5 m wide. In Figure 2.15, you can see the plan of the planned staircase. In the figure, it can be seen that the planned landing width is 1.75 m which is obtained from half the width of the planned stairs. Apart from getting the number of steps and the width of the stairs, the optrede and aptrede plan data can also be used to determine the angle of inclination of the stairs to be made. In this plan, the angle of inclination of the stairs is planned at 29.54□. For the thickness of the Stair Slab, it is assumed that the minimum thickness is 150 mm. These data will later be used in calculating Stair loading and Designing Stair reinforcement.

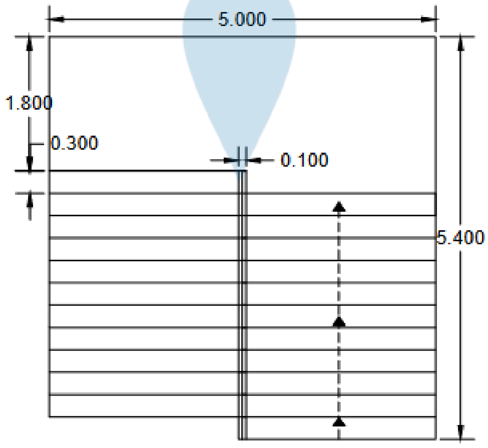

Figure 9. Staircase Floor Plan

After obtaining the dimensions of the stairwell, the angle of inclination of the stairs and the thickness of the stair Slabs, the Designing for loading the stairs can be carried out. Loading onThe stairs themselves can be seen in 2 places, namely on the sloping stairs and on the landing of the stairs. There are 2 types of loading that work on stairs, namely dead load and live load. The calculation of loading on the stairs can be seen bellow:

Table 18. Stair Loading Calculation

Stair Loading (qtg)		
Dead Load(qD)		
Stair Own Weight	(t/cosα) x concrete volume = 4.138	kN/m
Stair Weight	½ x Optred x concrete volume = 2.040	kN/m
Specific Weight and Tegel	$0.05 \times vol\ ubin$= 0.05 × 21 = 1.505	kN/m
Railing Weight	1	kN/m
Total	8,228	kN/m
Live Load(qL)		
Live Load	1.3	kN/m
Bordes loading (qbd)		
Dead Load(qD)		
Stair Own Weight	$t \times vol\ beton$ = 3.6	kN/m
Specific Weight and Tegel	$0.05 \times vol\ ubin$ = 0.05 × 21 = 1.05	kN/m
Railing Weight	1	kN/m
Total	5.65	kN/m
Live Load(qL)		
Live Load	1.3	kN/m

It can be seen the type of loading and the amount of loading acting on the Stair. On the stairs, there are 2 types of loading that will be used to calculate the Designing of the stairs, namely as follows.

a. Loading by the Stair itself. Stair load consists of dead load which consists of Stair weight, species weight and tiles, railing weight, Stair weight and live load which consists of live load. Based on the calculations, the total dead load acting on the Stair was 8,228 kN/m. Meanwhile, the live load that will be used refers to SNI 1727:2020 concerning the minimum design load for stairs in a 3-storey building, which is 1.3 kN/m.

b. Bordes loading. Bordes is part of the Stair as a place to rest towards the next Stair. Bordes in the form of a flat Slab between the steps. Loading on stairs and landings consists of a dead loadconsisting of Stair weight, specific weight and tiles, railing weight and live load consisting of live loads. From the calculation results, a dead load of 5.65 kN/m and a live load of 1.3 kN/m is obtained according to the minimum live load in SNI 1727:2020 regulations.

From the calculation of the load on the Stair, the internal force modeling on the Stair will be modeled using SAP2000. The modeling aims to determine the forces acting on the Stair due to loading such as shear forces and moments. Based on the modeling that has been done, a Shear Force Diagram (SFD) model is obtained which shows the shear force and the Bending Moment Diagram (BMD) which shows the moment due to Dead Load and Live Load. SFD Modeling and Stair BMD. Based on this modeling, the output is obtained in the form of shear forces and moments as shown in Table 2.19. The results of these forces will be used in the next step, namely to plan the reinforcement of the stairs.

. Table 19. Stair Modeling Data Using SAP2000

Shear Force Diagrams(SFD)		
Shear Force Due to Dead Load (V_{DL})	24.3	kN
Shear Force Due to Live Load (V_{LL})	19.54	kN
Bending Moment Diagrams(BMD)		
Moment Due to Dead Load (M_{DL})	17.33	kNm
Moment Due to Live Load (M_{LL})	5.65	kNm

After knowing the loading acting on the stairs and the forces generated, the next step is Designing the reinforcement of the stairs. In order to plan the reinforcement of the stairs, it is necessary to first find the design forces resulting from the combination of forces due to dead load and live load as shown in Table.

Table 20. Calculation of Stair Plane Force

The Moment of Plan (Mu)		
Mu1	$1.4 \times MDL = 24.262$	kNm
Mu2	$1.2 \times MDL + 1.6 \times MLL = 29.836$	kNm
Mur	Max (Mu1; Mu2) = Mu2= 29.836	kNm
Planned Shear Force (Vu)		
Vu1	$1.4 \times VDL = 1.4 \times 16.51 = 34.02$	kNm

Vu2	$1.2 \times VDL + 1.6 \times VLL = 60.42$	kNm
Vur	Max (Vu1; Vu2) = Vu2= 60.424	kNm

There is a calculation of the design force to be used in Designing the reinforcement of the stairs. The design force consists of design moments and design shear forces obtained from a combination of moments and shear forces due to dead loads and live loads. In this calculation, there are 2 types of combinations used. Of the two combinations, the combination that has the greatest value is selected to be used as design shear force (Vu) and design moment (Mu).

After obtaining the design style, the Designing of stair reinforcement can be carried out. On the Stair there are 2 types of reinforcement, namely reinforcement of the support Stair and reinforcement of the field Stair. This is because each stair area has a different moment so it needs to be reviewed on the support and the ground. Before the reinforcement calculation is carried out, it is necessary to know the initial data related to the stair design as shown in Table.

Table 21. Stair Reinforcement Known Data

Known Data	Value	Unit
Concrete Quality (f'_c)	25	MPa
Steel Stress (f_y)	420	MPa
Concrete Cover (d_s)	20	mm
Stair Slab Thickness (t)	150	m
Viewed width (b_w)	1000	mm
Diameter of Flexural Reinforcement (D_{tul})	13	mm
Diameter of Shear Reinforcement (d_{tul})	8	mm
Effective Thickness (d)	t - (ds + ½ + Dtul) = 121.5	mm

In the table above, it is known that the concrete cover to be used is 20 mm thick. In addition, in this design the reinforcement to be used for flexural reinforcement is deformed reinforcement with a diameter of 13 mm (D13) and shear reinforcement uses plain reinforcement with a diameter of 8 mm (d8). Therefore, the effective thickness of the Stair Slab to be used in reinforcement Designing is 123.5 mm. Once these initial data are known, the Designing of stair reinforcement can be carried out. In this plan, first review the reinforcement of the stairs in the support area. The calculation of the support reinforcement can be seen in Table below

Table 22. Calculation of Support Stair Reinforcement

Main Reinforcement		
Bending Moment (Mu)	0.8×Mur= 14.918	kN
Flexural Resistance Factor (Rn)	Mu / (0.9 x bw x d2) = 1.12	MPa
Minimum Rebar Ratio (pmin)	0.002	
Maximum Reinforcement Ratio (pmax)	0.75 x (0.85 x f'c x β / fy) x (600/600+fy) = 0.019	
Required Rebar Ratio (pneed)	0.85 x f'c / fy x (1-(1-2 x Rn/0.85 x f'c)^1/2) = 0.0049	
Reinforcement Surface Area Required (As required)	p× $bw \times d$ = 333.89	mm2
Distance Between Rebars (s)	bw × 0.25 × π × $Dtul2$ / As = 221.654 = 400	mm
Surface Area of Used Reinforcement (Used As)	bw × 0.25 × π × $Dtul2$ / s = 442	mm2
Principal Reinforcement used	D13-400	
Shear Strength (Vc)	1/6 x (f'c^1/2) x bw xd = 102916.67 = 102.916	kN
Shear Strength Capacity (ϕVc)	ϕ× Vc = 77187.5	kN
Safety Against Sliding	Vu <ϕVc = 60.424 < 77.19 (OK)	
Sharing Reinforcement		
Minimum Reinforcing Surface Area (As min)	$pmin \times bw \times t$= 300	mm2
Distance Between Rebars (s)	$bw \times 0.25 \times \pi \times dtul2$ /As = 167.612 = 150	mm
Surface area Divide (As)	$bw \times 0.25 \times \pi \times dtul2$/s = 331.241	mm2
Reinforcement Check	As divided > As min = 331.241 > 300 (OK)	
Reinforcement for those used	d8-150	

On Table above, there is a calculation for the reinforcement of the support Stair which is divided into main reinforcement and dividing reinforcement. In Designing the main reinforcement, the plan must be able to withstand the design bending moment that occurs at the support of the Stair which is equal to 26,173 kN. In this plan, Designing for stair reinforcement will be reviewed every 1 m wide (bw). Then, from the known bending moment, the bending resistance factor (Rn) will be calculated. after obtaining the value of Rn, the next step is to determine the minimum, maximum reinforcement ratio (p) and the required reinforcement ratio based on the initial design data.

Required reinforcement ratio (prequired) must meet the condition pmin < pneed < pmax. If the value of the reinforcement ratio needs to be less than the minimum ratio, then the minimum ratio is used. Meanwhile, if the reinforcement ratio value needs to be more than the maximum ratio, then the maximum reinforcement ratio value is used. From the required reinforcement ratio, the reinforcement surface area (As required) is calculated for every 1 m of width. The required As value will later be used in calculating the spacing of the reinforcement that will be used in the insHeightation of the main reinforcement. The reinforcement spacing obtained can be rounded off and a reinforcement spacing of 150 mm is obtained. From the reinforcement distance, the surface area of the reinforcement to be used in the insHeightation is sought.

After knowing the details of the reinforcement to be used in the form of D13-150, check the shear forces that occur. before checking, calculation of the shear strength (Vc) and shear strength capacity (ϕVc) that can be supported by the design of the Stair. The shear strength capacity value (ϕVc) must be greater than the design shear strength value (Vu). After the main reinforcement is obtained and the main reinforcement plan has been checked against the design forces that occur, the next step is to calculate the required reinforcement.

The calculation of the reinforcement design will use the minimum reinforcement ratio. So that in the calculation will be determined in advance the area. minimum reinforcement surface (As_{min}). Through the surface area of the minimum reinforcement, it can be known the distance between reinforcement required in the insHeightation of reinforcement for. The spacing between reinforcement can be used if the design has a reinforcement surface area that is greater than the minimum reinforcement surface area. From the calculation of the reinforcement that has been carried out, it is determined that the details of the reinforcement used are d8-150. Next step is to find a plan for stair reinforcement in the field area. In the calculation of stair reinforcement in the field area, the type of calculation performed is the same as the calculation of stair reinforcement in the support area. The thing that distinguishes the Designing of the stairs of the support area and the field is the style of plan that works in each of these areas.

Table 23. Field Stair Reinforcement Calculation

Main Reinforcement		
Bending Moment (Mu)	$0.5 \times Mur$ =16,354	kN
Flexural Resistance Factor (Rn)	Mu / (0.9 x bw x d2) = 1.23	MPa
Minimum Reinforcement Ratio (pmin)	0.002	
Maximum Reinforcement Ratio (pmax)	0.75 x (0.85 x f'c x β / fy) x (600/600+fy) = 0.019	
Required Rebar Ratio (pneed)	0.85 x f'c / fy x (1-(1-2 x Rn/0.85 x f'c)^1/2) = 0.003	
Reinforcement Surface Area Required (As required)	p$\times$ $bw \times d$ = 367.137	mm2
Distance Between Rebars (s)	$bw \times 0.25 \times \pi \times Dtul2$ / As = 361.68 = 350	mm
Surface Area of Used Reinforcement (Used As)	$bw \times 0.25 \times \pi \times Dtul2$ / s =442	mm2
Main reinforcement used	D13-150	
Shear Strength (Vc)	1/6 x (f'c^1/2) x bw xd = 102.92	kN
Shear Strength Capacity (ϕVc)	$\phi\times Vc$ = 77.19	kN
Safety Against Sliding	u <ϕVc = (OK)	
Sharing Reinforcement		
Minimum Reinforcing Surface Area (As min)	$pmin \times bw \times t$= 300	mm2
Distance Between Rebars (s)	$bw \times 0.25 \times \pi \times dtul2$ /As = 167.62 = 150	mm
Reinforcing Surface Area For (As for)	$bw \times 0.25 \times \pi \times dtul2$/s = 300	mm2
Reinforcement Check For	As divided > As min (OK)	
Reinforcement for used	d8-150	

The thing that distinguishes this calculation from the calculation of the support reinforcement is the amount of bending moment acting on the plane. Through the calculations carried out, it is obtained details of reinforcement in the field Stair in the form of main reinforcement D13-150 and reinforcement for the same as in the reinforcement reinforcement, namely reinforcement d8-150.

Planned Bordes Beams

After the Designing of the stairs has been completed, the next plan is to plan the landing beam.

The landing beam is a beam that is located side by side with the stairs. The landing beam can help withstand the reaction force generated by the stairs and channel it to the nearest column. Before Designing the landing beams, it is necessary to know in advance the Known Data of the landing beams.

Table 24. Bordes Beam Known Data

Known Data	Value	Unit
Border Width (L)	4	m
Beam Height (h)	350	mm
Beam Width (b)	500	mm
Floor Height (p_{lt})	3.5	m
Wall Thickness (t_d)	0.15	m
Diameter of Flexural Reinforcement (Dtul)	19	mm
Diameter of Shear Reinforcement (dtul)	8	mm
Flexural Reinforcement Stress (fy)	420	MPa
Shear Reinforcing Stress (fy)	280	MPa
Concrete Cover (d_s)	40	mm
Effective Thickness (d)	h - ds - dreinforcement - (1/2 x Dreinforcement) = 292.5	mm

It is known that the landing beam to be designed has a length of 4 m with dimensions of 350 x 500 mm. In the drilled beam, it is planned to use threaded reinforcement with a diameter of 19 mm (D19) as flexural reinforcement and plain reinforcement with a diameter of 8 mm (d8) as shear reinforcement. After knowing the design data of the landing beam, the next step is to calculate and model the loading of the landing beam. Calculation of landing beam loading can be seen

Table 25. Calculation of Bordes Beam Loading

Dead Load(qD)		
Self Weight	$h \times b \times vol\ beton$= 0.3 × 0.25 × 24 = 1.8	kN/m
Wall Weight	(hlt/2 - h) x td x Brick Vol = 4.25	kN/m
Stair Reaction	VDL = 29.04	kN/m
Total	35.09	kN/m
Live Load(qL)		
Stair Reaction	V_{LL}= 24.01	kN/m

On the landing beam, the type of loading that occurs is the same as loading on general beams, namely dead load and live load. In dead load loading, the type of load acting is the load due to its own weight, the weight of the wall, and the reaction of the Stair in the form of a shear force. From the calculation results, the total dead load on the landing beam is 21.935 kN/m. In the live load loading, the type of load acting is the load due to the reaction of the stairs in the form of a shear force due to the live load (VLL) of 24.01 kN/m. From the calculation results, the total live load on the landing beam is 10.86 kN/m. After obtaining the loading acting on the landing beam, then step next is do modeling beam landing withusing SAP2000. Bordes beam modeling results using SAP2000 SFD Modeling and BMD Bordes Beams. The modeling aims to determine the shear forces and bending moments that occur due to loading on the landing beam. Based on this modeling, the output results are obtained which can be seen in Table 2.26. This value will be used in the design of flexural and shear reinforcement in bordes beam.

Table 26. Bordes Beam Modeling Data Using SAP2000

Shear Force Diagrams(SFD)		
Shear Force (V_u)	73,264	kN
Bending Moment Diagrams(BMD)		
Bending moment of support (M_u)	107,365	kNm
Pitch bending moment (M_u)	53,683	kNm

After knowing the shear forces and bending moments acting on the landing beam, then proceed with the calculation of the landing beam reinforcement. Bordes reinforcement is divided into two parts, namely support reinforcement and field reinforcement. The results of the Designing calculations for the reinforcement of the landing beam supports can be seen in Table

Table 27. Calculation of Bend Support for Bordes Beams

Flexural Reinforcement		
Bending Moment (Mu)	53,682	kN
Flexural Resistance Factor (Rn)	Mu / 0.9 xbx d2 = 1.394	MPa
Reinforcement Ratio Minimum (pmin)	0.002	
Maximum Reinforcement Ratio (pmax)	0.75 x (0.85 x f'c x β / fy) x (600/600 + fy) = 0.0189	
Required Rebar Ratio (p need)	(0.85 x f'c / fy) x (1 - (1-(2 xRn/0.85xf'c) = 0.0034	
Reinforcement Surface Area Required	p$\times b \times d$ = 502,599	

Flexural Reinforcement		
(As required)		mm2
Number of Reinforcement (n)	As / (0.25 x$\pi \times Dtul2$) = 1.771 = 2	piece
Wide Reinforcing SurfaceUse (USuse)	$n \times 0.25 \times \pi \times$ Dtul2 = 567.286	mm2
Flexural Reinforcement used	2D13	
Nominal Moment Check		
a	(As x fy / 0.85 x f'c xb) = 22.42	mm
c	a/β = 26.38	mm
strain (ε_t)	(0.003 x (dc))/c = 0.03 > 0.005	
Nominal Moment Capacity (ϕM N)	ϕ$\times As \times fy \times (d$ - a/2) = 60.32	kNm
Moment Check	ϕMn > Mu = 60.32> 53.682 (OK)	

It can be seen the calculation of the field bending reinforcement of the drilled beam and checking the bending moment that occurs. The first step in the design is to determine the flexural resistance factor and reinforcement ratio. The reinforcement ratio used must be within the minimum and maximum reinforcement ratio range. From the reinforcement ratio, the required reinforcement surface area can be determined and the amount of reinforcement to be used is determined. In the support area of the landing beam, 2 pieces of threaded reinforcement with a diameter of 13 mm (3D13) are used. From the reinforcement plan, it is known that the reinforcement design has a nominal moment capacity of 60.32 kNm so that it is safe and can withstand the bending moment that occurs, which is 53.682 kNm. Besides that, Also check the strain that occurs in the reinforcement design. From this design, it is obtained that the strain that occurs is 0.03, where this value is safe because it is greater than the strain according to the provisions, which is equal to 0.005.

After obtaining the details of the reinforcement in the bearing area of the landing beam, the next step is to determine the reinforcement of the landing beam field. The calculation step for the field reinforcement of the bored beam is the same as for the calculation of the support area. The thing that distinguishes these calculations is the amount of bending moment that works. In the field area, the bending moment that works tends to be smaller in value than the support area. The results of the calculation of the field reinforcement of the landing beam can be seen in Table.

Table 28. Calculation of Bending Field Bending Reinforcement Bordes

Flexural Reinforcement		
Bending Moment (Mu)	53,683	kN
Flexural Resistance Factor (Rn)	Mu / 0.9 xbx d2 = 1.39	MPa
Reinforcement Ratio Minimum (pmin)	0.002	
Maximum Reinforcement Ratio (pmax)	0.75 x (0.85 x f'c x β / fy) x (600/600 + fy) = 0.0189	
Required Rebar Ratio (p need)	(0.85 x f'c / fy) x (1 - (1-(2 x Rn/0.85xf'c) = 0.003	
Reinforcement Surface Area Required (As required)	p× $b \times d$ = 502,599	mm2
Number of Reinforcement (n)	As / (0.25 x$\pi \times Dtul2$) = 2	piece
Wide Reinforcing SurfaceUse (USuse)	n × 0.25 × π × D$tul2$ = 567.286	mm2
Flexural Reinforcement used	2D13	
Nominal Moment Check		
a	(As x fy / 0.85 x f'c xb)= 22.42	mm
c	a/β = 26,382	mm
strain (ε_t)	(0.003 x (dc))/c = 0.003 > 0.005	
Nominal Moment Capacity (ϕM N)	ϕ× $As \times fy \times (d - a/2)$ = 60.31	kNm
Moment Check	ϕ Mn > Mu = 60.31 > 53.683 (OK)	

In Table, the results of the calculation of the landing beam reinforcement in the field area are given. In the field area, the bending moment that occurs is smaller than the supporting bending moment, which is only 53,683 kNm. Therefore, in these calculations, the flexible reinforcement to be used is 2 pieces of reinforcement with a diameter of 13 mm (2D13). Then the Designing results are also checked nominal moment capacity. From the calculation results, it is known that the design of the reinforcement is capable of withstanding the bending moment that occurs of 53,683 kNm because the nominal moment capacity is 60.31 kNm. In addition, the design of the reinforcement is also safe against the strain that occurs, which is equal to 0.003.

After knowing the flexural reinforcement of the supports and the field of the landing beam, it is then necessary to calculate the required shear reinforcement to be used. This is so that the design of the landing beam is safe against shear forces that occur due to loading. The results of the calculation of shear reinforcement can be seen in Table.

Table 29. Calculation of Bordes Beam Shear Reinforcement

Data	Calculation	Unit
Shear Force (Vu)	161,048	kN
Shear Strength (Vc)	1/6 x (f'c)^1/2 xbxd =121,875	kN
Shear Strength Capacity (ϕVc)	ϕ× Vc =91,406	kN
Safety Against Sliding	Vu <ϕVc = 161.048 <91.406 (Required shear reinforcement)	
Nominal Shear Strength (Vs)	Vu/ϕ-Vc = 92,856	kN
Distance between support reinforcement (s1)	(0.25 × π × $dtul2$ × fy × d) / Vs =146.25	mm
Distance Between Field Reinforcement (s2)	(0.25 × π × $dtul3$ × fy × d) / b = 168.96	mm

The calculation of the need for shear reinforcement in the bordes beam is performed. From these calculations, it is known that the landing beam requires shear reinforcement because the shear force that arises cannot be resisted by the concrete. In this shear reinforcement, plain reinforcement with a diameter of 8 mm (d8) is used. Before determining the distance between the reinforcement, it is necessary to calculate the nominal shear strength value that will be retained by the shear reinforcement. Then it can be determined the distance between the shear reinforcement for each part of the support and the landing beam field. From the results of these calculations, it was found that in the support area shear reinforcement with a spacing of 75 mm (d8-75) would be used. As for the landing beam field area, shear reinforcement with a spacing of 150 mm (d8-150) will be used.

2.2.3. Seismic Force Distribution Designing

The seismic force distribution is Designing of forces that occur due to earthquakes that occur in a structure, such as axial forces and lateral forces. To find out the distribution of seismic forces, the step taken is to determine the class of the soil site from the area to be built based on SPT (Soil Penetration Test) data.

From the SPT soil data, a soil site class calculation is carried out to find out the land site class in the area being built, in this case the Gunung Kidul area. To find out the class of the soil site, you can use one of the 3 parameters, which are as follows.

1. The average blow (N), which is the average standard penetration resistance of the soil in layers to a depth of 40 m from the soil surface. Results based on SPT (Static Penetration Test) data.

2. The average shear strength (Su), which is the average non-linear shear strength in the 40 m layerfrom the ground. Results based on CPT (Cone Penetration Test) data.
3. The average propagation velocity ($\bar{V}$s), namely the average shear wave propagation speed at small shear strains in layers up to 40 m from the soil surface.

In the Gunung Kidul area, soil site classes will be based on SPT data as shown in Table 2.30. Then the data is processed to find out the average stroke of

Table 30. Soil Type Calculation Based on SPT Data

Layers	depth	N-SPT	Cn	Cw	N1	delta z	delta z/N-SPT
Layer 1	2	34	1.369023	1	46.54678	2	0.059
	4	49	1.13723	1	55.72426	2	0.041
	6	51	1.00164	1	51.08362	2	0.039
	8	52	0.905437	1	47.08271	2	0.038
	10	52	0.830816	1	43.20243	2	0.038
	12	54	0.769846	1	41.57171	2	0.037
	14	55	0.718297	1	39.50636	2	0.036
	16	55	0.673644	1	37.0504	2	0.036
	18	57	0.654572	1	37.3106	2	0.035
	20	59	0.636529	1	37.55524	2	0.034
Layer 2	22	60	0.619411	1	37.16465	2	0.033
	24	60	0.603126	1	36.18756	2	0.033
	26	60	0.587598	1	35.25585	2	0.033
	28	60	0.572758	1	34.36549	2	0.033
	30	60	0.55855	1	33.51297	2	0.033
	32	60	0.54492	1	32.69521	2	0.033
	34	60	0.531824	1	31.90946	2	0.033
	36	60	0.519222	1	31.15334	2	0.033
	38	60	0.507078	1	30.42468	2	0.033
	40	60	0.495359	1	29.72155	2	0.033
Σ						40	0.728
N-SPT						54,955	

Table 31. Land Site Class Classification Based on SNI 1726:2019

Site Class	Vs (m/sec)	N or Nch	Su(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	³100
SD (medium soil)	175 to 350	15 to 50	50 to 100

Site Class	Vs (m/sec)	N or Nch	Su(kPa)
SE (soft soil)	<175	<15	<50
	Or any soil profile containing more than 3m of soil with the following characteristics: a. Plasticity index, PI > 20, b. Moisture content, w ³ 40% c. Nilaire shear strength, Su < 25 kPa		
SF (special soil, requiring specific geotechnical investigation and site-specific response analysis following 0)	Any soil profile that has one or more of the following characteristics: a. Prone and has the potential to fail or collapse due to earthquake loads such as easy liquefaction, very sensitive clay, weak cemented soil b. Very organic loam and/or peat (H thickness > 3m)		

It obtained$N$ = 40/0.728 = 54.955. By comparing the results of Ñ with Table 2.31, the soil site class obtained is included in the SD soil site class which is medium soil because Ñ is more than 50 which is a type of Very Dense Soil and Soft Relief. The soil site class obtained is used to determine the spectrum response of the Gunung Kidul area according to the soil site class. Based on the RSA application or Response Spectrum Application, data is obtained as shown in Table

Table 32. RSA Analyses of Gunung Kidul Regency

Data	Value	Information
SDS	0.81	Parameter of acceleration response spectrum in period short, damping5%, defined in 0
SD1	0.63	Parameter response spectrum acceleration in 1 second period, 5% attenuation, defined in 0
To	0.156	-
Ts	0.778	-
QL	20	Long period transition map

Table 33. Response Spectrum of Gunung Kidul SD Site Class Based on RSA

Q	sa
0.000	0.324
0.156	0.810
0.778	0.810
0.878	0.718

Q	sa
0978	0.644
1,078	0.585
1.178	0.535
1,278	0.493
1,378	0.457
1,478	0.426
1,578	0.399
1678	0.375
1,778	0.354
1878	0.336
1978	0.319
2078	0.303
2.178	0.289
2,278	0.277
2,378	0.265
2,478	0.254
2,578	0.244
2,678	0.235
2,778	0.227
2,878	0.219
2,978	0.212
3,078	0.205
3.178	0.198
3,278	0.192
3,378	0.187
3,478	0.181
3,578	0.176
3,678	0.171
3,778	0.167
3,878	0.162
3,978	0.158
4,078	0.154
4,178	0.151
4,278	0.147
4,378	0.144
4,478	0.141
4,578	0.138
4,678	0.135
4,778	0.132
4,878	0.129
4,978	0.127
5,078	0.124

Q	sa
5.178	0.122
5,278	0.119
5,378	0.117
5,478	0.115
5,578	0.113
5,678	0.111
5,778	0.109

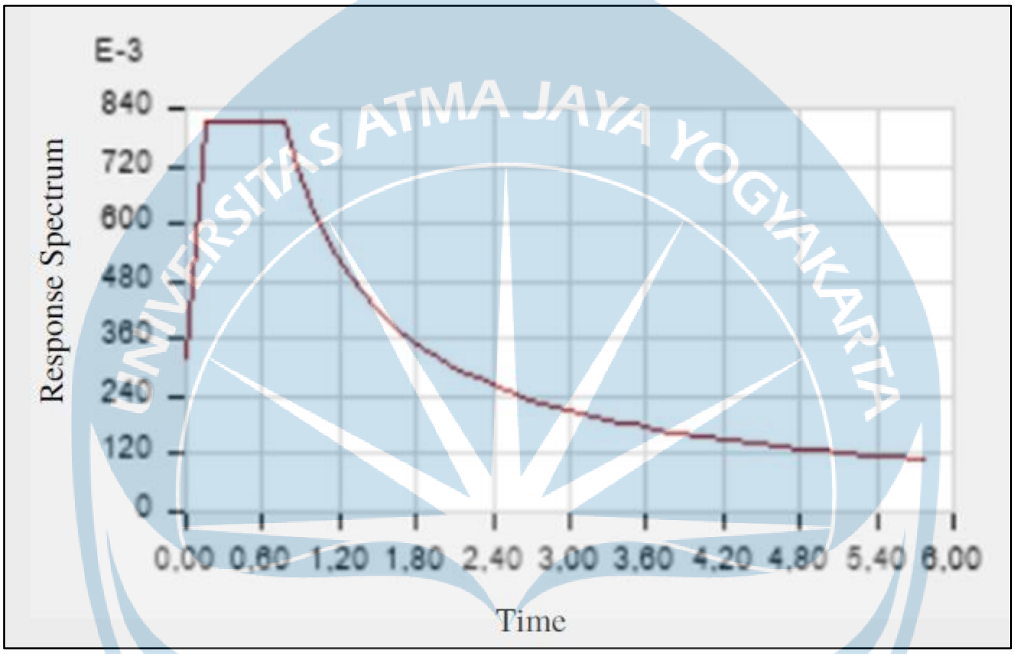

Figure 10. Spectrum Response Graph of Gunung Kidul Region (Output Etabs)

The next step is to determine the seismic priority factor and seismic design category. This is done with the aim of ensuring the structural detail meets the requirements in accordance with the estimated earthquake intensity. Determination of the seismic priority factor can be determined from the building risk category in Table:

Table 34. Building Risk Category for Earthquake Loads

Utilization Type	Risk Category
Buildings and non-buildings that pose a low risk to human life in the event of a failure, including, but not limited to, among others: Agricultural, plantation, animal husbandry and fishery facilities, Temporary facilities Warehouse Guardhouses and other small structures	I

Utilization Type	Risk Category
All buildings and other structures, except those included in risk categories I, III, IV, including, but not limited to: a. Housing area Home shop and home office b. Market c. Office building d. Apartment/flat building e. Shopping centers/malls f. Manufacturing facility g. Factory	II
Buildings and non-buildings that pose a high risk to human life in the event of a failure, including, but not limited to: a. Cinema b. Meeting hall c. Stadium d. Health facilities that do not have surgical and emergency units e. Child care facilities f. Jail g. Buildings for the elderly	III

Table 35. Priority Factor based on Building Risk Category

Risk category	The priority factor of the earthquake, I_C
I or II	1.0
III	1.25
IV	1.50

Table 36. Seismic Design Category based on Short Period Acceleration

S Grade	Risk category	
	I or II or III	IV
S_{DS} < 0.167	A	A
0.167 lbs_{DS}< 0.33	B	C
0.33 lb S_{DS}< 0.50	C	D
0.50£ S_{DS}	D	D

Table 37. Seismic Design Category based on 1 Second Period Acceleration

S Grade	Risk category	
	I or II or III	IV
SDI < 0.067	A	A
0.067 SIN< 0.133	B	C
0.133 SIN< 0.20	C	D
0.20 SIN	D	D

It is found that the Nursing Home building has a building risk category of II so that the seismic priority factor (Ie) is 1.00 based on Table 2.35. For the short period seismic design category, with an SDS of 1.119242 in the building category II, the risk category is in category D and with an SD1 of 0.49809 in the building category II, the seismic design category is also in category D. So from Therefore, it is concluded that the seismic design risk category is in category D. The seismic resisting system is taken as a special moment-bearing reinforced concrete frame system (SNI 1726: 2019, Table 12). Special moment-bearing reinforced concrete frame system is one of the methods to withstand earthquake loads and inelastic responses that require the structure to be ductile or flexible so that it can be categorized as safe. The seismic resisting system is a structural frame system that has the strength to withstand planned shear forces, axial forces, and moments.

Table 38. Special Moment Resisting Reinforced Concrete Frame System Data

Known data	Value	Information
R	8	Response modification coefficient for bearing reinforced concrete frame systems special moment.
C_d	5,5	Lateral drift magnification factor for special moment-bearing reinforced concrete frame systems.
Ω_0	3	Strong factor is more system to system Framework concrete boned bearer special moment.
C_s	0.139905	Factor response seismic with using a special moment-bearing reinforced concrete frame system.

The next step is to calculate the planned weight of the structure, which consists of the weight of the roof and the weight of the floor, as shown

Table 39. Calculation of Loading Structure

Data	Value	Unit	Information
Roof Weight			
Dead Load			
Roof	(29.089 x 16) + (29.08 x 10) = 756,314	kN	Multiplication of specific gravity roof and dimensions
Column	(0.6 x 0.6) x 82 x 24 = 708.48	kN	Multiplication of specific gravity concrete and dimensions
Primary Beam	(0.3 x 0.6) x 24 x 114 = 492.74	kN	Multiplication of specific gravity concrete and dimensions
Wall	2 x 114 x 2.55 = 581.4	kN	Multiplication of specific gravity concrete and dimensions
ceiling	40 x 17 x 0.2 = 136.24	kN	Multiplication of specific gravity ceiling and dimensions
Total Dead Load	2675.48	kN	The sum of the roof weight, column weight, wall weight, ceiling weight, and weight beam
Total Roof Weight (W)	2675.48	kN	On roof, only use dead load because there is no live load due to seismic loads
Floor Weight 3			
Dead Load			
Column	(0.6 x 0.6) x 24 x 82 = 708.48	kN	Multiplication of specific gravity concrete and dimensions
Primary Beam	(0.3 x 0.6) x 225 x 24 = 972	kN	Multiplication of specific gravity concrete and dimensions
Wall	3.5 x 0.2 = 16.8	kN	Multiplication of specific gravity concrete and dimensions
Slab	0.12 x 24 x 1505 = 4334.4	kN	Multiplication of specific gravity concrete and dimensions
Tile	0.03 x 24 x 1505 = 1083.6	kN	Multiplication of specific gravity concrete and dimensions
Specs	0.02 x 24 x 1505 = 276	kN	Multiplication of specific gravity concrete and dimensions
ceilingd	0.1862	kN	Multiplication of specific gravity concrete and dimensions
Bordes	16 x 4 = 64	kN	Multiplication of specific gravity concrete and dimensions

Data	Value	Unit	Information
Total Dead Load	7180516	kN	The sum of the loads counted dead
Live Load			
Slab	4.79 x 1505= 7208.95	kN	Multiply the weight of the bordes and dimensions
Bordes	4.79 x 4 = 19.16	kN	Multiply the weight of the bordes and dimensions
Total Live Loads	2768.62 x 0.3 = 7199.68	kN	The sum of Slab s and landings, then reduced 0.3
Floor Weight 2			
Dead Load			
Column	708.48	kN	Multiplication of specific gravity concrete and dimensions
Primary Beam	972.00	kN	Multiplication of specific gravity concrete and dimensions
Secondary Beam	112.32	kN	Multiplication of specific gravity concrete and dimensions
Wall	4334.40	kN	Multiplication of specific gravity concrete and dimensions
Slab	1.05	kN	Multiplication of specific gravity concrete and dimensions
Tile	1083.60	kN	Multiplication of specific gravity concrete and dimensions
Specs	0.19	kN	Multiplication of specific gravity concrete and dimensions
ceiling	16.80	kN	Multiplication of specific gravity concrete and dimensions
Bordes	64.00	kN	Multiplication of specific gravity concrete and dimensions
Total Dead Load	7292.83	kN	The sum of the loads counted dead
Live Load			
Slab	7208.95	kN	Multiplication of specific gravity concrete and dimensions
Bordes	19.16	kN	Multiplication of specific gravity concrete and dimensions

Data	Value	Unit	Information
Total Live Loads	19.16	kN	The sum of Slab s and landings, then reduced 0.3

After obtaining the shear force and the effective weight of the building, the seismic distribution of forces is calculated. The results of the calculation of the seismic force distribution are then inputted into the structural Designing on ETABS.

$k = 0{,}5Ts + 0{,}75$, with (Seismic Force Distribution)$Ts = 0{,}44502$

$C_{vx} = \frac{W \times h^k}{\Sigma W \times h^k}$. (Seismic Force Distribution)

$F_x = C_{vx} x V$(Seismic Force Distribution)

Table 40. Seismic Distribution Force

Stories	W(kN)	Elevation h(m)	W.hk (kNm)	Cvx	Axial Force, Fx (kN)
Roof	2675.48	10.54	15650.64	0.24	0.00
3rd floor	7292.84	7.00	31384.85	0.48	0.00
2nd Floor	14492.51	3.50	37084.70	0.44	0.00
Base	0.00	0.00	0.00	0.00	0.00
Total	24460.83	-	84120.18	-	-

The results of the seismic force distribution if projected, then each floor has its own seismic force with the load of each floor where styleseismic on the roof is 2675.48 kN, on the 3rd floor is 7292.84 kN because it supports the roof force, the 2nd floor is14492.51 kN due to supporting the roof force and the 2nd floor force while on the base (ground floor) is 0 because it touches the ground so there is no lateral force.

2.2.4. Slab Designing

The Slab is a floor that is not directly above the ground, it is a floor level dividing one level with another level. The floor Slab is supported by beams that rest on building columns. The floor Slab consists of field reinforcement and support reinforcement which each has an X direction and Y direction. To plan the floor Slab , Slab loads are needed. Slab loads consist of floor loads, which consist of dead loads, super dead loads and live loads. The floor load consists of several types of loading, namely self load, sand load, tile load, species load, ceiling load, and finishing.

Table 41. Slab Design Data

Data	Value	Unit	Information
Type	Type II Slab	-	-
Concrete quality (f'c)	25	MPa	-
Slab Thickness (t)	120	mm	-
The thickness of the blanket (ts)	20	mm	-
Tension reinforcement (fy)	280	MPa	Tension reinforcement which allowable is 280 MPa
Diameter reinforcement (d)	10	mm	-
Effective length X direction (dx)	t-d	mm	Depends on the diameter reinforcement used
Effective length Y direction (dy)	t - d - ts	mm	Depends on the diameter reinforcement used
Press face width structure (b)	1000	mm	Based on SNI 1727:2020
Slab /floor loads	0.12 x 25 = 3	kN/m2	Because the thickness of the floor is 120 mm, while the volume load is 25 kN/m3, the floor load is 3 kN/m2
Tile load and specs	0.05 x 21 = 1.05	kN/m2	Because the thickness of the tiles and the species is 0.05 m and loads the volume is 21 kN/m3, then the load is 1.05 kN/m2
Sand load	0.05 x 18 = 0.9	kN/m2	Because the thickness of the sand in the casting is 0.05 m and the volume load is 18 kN/m3, the sand load is is 0.9 kN/m2
Ceiling load	0.2	kN/m2	Used material PVC (Polyvinyl Chloride) because more supple, light, and durable
Slab InsHeightation	0.05 x 22 = 1.1	kN/m2	Is a load resulting from finishing work, namely insHeightation ceramics

Data	Value	Unit	Information
Dead load (DL)	3 + 1.05 + 0.9 + 0.2 + 1.1 = 6.25	kN/m2	Total of floor load, sand load, tile and species load, ceiling load, and finishing
Live load (LL)	250 kg/m^2 / 1000 kN = 2.5	kN/m2	Burden life building office is 250 kg/m2 based on SNI 1727:2020
Planned load (qu)	11.5	kN/m2	Combination 1.2DL+1.6LL

In Slab Designing, reinforcement calculation will be carried out. Slab reinforcement design can be seen in Figure. Reinforcement on the Slab consists of 2 types of reinforcement, namely as follows. Field reinforcement, is the principal reinforcement whose position is in the middle of the span. There are 2 pieces of field reinforcement on the floor Slab, namely the main field reinforcement in the X direction and the main field reinforcement in the Y direction.
Support reinforcement, is reinforcement whose position is around the support area. On the floor Slab, used 2 types of reinforcement, namely as follows.

Principal reinforcement, is the main reinforcement that extends in the direction of the length of the beam. The main reinforcement is insHeighted in the X direction and Y direction. Reinforcement for, namely reinforcement used to maintain the position or position of the main reinforcement at the time of casting so that it does not change from its original place. Reinforcement for insHeighted in a direction perpendicular to the main reinforcement. The insHeightation consists of an X direction and a Y direction.

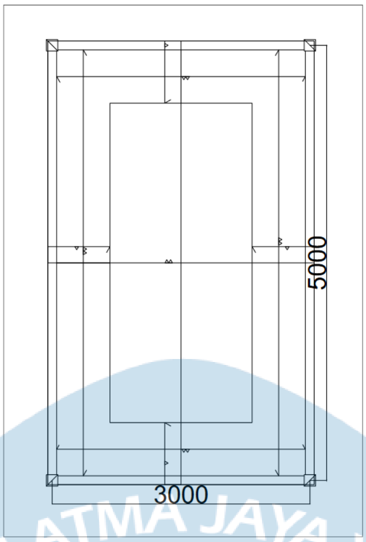

Figure 11. Slab Reinforcement Design

An example of Slab Designing will use AutoCAD Drawings. The first step is to determine the dimensions of the Slab with the shortest dimension being Lx and the longest dimension being Ly which on Slab 1, Lx and Ly are 3 m and 5 m respectively. Then do a comparison between Ly and Lx, we get 1.7 so we use a two-way Slab, which is used to get the moment coefficient. After getting the coefficient (C) for each moment, do the calculations for each field moment and the support in the X and Y directions.

Mu = 0.001 x qu x Lx2 x C (Calculation of Slab Moment Coefficient)

Table 42. Class II Slab Moment Coefficient

Kondisi pelat		Nilai Momen Pelat	Perbandingan L_y/L_x																
			1	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2	2.1	2.2	2.3	2.4	2.5	2.6
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
Lx, Ly	1.1	$Ml_x = 0.001.Cl_x.q.L_x^2$	44	52	59	66	73	78	84	88	93	97	100	103	106	108	110	112	125
	1.2	$Ml_y = 0.001.Cl_y.q.L_x^2$	44	45	45	44	44	43	41	40	39	38	37	36	35	34	33	32	25
	1.3	$Mt_x = -0.001.Ct_x.q.L_x^2$	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	1.4	$Mt_y = -0.001.Ct_y.q.L_x^2$	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2.1	$Ml_x = 0.001.Cl_x.q.L_x^2$	36	42	46	50	53	56	58	59	60	61	62	62	62	63	63	63	63
	2.2	$Ml_y = 0.001.Cl_y.q.L_x^2$	36	37	38	38	38	37	36	36	35	35	35	34	34	34	34	34	15
	2.3	$Mt_x = -0.001.Ct_x.q.L_x^2$	36	42	46	50	53	56	58	59	60	61	62	62	62	63	63	63	63
	2.4	$Mt_y = -0.001.Ct_y.q.L_x^2$	36	37	38	38	38	37	36	36	35	35	35	34	34	34	34	34	34

After getting the moment of support or field in each direction X and Y, this moment is used to find the design strength or design stress and is used to find the effective Slab distance. Calculation of Designing Slab stress and effective distance to the diameter of the reinforcement to be tested can be seen in Table

$$R_n = \frac{M}{\Phi b d^2} \text{(Slab Plan Tension)}$$

$$a = \left(1 - \sqrt{1 - \frac{2Rn}{0{,}85fc'}}\right) \text{ (Jarak Efektiv Plat)}$$

Table 43. Calculation of Planned Tension and Effective Distance of Slab 1

Diameter,d(mm)	Reinforcement Type	Coefficient Moment Ly/Lx = 1.7	Moment (kNm)	Design Tension (MPa)	DistanceEffective Slab (mm)
10	X direction field	59	6.107	0.752	3.43
	Y direction field	36	3,726	0.459	2.03
	X direction support	59	6.107	0.752	3.43
	Y direction support	36	3,726	0.459	2.03

From Table, it is possible to calculate the main field and support reinforcement. The first step is to find the required reinforcement area and the minimum reinforcement area because the concrete quality is less than 31.36 MPa. The calculation of the species obtained is then rounded down for every multiple of 25 mm because usually the species are insHeighted at a distance of 150 mm, 175 mm, 200 mm, and so on. The space obtained is used to find the area of reinforcement

$$A_s = \frac{0{,}85a \cdot b \cdot fc'}{fy} \text{.(Required Reinforcement Area)}$$

$$A_{s,u} = \frac{1{,}4b \cdot d}{fy} \text{(Minimum Reinforcement Area)}$$

$$s_1 = \frac{\pi D^2 S}{4As} \text{ (Reinforcement Space; SNI 1727: 2020)}$$

$$s_2 = 2\ h \text{ (Reinforcement Space; SNI 1727: 2020)}$$

$$A_{s,used} = \frac{\pi D^2 S}{4s}$$

Table 44. Calculation of Area and Spacing of Main Slab Reinforcement 1

d (mm)	Reinforcement Type	Reinforcement Area, As(mm2)	Vast Bones, As,u (mm2)	Space, s1 (mm)	Spacing, s2 (mm)	Used Spacing(mm)	Area Used (mm2)	Reinforcement
10	Field X direction	259.74	475	165,413	240	100	150	D10-150
	Field Y direction	157.36	475	165,413	240	125	180	D10-180
	Support X direction	259.74	475	165,413	240	100	150	D10-150
	Support Y direction	157.36	475	165,413	240	125	180	D10-180

The type and area of reinforcement obtained based on conditional conditions will be carried out by 2 checks, namely ULS and SLS checks.

Ultimate Limit State Design(ULS)

In this check, it will be reviewed based on the design moment where the Slab design moment must be smaller than the moment that occurs. In this check, the design moment of the Slab is obtained from the reduced nominal moment of 0.9 (Φ). The value of the design moment will depend on the specification of the reinforcement used, such as the diameter of the reinforcement and the stress of the reinforcement. The first step is to find the height of the compressed concrete stress block. Then, the block height is used to find the nominal moment and is reduced by 0.9 to get the Slab design moment.

$$a = \frac{A_{s,pakai} \times fy}{0{,}85 \times f'c \times b} \text{(Concrete Block Height in Compression)}$$

$$M_n = \frac{As \times fy \times \left(d - \frac{a}{2}\right)}{1000000} \text{ (Momen Nominal Balok)}$$

$$M_d = \Phi M_n \Phi = 0{,}9$$

ULS Slab Checking

Diameter (mm)	Reinforcement Type	Moment (kNm)	Area Used (mm2)	Reinforcement	a(mm)	Mn (kNm)	Md (kNm)	Safe(2.35) <
	Field X	6.107	150	D10-150	3,423	13,427	12,084	Safe

	direction							
10	Field Y direction	3,726	180	D10-180	2073	11,259	10.134	Safe
	Support X direction	6.107	150	D10-150	3,423	13,427	12,084	Safe
	Support Y direction	3,726	180	D10-180	2073	11,259	10.134	Safe

Serviceability Limit State Design(SLS)

It will be reviewed based on the reinforcement ratio. Reinforcement ratio is the ratio between the area of reinforcement to the concrete cross-sectional area where the reinforcement ratio is planned not to be less than the minimum reinforcement ratio and not to be greater than the maximum reinforcement ratio. The use of the reinforcement ratio will affect the deflection of the Slab because if it is smaller, the deflection will be downward, whereas if it is larger, the deflection will be upward. Therefore, the reinforcement ratio must be between the minimum and maximum reinforcement ratios.

$$\rho_{\min} = \frac{1{,}4}{\mathrm{fy}} \times 100\% \text{(Minimum Rebar Ratio) (Maximum Reinforcement Ratio)}$$

$$\rho = \frac{\mathrm{As}}{\mathrm{b} \times \mathrm{d}} \times 100\% \text{ (Ration of Design Reinforcement)}$$

$$\rho_{\max} = \frac{382{,}5 \times \beta 1 \times f'^{c}}{(600 + \mathrm{fy}) \times \mathrm{fy}} \times 1$$

Table 45. SLS Slab Checking

d (mm)	Reinforcement Type	***ρ***min (%)	***ρ*** (%)	***ρ*** max (%)	Check
	Field X direction	0.500	0.551	3,241	Safe
	Field Y direction	0.500	0.459	3,241	Safe
	Support X direction	0.500	0.551	3,241	Safe
10	Support Y direction	0.500	0.459	3,241	Safe

From checking the ULS and SLS, it can be concluded that the specifications of the reinforcement used are reinforcement with a diameter of 10 mm for floor Slab Designing because it complies with the safety of the ULS and SLS.

$A_{s,b} = 20\%xAs, u$(Reduction Reinforcement Area). (Wide Face Press Structure)

$$A_{s,b} = 0{,}002 \times xbh$$

$$s1 = \frac{\pi D^2 s}{4As} \text{ (Take Smallest Value)}$$

$s_2 = 5$ h(Smallest Bar Spacing) (Planned Reinforcement Area)

$$A_s = \frac{\pi D^2 S}{4s}$$

Calculation of Area and Spacing of Support Reinforcement for Slab A1

d (mm)	Reinforcement Type	Reinforced Area n, As, b (mm2)	Reinforced Area n, As, b (mm2)	Space, s1 (mm)	Space, s2 (mm)	Used Spacing(mm)	Reinforcement Area As (mm2)	Sharing Reinforcement
10	Field X direction	-	-	-	-	-	-	-
	Field Y direction	-	-	-	-	-	-	-
	Support X direction	95	240	327.38	600	300	261.90	d10-300
	Support Y direction	95	240	327.38	600	300	261.90	d10-300

From the Designing area and reinforcement spacing, type II Slabs are used with a reinforcement diameter of 10 mm. For the use of reinforcement in one-way Slab s, only the main field reinforcement in the X direction, the main reinforcement in the X direction, and the reinforcement for the Y direction are used. Design results for other floor Slab s.

Beam Designing

Beams are structural elements that are rigid and function to channel them to columns and be forwarded to the foundation so that beams are part of the core structure of the building apart from columns and foundations. Building beams are divided into 2, namely primary beams and secondary beams which have their respective functions. For beam Designing, it is necessary to have pinpoint moments and beam pitch moments which are obtained from the data that has been inputted into the ETABS. In beam Designing, B266 beam will be used as an example of the design because of all the beams with the same dimensions, B266 beam has the greatest moment, namely 239.564 kNm for the support area and 235.56 kNm for the field area.

Beam design consists of Designing longitudinal reinforcement and Designing transverse reinforcement which are located in the support area and field area respectively.

Table 46. Known Data in Design of Beam Reinforcement

Known data	Value	Unit	Information
Concrete quality (f'c)	25	MPa	-
The thickness of the blanket (ts)	40	Mm	-
Reinforcement stress longitudinal (fyiron)	420	MPa	Tension reinforcement which allowable is 420 Mpa
Slab Tension (fysteel)	280	MPa	Tension Slab which allowable is 280 Mpa
Diameterlongitudinal reinforcement (d1)	19	mm	Usually reinforcementlongitudinal diameters are 13 mm, 16 mm, and 19 mm. In this design, a diameter of 19 mm is used
Diametertransverse reinforcement (d2)	10	mm	Usually reinforcementlongitudinal is the diameter 8mm and 10mm. In this design, a diameter of 10 mm is used
Cross-sectional area reinforcement	283,643	mm2	-
longitudinal (As,longitudinal)			-
Cross-sectional area of the transverse reinforcement (As,transverse)	78.57	mm2	-
Beam length(L)	5000	mm	Using one of the beams as an example of Designing, i.e. beam B266
Area negative moment support (Mu-)	239,564	kNm	Based on ETABS results
Area positive moment field (Mu+)	119,782	kNm	Based on ETABS results
Beam height(h)	600	mm	The minimum requirement for beam height is L/12 = 583.33 mm. In this plan, 600 mm high is used

Known data	Value	Unit	Information
Beam width(b)	300	mm	Condition mark wide beam is h/2 so the value is between 300 mm
Distance of compression fiber to center of tension reinforcement (ds)	540.5	mm	The distance from the farthest compressed fiber to center reinforcement pull longitudinal
Distance of compression fiber to center of reinforcement press (d')	hd$_s$= 59.5	mm	The distance from the farthest compressed fiber to center reinforcement press longitudinal
Slab height (hf)	100	mm	Height Slab which inserted into the beam according to SNI 2847:2019
β1	0.85	-	Based on SNI 2847:2019

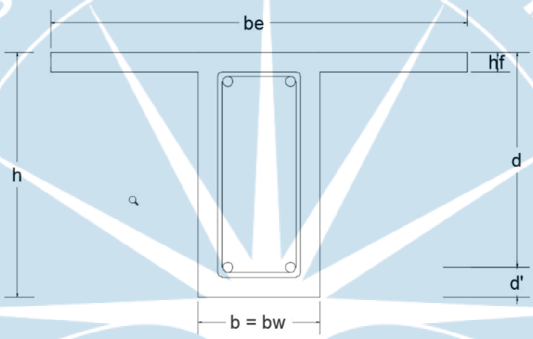

Figure 12. Cross Section of T Beam at Support Area

Design of Longitudinal Beam Reinforcement

Longitudinal reinforcement is the main reinforcement that resists axial loads and moments. The Designing stages of Designing beam longitudinal reinforcement can be seen To calculate the longitudinal reinforcement, the first step is to know the moment in the support area and the field area. To find out this moment, it can be done by looking at the output results on ETABS, By looking at Figure 2.22 in the "Moments" or BMD (Bending Moment Design) section, the left and right sides are the support areas while the middle side is the field area. The greatest moment is chosen between the two support areas. The moment is divided into 2 sides, the upper side is a negative moment and the lower side is a positive moment. Based on the figure, the moment at the support area (Mu-) is 128.42 kNm, while the moment at the field area (Mu+) is 143.71 kNm. The positive moment of the field area used must also meet the requirements as well. To look for positive moments for the support area,

$M_u{}^-$ =based on ETABS results (Moment on Beam)

$M_u{}^{+} = \frac{Mu_{tumpuan}^{-}}{2}$(Moment in Beam) based on ETABS* results (Moment in Beam)

After the positive moments and negative moments in each area are obtained, the next step is to determine the design strength which is the planned strength for that area. Determination of design strength. Then, the next step is to determine the reinforcement ratio to be used. Reinforcement ratio is the ratio of concrete quality to reinforcement stress. Determination of the reinforcement ratio must comply with predetermined requirements. The results of calculating the beam reinforcement ratio can be seen in Table 2.49.

$$R_n = \frac{Mu \times 1000000}{\Phi b d^2}, \text{ dengan } \Phi = 0,9. \quad \text{(Design Strength Calculation)}$$

$$\boldsymbol{\rho}_{min} = \frac{\sqrt{f'c}}{4fy_{besi}} \ \text{(Min Reinforcement Ratio)}$$

$$\boldsymbol{\rho} = \frac{0,85f'c}{fy}\left(1 - \sqrt{1 - \frac{2Rn}{0,85f'c}}\right) \ \text{(Reinforcement Ratio)}$$

$$\boldsymbol{\rho}_{max} = 0,75\left(\frac{0,85 \times f'c \times \beta 1}{fy_{besi}} \cdot \frac{600}{600 + fy_{besi}}\right) \ \text{(Max Reinforcement Ratio)}$$

Table 47. Example of Beam Reinforcement Ratio Designing B266

d1 (mm)	Reinforcement Type			Rn(kN)	***ρ***min	***ρ***	***ρ*** max	***ρ***use
19	Support	Mu-	239.56	3037	0.003	0.008	0.019	0.008
		Mu+	119.78	1,519	0.003	0.004	0.019	0.004
	Field	Mu+	235.56	2,986	0.003	0.008	0.019	0.008
		Mu-	58.89	0.747	0.003	0.002	0.019	0.003

From the reinforcement ratio obtained, the next step is to determine the area of the planned reinforcement. the reinforcement area is still in the form of the assumed reinforcement area. From the area of the reinforcement, then it can calculate the amount of reinforcement needed in the beam. The results obtained for the amount of reinforcement are required to be rounded up so that the amount of reinforcement used has an exact amount. By getting the exact amount of reinforcement, it means that the required reinforcement area can be known. The Designing results for the amount of longitudinal reinforcement can be seen in Table.

Table 48. Amount of Longitudinal Reinforcement Beam B266

d1(mm)	Reinforcement Type		As,necessary (mm2)	Reinforcement Amount	Reinforcement used	As,total(mm2)
19	support	Mu-	1271.01	5	5D19	1418.214
		Mu+	608.87	3	3D19	850,929
	Field	Mu+	1247.86	5	5D19	1418.214
		Mu-	482.59	2	2D19	567,286

In the longitudinal reinforcement plan, ULS (Ultimate Limit State Design) checking will be carried out from the results of the reinforcement plan obtained. In this check, it will be reviewed based on the design moment where the beam design moment must be smaller than the moment that occurs. In this check, the design moment is obtained from the nominal moment which is reduced by 0.9 (Φ).

$$a = \frac{As \times fy_{besi}}{0{,}85 \times f'c \times b} \text{ (Height of Beam).}$$

$$z = d - \frac{a}{2} \text{(Stress of Beam in the cover)}$$

$$T_s = As \text{ x fybesi}$$

$$\Phi M_n = \Phi \times Ts \times z \times 10{-}^6 \text{, with (Moments of Design)} \Phi = 0{,}9$$

Table 49. ULS Checking of B266 Beams

d1(mm)	Reinforcement Type			A (mm)	Z (mm)	Ts (N)	ΦMn (kNm)	Check: Mu n < Design Moments
19	support	Mu-	239.56	93.43	493,782	595650	264.70	Safe
		Mu+	119.78	56.06	512,469	357390	164.83	Safe
	Field	Mu+	235.56	93.43	493,782	595650	264.70	Safe
		Mu-	58.89	37.37	521,812	238260	111.89	Safe

From checking the ULS, it was concluded that the diameter of the longitudinal reinforcement that met the safety requirements was 19 mm, the amount of longitudinal reinforcement in the field area and supports can be seen in Table Longitudinal reinforcement Designing is recommended to use the same reinforcement diameter so that when insHeighting it, workers do not experience confusion about the specifications of the reinforcement in each beam. Beam Reinforcement Designing, the diameter of the longitudinal reinforcement in the beam is 19 mm.

Design of Beam Transverse Reinforcement

Transverse reinforcement is stirrup reinforcement or shear reinforcement which functions to withstand shear forces so as to prevent the occurrence and propagation of cracks so that they do not continue to the concrete compression section. The stages of transverse reinforcement work. Before determining the moment capacity of the beam, what is done is to determine the effective width of the Slab . The effective width of the Slab is taken using the smallest value. The results of the two equations are 1250 mm and 1900 mm respectively, so that the effective width of the Slab used is 1250 mm as the smallest effective width.

be1 = 0.25L (Effective Width of Slab)

be2= b + 16hf(Effective Width of Slab)

*Used be is the smallest between be1 and be2

The next step is to calculate the capacity moment. The beam capacity moment is the planned moment to determine the safety of the designed beam. There are two kinds of capacity moments in the beam, namely positive capacity moments and negative capacity moments.

Moment of positive capacity

Positive moment capacity is the moment capacity that causes bending in the positive direction (concave downward), usually occurs in the beam support area. From Table 2.50, the area of reinforcement for the top of the beam in the support area (As, top) is 1418,214 mm2, while the area of reinforcement for the bottom part of the beam in the support area (As, bottom) is 850,929 mm2, with a Slab height (hf) of 100 mm . From these data, an analysis of B266 beams is carried out, if T is smaller than Ca ($T < Ca$), then B266 analysis is carried out as a rectangular beam so that the effective width of the Slab (be) is equal to the width of the beam (bw) (be = bw).

Cc = 0.85 x f'c x hf x be (Block Compression Force)

fs' = 600 $(1 - \beta 1 \frac{d'}{\;})$ (Tension 10 Blocks)

Cs' = As,topx fs'(Beam Body Stress)

Ca= Cc+Cs' (Total Compressive Force)

T = fyconcretex Axles (Force Balance)

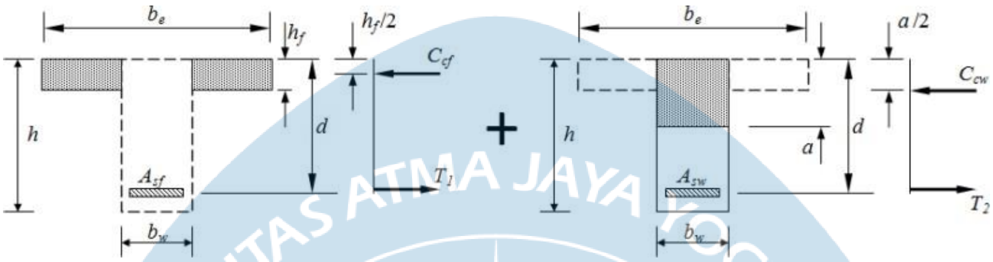

Figure 13. Projection of Compression on the Beam

Before performing the analysis, the T-beam cross-section is separated into 2 parts, namely the flange section and the beam cross-section. On the wing, we will look for the compressive force which will get the compressive force on the wing side of 2,656,250 N. On the body, the calculation is done in stages. it is obtained that the stress on the body of the beam is 296.55 MPa so that the compressive force on the body of the beam is 252,342.9 N. Add up the results and you will get a total compressive force of 420571.4464 N.

Then, check whether the reinforcement has yielded or not. If the reinforcement has yielded, then the section is under tension control, whereas if the reinforcement has not yielded, then a neutral axis is needed where the line that intersects the beam section has no stresses at any point on the line. Checking complaints is done by using

$\frac{A_{s,bawah}-A_{s,atas}}{b_e \times d_s^2} < \beta 1 \frac{0,85f'c}{fy_{besi}}\left(\frac{600}{600-fy_{besi}}\right)\frac{d'}{d_s}$. (Complaint Check)

By checkingyield yield, the result is -1.55346E-06 <0.0158 so that the reinforcement is not yet yielded. Due to the unyielding condition of the reinforcement, a neutral line is required as the capacity moment area. Then do the calculation of the stress towards the Slab area projected in Figure 2.25.

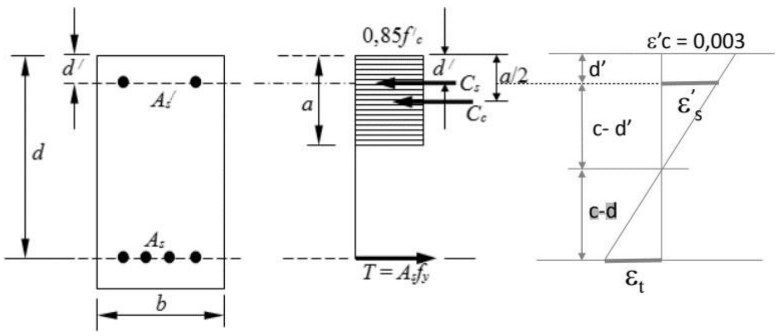

Figure 14. Beam Slab Area Stress

$$c = \frac{a}{0{,}8} \text{ (Obtain Neutral Line)}$$

$$f_s{}' = 600\left(\frac{c - d'}{c}\right) \text{ (Stress on Slab Area)}$$

By calculating the height of the stress block (a) is 56.061mm, the neutral axis is obtained at 65.954 mm so that the Slab area stress is 58.716 MPa. By comparing the Slab area stress with the steel stress, the Slab area stress is used because the Slab area stress is smaller than the steel stress. Furthermore, it can be calculated nominal moment and design moment. If the ULS (Ultimate Limit State Design) check turns out to be safe, then you can proceed with calculating the moment of capacity.

Md = Φ x Mn, where Φ = 0.8 (Bam Design Moment)

Mpr= 1.25 x Mn(Capacity Moment)

Obtained nominal moment is 803.185 kNm, then reduced by 0.8 to get the design moment so that it is obtained at 642.55 kNm. By checking the ULS (Ultimate Limit State Design), by comparing the design moment with the positive ultimate moment, namely 642.55 kNm > 119,782 kNm, it can be said that it is safe. Because the ULS is safe, it can be continued with the calculation of the capacity moment so that a positive capacity moment of 822.56 kNm is obtained.

Negative capacity moment

Negative moment capacity is the moment capacity that causes bending in the negative direction, usually occurs in the field area of the beam. In this plan, the effective stress width of the Slab uses a beam width (bw) of 300 mm. From Table 2.50, the area of reinforcement for the upper part of the beam in the field area (As, above) is 1418.214 mm2, while the area of reinforcement for the lower part of the beam in the field area (As, below) is 850.93 mm2, with a Slab height (hf) of 100 mm .

For the work stage, it is almost the same as working on positive capacity moments, but with a note that the effective Slab width (be) is replaced with beam width (bw) because at negative capacity moments the beam width parameter is used. On the wing, you will find the compressive force, where you will get the compressive force on the wing side which is equal to 637500 N. In the body, the calculation is done in stages. it is obtained that the stress of the beam body is 296.55 MPa so that the compressive force of the beam body is 252342.8679 N. The balance of forces in the T beam is 595650 N.

In Designing the transverse reinforcement in the support area, a shear force is needed in that area, which in this case is the B266 beam. The shear force obtained from the ETABS output is the basis for the design of the transverse reinforcement. Then, calculations are carried out to see the shear forces that occur in the projected beam sections in calculated Ve1 and Ve2 and with the beam clear span parameter (Ln) with the planned column dimensions of 300 x 600 mm.

$$L_n = L - b_{kolom}$$

$$V_{e1} = \frac{Mpr^{+} + Mpr^{-}}{Ln} + Vg.$$

$$V_{e2} = \frac{Mpr^{+} + Mpr^{-}}{Ln} - Vg.$$

The result of the shear force Ve1 and shear force Ve2 are 349.1526868 kN and 308.4326868 kN respectively. The theoretical calculation of shear forces in plastic hinges is carried out. To maintain the safety of the transverse reinforcement, the shear force is divided by 0.8 because there is an assumption that the shear force that occurs can be greater.

$$V_e = \left\{\frac{Ln - d}{Ln}(Ve1 - Ve2)\right\} + Ve2$$

$$V_s = \frac{Ve}{\Phi}, \text{dengan } \Phi = 0{,}8$$

it is found that the shear force that occurs theoretically is 344.3680868 kN, but again that the shear force that occurs in reality can be greater, so it is divided again by the reduction factor of 0.8. Thus, the planned shear force that can occur is 459.1574491 kN. Calculation of the use of transverse reinforcement in the support area is carried out by using 2 pieces of reinforcement with a diameter of 10 mm because it is a standard amount, but the amount of reinforcement can be added as needed. The use of reinforcement spacing used is usually a multiple of 25 mm, such as 25 mm, 50 mm, 75 mm, and so on.

$$A_{vtotal} = n \times A_v$$
$$S = \frac{\text{As total} \times \text{fy steel} \times d}{Vs}$$

The total area of reinforcement for transverse reinforcement in the support area is 157.14 mm2 with a spacing of 132 mm. Again, the reinforcement spacing is every multiple of 25 mm, so the spacing for transverse reinforcement is 125 mm rounded down.

Field area (outside plastic hinge)

The design of transverse reinforcement uses quite different parameters in calculating the shear forces because the ETABS output does not display shear forces in the field area. Therefore, the shear force in the field area is theoretically calculated based on the obtained shear force of 135.25 kN. The calculation of the shear force outside the plastic hinge is also similar to the calculation of the plastic hinge, the difference is the length of the field area. the shear force that occurs is -287.6300781 kN. Assuming that the shear force is still theoretical, then the shear force is divided by the reduction factor and reduced by the field area so that the shear force that can occur is -518.6317708 kN.

$$V_c = \frac{1}{6}\sqrt{f'c} \times bw \times d$$
$$V_{s1} = \frac{Ve}{\Phi} - Vc, \text{ with } \Phi = 0{,}8$$
$$V_{s2} = \frac{1}{3}\sqrt{f'c} \times bw \times d$$

* the largest shear force is selected

Calculation of the use of transverse reinforcement in the field area is carried out using 2 pieces of reinforcement with a diameter of 10 mm because it is the standard amount in the field area, but the amount of reinforcement can be added as needed. The total area of reinforcement for transverse reinforcement in the support area is 157.14 mm2 with a spacing of 132 mm. Again, the reinforcement spacing is every multiple of 25 mm, so the spacing for transverse reinforcement should be 125 mm rounded down, but with a note that the spacing of stirrup reinforcement in the field area should not be more than spaced in the support area, then the transverse reinforcement spacing for the field area is 100 mm.

Column Designing

Column is a structural component of a building whose function is to support loads from beams and Slab s and transmit them to the ground through the foundation. The strength of the column in carrying loads is based on its ability to carry a combination of axial load (Pu) and moment (Mu) simultaneously. Therefore, the column design of a building structure is based on the strength and stiffness of the cross section against axial load action and bending moment.

Table 50. Column Known Data

Known Data	Value	Unit	Information
Concrete Quality (f'_c)	25	MPa	-
Concrete Cover (d_s)	40	mm	-
Reinforcing Stress (fy)	420	MPa	Used reinforcement screw so Steel Tension is 420 MPa
Column Width (b)	600	mm	Dimensions assumeddepends on condition cross-sectional dimensions in SNI 2847:2019
Column Length (h)	600	mm	Dimensions assumeddepends on condition cross-sectional dimensions in SNI 2847:2019
Column Height (l)	3500	mm	In accordance height between floor building design
Reinforcement Diameter Longitudinal (d_1)	18	mm	Selection based on the most suitable size of the spColumn application
Reinforcement Diameter Transverse (d_2)	11	mm	Selection based on the most suitable size of the spColumn application

It is known that the dimensions of the column Designing to be used are 600 x 600 mm. In this plan, the column dimensions used only use 1 type of dimension because the planned building has 3 floors. From the initial assumption of the column dimensions, the column dimensions will be checked according to the column dimension requirements in SNI 2847:2019 Article 18.7.2.1. In these regulations, it is required that the column cross-sectional dimension is not less than 300 mm (b > 300 mm) and the ratio of the smallest cross-sectional dimension to the perpendicular dimension is not less than 0.4 (b/h > 0.4). Based on the planned column dimensions of 600 x 600 mm, the initial column dimensions have met these requirements.

After obtaining the column dimensions that will be used in Designing, the next step is to check the column slenderness requirements. Column slenderness check is an inspection carried out to check the possibility that the column design will experience buckling before reaching the limit state of material failure. Because the Designing of this column uses only 1 type of column dimension, the slenderness check can be carried out only on one of the columns. This inspection will review column C2 which is located on the 2nd floor. The first step is to find the value of the elastic modulus of concrete (Ec). Then look for the value of the moment of inertia in the x (Ix) and y (Iy) planes of the column section. Because the dimensions of the column section are square, then the value of the moment of inertia for the x field is the same as the y field. From the value of Ec and the moment of inertia of the column cross-section, it can be calculated the flexural stiffness of the compressed structure (EI). In addition to the column structural components, the moment of inertia and the bending stiffness of the beam structure related to the column under consideration can also be calculated. In column C2, it is found that there is Block B12 on the left and Block B325 on the right. The value of the moment of inertia of the beam can be found and the bending stiffness. From this value, the ratio of the stiffness of the beam and column (Ψ) will be calculated. The results of the calculation of these components can be seen in Table 2.53. It is found that there is a B12 beam on the left and a B325 beam on the right. The value of the moment of inertia of the beam can be found and the bending stiffness. From this value, the ratio of the stiffness of the beam and column (Ψ) will be calculated. The results of the calculation of these components can be seen in. It is found that there is a B12 beam on the left and a B325 beam on the right. The value of the moment of inertia of the beam can be found and the bending stiffness. From this value, the ratio of the stiffness of the beam and column (Ψ) will be calculated. The results of the calculation of these components can be seen in

$$E_c = 4700 \times \sqrt{f'c}$$

$$I_x = 0{,}7\left(\frac{1}{12} \times b \times h^3\right)$$

$$E_I = E_c \times I_x$$

$$I_X = 0{,}35\left(\frac{1}{12} \times b \times h^3\right)$$

Table 51. Calculation of Column Slenderness Checking Components

Data	Column C2	Beam B12	B325 beam	Unit
Length(L)	3500	3000	5000	mm
Column Net Length (Lu)	2900	-	-	mm
Concrete Elastic Modulus (Ec)	23500	23500	23500	MPa
The moment of inertia (I_X)	7560000000	1890000000	1890000000	mm4
Flexural Stiffness Press Structure (E_I)	1.7766E+14	4.4415E+13	4.4415E+13	mm2
Beam and Column Stiffness Ratio (Ψ)	6.42857			

From the calculation, the stiffness ratio is 6.42857. The stiffness ratio will be used to find the value of the column length factor (k) using the help of the nomogram obtained from SNI 2847:2019. In this design, structural components are assumed to be immovable structural components, so to find the value of k, the nomogram of immovable structural components can be used. Because the planned building only has 3 floors, the other stiffness ratios are considered to be 0. The results of the k value analysis using the nomogram can be seen in Figure.

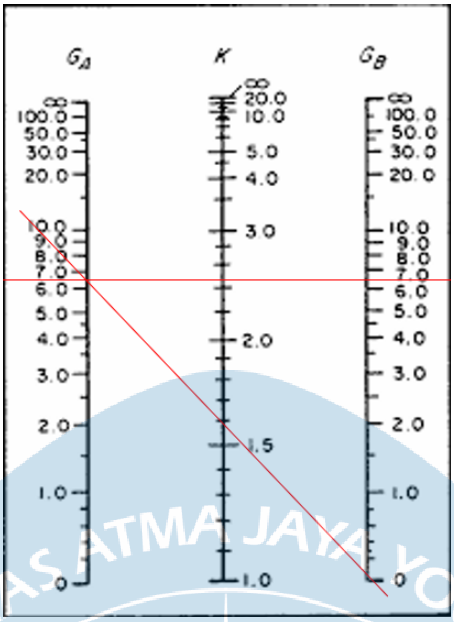

Figure 15. Nomogram of Unswayed Structural Components

In the nomogram image, input the value of the stiffness ratio that has been obtained before. As for the other stiffness ratios, they are considered to be of value. Then the two points are connected by a line and the intersection of the lines is sought with the k value scale in the middle.

$$\frac{k \times L_u}{r} \leq 34 - 12 \times \left(\frac{M_1}{M_2}\right)$$

Column Longitudinal Reinforcement Design

After checking the effect of the slenderness of the column, the next step is to plan the longitudinal reinforcement in the column. The longitudinal reinforcement serves to withstand axial loads and bending moments that occur due to loading. The Designing stage of the column longitudinal reinforcement can be seen in Figure

The load acting on the column is usually a combination of axial load and bending moment. The magnitude of the axial load and bending moment that the column can withstand depends on the dimensions of the foundation and the amount of reinforcement insHeighted. The relationship between axial load and bending moment is described in a diagram called a column interaction diagram which can provide an overview of the strength of the designed column. Column interaction diagrams are usually made by taking into account the strength of the column based on the 6 load conditions on the column cross section which are obtained from the column ETABS output results as shown

Table 52. ETABS In Column C2 floor 2

Data	P(kN)	M2(kNm)	M3(kNm)
P max	-350.1	-93,298	-106.14
P min	18.8995	1.5866	90.7897
M2max	-182.4955	146.6785	81.2177
M2min	18.8995	1.5866	90.7897
M3max	-350.0965	-93.2976	-106.1445
M3min	-18.9901	3.8959	-1,141

The load and moment conditions are given which will be used in modeling the column interaction diagram. These conditions are at the minimum and maximum loading, at the minimum and maximum x-plane moments and at the minimum and maximum y-plane moments. In this plan, the column interaction diagram is modeled with the help of the spColumn application. In the spColumn, input the column dimensions of 600 x 600 mm and the assumption that the diameter of the flexural reinforcement is 25 mm as many as 8 pieces. From the modeling results, the column interaction diagram shown in Fig

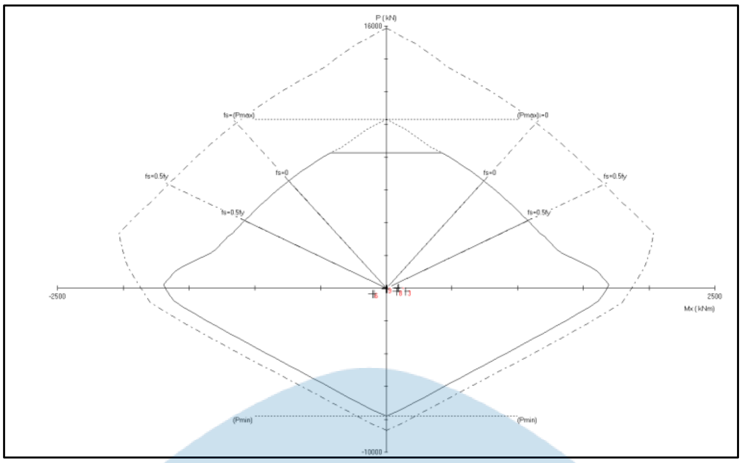

Figure 16. Column Interaction Diagram

Table 53. SpColumn Modeling Output Results

Pu(kN)	Mux(kNm)	Muy(kNm)	ϕMnx(kNm)	ϕMMrs (kNm)	ϕMn/mu	εt	ϕ	Mnx (kNm)	Mny (kNm)
350.1	93,298	106.1	1652.2	1652.2	17.77	0.0055	0.9	1835.8	1835
18.89	1.5866	90.78	1680.9	1681.0	18.68	0.0051	0.9	1867.7	1867
182.4	146.67	81.21	1665.2	1665.3	11.33	0.0054	0.9	1850.3	1850
18.89	1.5866	90.78	1681.0	1678.0	840.51	0.0051	0.9	1867.8	1864
350.0	93,297	106.1	1652.2	1652.2	17.77	0.0055	0.9	1835.8	1835
18.99	3.8959	1,141	1678.0	1681.0	419.52	0.0052	0.9	1864.5	1867

In the table, it can be seen that the ratio of the nominal moment capacity (ϕMn) to the bending moment that occurs (Mu) is more than 1. This indicates that the planned column design is safe in resisting the moment that occurs.

Table 54. Column Longitudinal Reinforcement Calculations

Known Data	Value	Unit	Information
RatioRepetition (p)	0.0106		Results modelingcolumn using spColumn
Check reinforcement ratio configuration	0.01 < p < 0.06 = 0.01 < 0.0106 < 0.06		The reinforcement ratio should be within the range 0.01 to 0.06
Total Column Cross-sectional Area (Ag)	$b \times h = 600\times 600 = 360000$	mm2	

Known Data	Value	Unit	Information
Total Reinforcing Surface Area (As total)	p× Ag = 3808.8	mm2	-
WideReinforcement Surface (As tul)	1/4×π×d2= 490.87	mm2	-
Number of Reinforcement (n)	As Total/As Reinforcement = 7,759 = 8	piece	-
Used Reinforcing Surface Area (As use)	8 x ¼ x π × d2 = 3927	mm2	-
Reinforcement Longitudinal	8D25	-	-

From the calculations and the results of the spColumn modeling, the longitudinal reinforcement in the column uses 8D25 reinforcement. The next step is to check the nominal moment of the column using the Strong Column - Weak Beam (SCWB) concept. This concept is the concept of a structural system, namely a special moment-bearing frame system. In this system, the column is planned to be stronger than the beam connected to the column. An overview of the SCWB concept can be seen in Figure

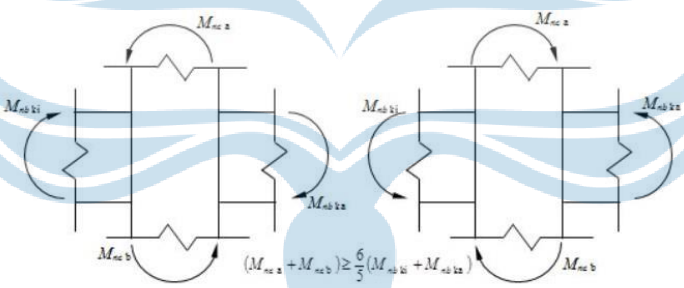

Figure 17. Strong Column Concept - Weak Beam (SCWB)

From the results of these checks, it is known that the planned column meets the requirements in the SCWB concept, namely Mnc > 1.2 Mnb. Therefore,

Table 55. Checking Strong Column - Weak Beam (SCWB) Concept

Data	Value	Unit
Nominal Moment Column C2 (Mnb)	32.97	kNm
Nominal Moment	43.33	kNm

Data	Value	Unit
B12 beam (M_{nb})		
Nominal Moment B325 beam (M_{nc})	1835.8	kNm
Check	$\sum Mnc > 1.2 \sum Mnb$ 2 × 1835.8 > 1.2 × (32.97+43.33) 3671.6 > 91.56 (OK)	kNm

Column Transverse Reinforcement Design

After obtaining the details of the longitudinal reinforcement to be used, the next step is to plan the transverse reinforcement of the column. Transverse reinforcement is useful to help concrete withstand shear forces that occur due to loading. In columnar structures, transverse reinforcement is divided into reinforcement in the plastic hinge area and outside the plastic hinge area. The column plastic hinge area is located at the end of the column in contact with the beam. Meanwhile, the column length between plastic hinges is referred to as the area outside the plastic hinge. In Designing for transverse column reinforcement, it is necessary to calculate and check the requirements in accordance with those contained in SNI 2847:2019. The Designing of column transverse reinforcement in the plastic hinge area and outside the plastic hinge can be seen in table below.

Table 56. Column Transverse Reinforcement Designing

Transverse Reinforcement of Plastic Joint Regions		
Length of the Plastic Joint Zone (Lo)	h = 600	mm
	Lu/6 = 483.33	mm
	450	mm
Spacing Between Rebars Longitudinal (p_x)	(b-2(ds+d2)-d1)/2 = 236.5	mm
Distance of Transverse Reinforcement (s)	b/4 = 150	mm
	6d1 =6 x 25 = 150	mm
	100 + (350-hx/3) = 137.833	mm
Effective Column Cross-sectional Width (bc)	b-2 (ds + d2/2) = 509	mm
Cross-sectional area Effective (A_{ch})	bc x bc = 259081	mm2

Cross-sectional area Column (Ag)	bxh = 3600	mm2
Reinforcement Requirements (Ash / Sbc)	0.3 (Ag/Ach - 1) x fc/fy =0.006955	
	0.09 x fc/fy = 0.005357	
WideReinforcement Surface (Ash)	0.00537 xsx bc = 376.7430739	mm2
Transverse Reinforcement Plastic Joints	3D13-150	-
Transverse Reinforcement of Regions Outside of Plastic Joints		
Distance of Transverse Reinforcement (s)	$6 \times d1 = 6 \times 25 = 150$	mm
	150	mm
Transverse Reinforcement Outside Plastic Joints	3D13-150	-

In examining the shear strength, the inspection procedure will use the provisions contained in SNI 2847:2019 article 18.7.6.1.1. In these provisions, the shear force (Ve) to be used must be determined from a review of the maximum forces that occur at the face of the joint at each end of the column. The shear force at the end of the column (Ve1) must be determined from the maximum flexural strength (Mpr) that occurs assuming the tensile strength of the longitudinal reinforcement is 1.25fy. The shear force at the end of the column (Ve1) need not exceed the value of the shear force at the beam joint (Ve2) based on the bending moment of the beam. In addition, the value of Ve cannot be less than the factored shear force (Ve3) based on the results of structural analysis using ETABS.

Table 57. Column Shear Strength Check

Shear Strength in Plastic Joint Areas		
Column Maximum Moment Strength (Mprc)	146.6785	kNm
Distribution Factor (DF)	0.5	-
Maximum Moment Strength of Left Beam (Mprb1)	22.7512	kNm
Maximum Moment Strength of Right Beam (Mprb2)	28,297	kNm

Shear Strength in Plastic Joint Areas		
Column End Shear Force (Ve1)	(Mprc Top + Mprc Bottom1)/Lu = 58.42	kN
Shear Force At Joint Beam (Ve2)	(Mprc Top + Mprc Bottom2)/Lu = 60.34	kN
Factored Shear Force (Ve3)	39.9866	kN
Effective Thickness (d)	b - ds - d2 - d1/2 = 600 - 40 – 11 – 18/2 = 540	mm
Shear Strength of Reinforcement (Vs)	Ve/ϕ= 60.34/0.75 = 80.453	kN
Minimum area Shear Reinforcement (Av)	(Vs x S) / (fy xd) = (80.453 x 150) / (420 x 540) = 0.0532	mm2
Shear Strength in the Outer Area of Plastic Joints		
Maximum Load on Column (Nu)	90.79	kN
Shear Strength of Concrete (Vc)	0.17 x (1 + Nu/14Ag)λ$\sqrt{fc}$xbxd = 0.17 x (1 + 90.79/14 x 360000) x 1 x (25^1/2) x 600 x 540 = 1.60737E+12	kN

In the table above, the column moment strength (Mprc) is obtained from the spColumn assuming the tensile strength of the reinforcement is 1.25fy. The distribution factor used in the calculation is 0.5 because the top and bottom column dimensions are the same. The maximum moment strength value of the beam (Mprb) is obtained from the nominal moment value of the beam connected to the end of the column assuming the tensile strength of the reinforcement is 1.25fy. From this value, the value of the design shear force will be sought in accordance with the provisions contained in SNI 2847:2019 article 18.7.6.1.1. In these provisions there are 3 design shear force values that need to be reviewed, namely column end shear force (Ve1), shear force at beam joints (Ve2) and factored shear force (Ve3). Of the three values, the value of Ve1 does not need to exceed Ve2, but must not be less than the value of Ve3, so the value Ve2 = 60.34 kN is used. After obtaining the value of the shear force, it can be calculated the shear strength of the reinforcement and the minimum area of shear reinforcement required in the plastic hinge area. As for the shear force in the area outside the plastic hinge, the shear strength under consideration is the concrete shear strength (Vc).

From the calculation and Design of the shear forces in the table above, it is found that the minimum area of shear reinforcement is still below the surface area of the transverse reinforcement that has been used in plastic joints. So that the 3D13-100 transverse reinforcement can still be used in plastic hinges. As for the area outside the plastic hinge, it is known that the concrete shear strength (Vc) is able to withstand the shear force resulting from structural analysis (Ve3). Therefore, the transverse reinforcement in the outer region of the plastic hinge, namely 3D13-150, can still be used. Based on these inspections, it was concluded that the designed transverse reinforcement was safe against the shear forces that occurred.

Beam-Column Relations

The beam-column relationship is the meeting area between the column and the beam which must be well detailed. This meeting area is a critical area in a reinforced concrete frame structure. This is because the consequences arising from the moment of the column above and below it and the moment originating from the beam will cause the area to experience a large shear force. In the area of the beam-column connection, there will be a tensile force (T) and also a compressive force

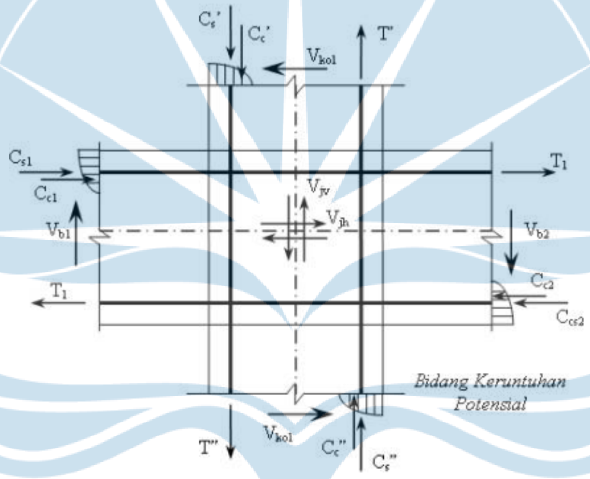

Figure 18. The Forces Acting on Beam-Column Relationships

In the figure, it can be seen that the beams stretching on the column will cause a compressive force and a tensile force on the joint. These forces arise due to the longitudinal reinforcement in the beam. Therefore, the force in the longitudinal reinforcement of the beam in front of the beam-column connection must be determined assuming that the flexural tension reinforcement stress is 1.25fy

Table 58. Design Data of Beam-Column Relations

Data	Value	Unit	Information
Distribution Factor (DF)	0.5	-	Used 0.5 due to dimensions columnon and below the same

Data	Value	Unit	Information
Left Beam Moment Capacity (Mpr)	32.97	kNm	The moment used in Beam B12 is obtained by assuming the stress 1.25fy steel
Left Beam Reinforcement	4D19 (A_s= 1253mm2)	-	On beam left,the reinforcement being reviewed is the top reinforcement of the beam
Right Beam Moment Capacity (Mpr)	43.33	kNm	The moment used in the B325 beam is obtained by assuming the stress 1.25fy steel
Right Beam Reinforcement	2D19 (A_s= 630.21mm2)	-	In the right beam, the reinforcement under consideration is the bottom reinforcement beam
Joint Width (b)	300	mm	Obtained from wide beam cross section
Joint Height (h)	400	mm	Obtained from dimensions column cross section
Width outside Joints (x)	(hb)/2 = (400-300)/2 = 50	mm	The remaining column widths are not connected beam
Effective Joint Width (bj)	$b + h$ = 300 + 400 = 700	mm	Based on SNI 2847:2019 chapter 18.8.4.3, the smallest value is 400 mm
	$b + 2(x)$ = 300+2(50) = 400	mm	
Effective joint area (Aj)	$bj \times bj$ = 600 × 600 = 360000	mm2	-

In this plan, column C2 will be reviewed with beams that frame the column, namely beam B12 (left) and beam B325 (right). Because the amount of reinforcement in the right and left beams is the same, the compressive and tensile forces on both sides will have the same value. From these data will be calculated the shear force at the joints of the beam-column connection.

Table 59. Calculation of Joint Shear Force on Beam-Column Relations

Data	Value	Unit	Information
Beam Joint Shear Force (Vu)	$\sum M_{prb\ atas} \times DF + \sum M_{prb\ bawah} \times DF \,/\, Lu$ = 29,883	kN	Shear force due to beam moment

Data	Value	Unit	Information
Left Beam Tensile Force (T1)	$As(1{,}25fy)$ = 1253 (1.25 × 420) = 657825 = 657.825	kN	Tensile force due to the top reinforcement beam (4D19)
Right Beam Tensile Force (T2)	$As(1{,}25fy)$ = 630.21 (1.25 × 420) = 330860.3 = 330.86	kN	The resulting tensile force by reinforcement lowerbeam (2D19)
Press Style Left Beam (C1)	Q1=657,825	kN	Has the same value as the Tensile force because the reinforcement between the two beams is the same
Right Beam Compression Force (C1)	Q2=330.86	kN	
Sliding Style joint(Vj)	$T1+C2-Vu$ = 958,803	kN	-
Strong Shear joint(Vn)	$1{,}2\lambda\sqrt{fc}\times Aj$= 1.2 x 1 x √25 x 16000 = 2160000 = 2160	kN	Based on SNI 2847:2019 article 18.8.4.1, strong formula shift nominal value for the beam-column connection which restrained on three sides
CapacityStrong Shear Joint (ϕVn)	$\phi\times Vn$ = 0.75× 2160 = 1728	kN	-

From the calculation resultsFrom this, the value of the joint shear force in the beam-column connection is 958,803 kN. Meanwhile, the value of the shear strength capacity of the beam-column connection is 1728 kN. Because the value of ϕVn > Vj, the design of transverse reinforcement can be used in beam-column joints. So that in the area of the beam-column connection, stirrup reinforcement 3D13-100 will be used, the same as in the transverse reinforcement column in the plastic hinge area.

Conclusion

In Designing the superstructure of a building, the first step is to plan the superstructure because the load of the structure will support the load of the structure above it. For example, in the

Gunung Kidul Nursing Home building, the roof will support the wind load and the roof itself, the 3rd floor will support the roof load and the 2nd floor will support the roof load, the 3rd floor load, and the 2nd floor itself, and so on. It can be concluded from this statement that basically, the structure will support its own load and the load on it. Each design of the structure will be checked for safety with the Ultimate Limit State Design (ULS) and/or Serviceability Limit State Design (SLS) parameters. The ULS parameter will compare the stress that occurs to the stress of the structural material,

In buildings, the first step is to plan the roof, be it wind loads, live loads and dead loads on the roof. Roof Designing consists of Designing Purlin and truss Designing. Based on the Designing that has been done, the Purlin used is a C100x50x20 profile with a thickness of 2 mm because besides being safe based on ULS and SLS parameters, the costs incurred are cheaper because it has a thinner thickness than the profiles that have been tested before. The required sag-rod used is a diameter of 10 mm and a diameter for wind rods is 16 mm. For truss Designing, the truss profile used is a 2L50x50x5 profile with 2 threaded bolts of type A-325 with a diameter of 12 mm for each connection. After the roof Designing has been completed, structural Designing can be carried out, the first is the Designing of stairs in the form of Designing of stairs and Designing of landing beams. Based on the Designing results, it was found that the height of the steps is 170 mm and the width is 350 mm with the number of steps being 20. In the areas of support and field, the principal reinforcement and reinforcement used are D13-150 and d8-150. For the design of landing beams, after checking the ULS, flexural reinforcement 3D13 is used in the support area and 2D13 in the field area. For shear reinforcement it takes d8-75 in the support area and d8-150 in the field area. it was found that the height of the steps is 170 mm and the width is 350 mm with the number of steps being 20 units. In the areas of support and field, the principal reinforcement and reinforcement used are D13-150 and d8-150. For the design of landing beams, after checking the ULS, flexural reinforcement 3D13 is used in the support area and 2D13 in the field area. For shear reinforcement it takes d8-75 in the support area and d8-150 in the field area. it was found that the height of the steps is 170 mm and the width is 350 mm with the number of steps being 20 units. In the areas of support and field, the principal reinforcement and reinforcement used are D13-150 and d8-150. For the design of landing beams, after checking the ULS, flexural reinforcement 3D13 is used in the support area and 2D13 in the field area. For shear reinforcement it takes d8-75 in the support area and d8-150 in the field area.

Then, an analysis of the distribution of seismic forces was carried out in the Gunung Kidul area as a construction site to determine the lateral forces that occur on each floor. The loading results for each floor are inputted into the ETABS. After that, Slab Designing is carried out on the reinforcement in the support area and the field area. As a reference, use the Designing on Slab 1 by using class II Slabs with their respective moment coefficients. Based on the analysis of ULS and SLS, it was found that the main reinforcement and the main reinforcement were used. Next is beam Designing because it is one of the structures that supports the floor load that occurs. Beam Designing will use B266 beams as a reference. Beam design consists of transverse reinforcement and longitudinal reinforcement. Longitudinal reinforcement Designing found that the diameter of the reinforcement used was 19 mm with a different amount of reinforcement according to the needs of the support or field area. In the transverse reinforcement, the diameter of the reinforcement used is 10 mm with a number of needs that vary according to the area of support or field.

For column Designing, column C2 is used as Designing reference. Column design also consists of transverse reinforcement and longitudinal reinforcement. For longitudinal reinforcement, 8D25 reinforcement is used which is safe after ULS checking based on the Strong Column - Weak Beam (SCWB) concept. For transverse reinforcement, in the area of abutments or plastic hinges, the reinforcement used is 3D13-100, while in the field area or outside the plastic hinges is 3D13-150.

To connect beams and columns, beam-column relationship Designing is carried out. The columns used in this plan are Column C2, while the beams used are Beams B12 and B18 because they are one unified structure. To connect the beams and columns, based on the design, transverse reinforcement 3D13-100 is used because the value of the shear strength capacity is greater than the joint shear force between the beams and columns.