CHAPTER II
SUPER STRUCTURE

2.1. Introduction

Super structure consists of roof structure, beam, column, beam column joint, slab, and stair. This structure is located above the soil surface, with exception of tie beam that is located right on soil surface. Mainly, the material of super structure is reinforced concrete for beam, column, slab, and stairs. Meanwhile, the roof structure uses steel structure for purlin and truss.

The design for super structure will begin from site class determination to determine the structure system. Then, preliminary design by using modeling software like ETABS to know the internal forces from inputted various loading that has been calculated. Next, the internal forces output is analyzed by using several regulations and requirements to check the strength of structural design. The regulations that are used in this design are as follows:

- SNI 1726:2019 Perencanaan Ketahanan Gempa
- SNI 1727:2013 Beban Minimum untuk Perancangan Bangunan Gedung dan Struktur Lain
- SNI 1729:2015 Spesifikasi untuk Bangunan Gedung Baja Struktural
- SNI 2847:2019 Persyaratan Beton Struktural untung Bangunan Gedung
- Peraturan Beton Bertulang Indonesia (PBI) tahun 1971

The general procedure of super structure design is shown in Figure 2. 1.

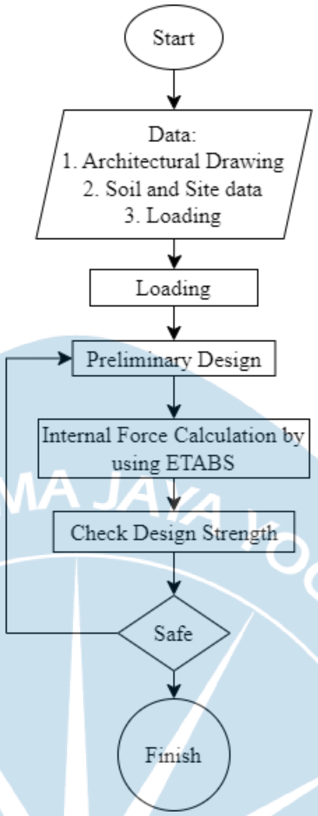


Figure 2. 1 General Super Structure Flowchart

2.2. Soil Interpretation Data and Site Class Determination

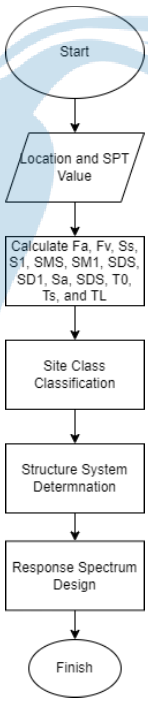


Figure 2. 2 Soil Interpretation Data Flowchart

The soil interpretation is based on the bore log of Standard Penetration Test that was given to represent the soil data of the site. To interpret the data, standard from SNI 1726:2019 is used. The procedure of soil data interpretation is shown in Figure 2. 2.

The risk category for building can be seen in SNI 1726:2019 table 3. There are 4 categories which are risk I, II, III, and IV. This structure is on risk category II which is an apartment. The earthquake priority factor is determined based on the building risk category. The earthquake priority factor (Ie) is presented in accordance with SNI 1726:2019 table 4. In this work, the structure of the residential house is included in building risk category II so the earthquake priority factor (Ie) is 1.00. Ground acceleration parameters (Ss and S1) are affected by soil properties at project location. The values of Ss and S1 are used to determine the spectral response the acceleration of the MCER earthquake at ground level, where Ss and S1 are respectively the spectral response parameter of the mapped MCER earthquake acceleration for short periods and 1.0 second periods. The values of SS and S1 are based on the construction location and can be taken from the official website of Spectra Design Indonesia rsa.ciptakarga.pu.go.id which are 1.28 and 0.5457 for SS and S1 respectively. All these parameters can be seen in Table 2. 1.

Characteristics of the project site, especially those related to aspects geotechnical engineering, must be properly identified in the planning process through site investigation activities. The results of the investigation of the project site will be used as a basis for determining site classification. In SNI 1726:2019 table 5, sites are classified into 6 types, namely SA (hard rock), SB (rock), SC (hard soil), SD (medium soil), SE (soft soil), and SF (special site). The classification can use N SPT Value from Bore Log, where it shows that the site is classified as SD class because the $\overline{N}_{ch}$ value is in the range of 15 until 50 (Table 2. 2).

Table 2. 1 Site Parameter

Risk Category	II
Ie	1
Ss	1.28
S1	0.5457

Table 2. 2 Calculation of Class Site Classification

Depth (m)	**Material**	**SPT N Value**	**Thickness (m)**	**Thickness/N**
2	Medium Sand	23	2	0.08696
4	Medium Sand	34	2	0.05882
6	Medium Sand	36	2	0.05556
8	Medium Sand	36	2	0.05556
10	Medium Sand	38	2	0.05263
12	Medium Sand	40	2	0.05000
14	Medium Sand	42	2	0.04762
16	Rough Sand	44	2	0.04545
18	Rough Sand	45	2	0.04444
20	Rough Sand	50	2	0.04000
22	Rough Sand	52	2	0.03846
24	Rough Sand	52	2	0.03846
26	Rough Sand	54	2	0.03704
28	Rough Sand	55	2	0.03636
30	Rough Sand	55	2	0.03636
32	Rough Sand	57	2	0.03509
34	Rough Sand	58	2	0.03448
36	Rough Sand	60	2	0.03333
38	Rough Sand	60	2	0.03333
40	Rough Sand	60	2	0.03333
42	Rough Sand	60	2	0.03333
44	Rough Sand	60	2	0.03333
45	Rough Sand	60	1	0.01667
Σ			45	0.97663
$\bar{N}$ch= ΣT/Σ(T/N)	46.07672191	Site Class SD		

2.3. Structure System Determination

The designed structure must be defined as belonging to the design category seismic (KDS) in accordance with SNI 1726:2019 table 8 and 9 based on the relationship SDS and SD1 with KDS. From that table, the seismic design for S_{DS} and S_{D1} is D.

The seismic force resisting structural system is decided based on the seismic potential of location. In this case, location with high potential of earthquake is recommended to use earthquake-resistant special moment structural systems or SRPMK (*Sistem Rangka Pemikul Momen Khusus*) so the structure parameters are obtained is R equals 8, Cd equals 5.5, and omega equals 3 (see Table 2. 3). Parameters R, Cd, and omega for each type of seismic force resisting structural system is can be seen in SNI 1726:2019 table 9.

Table 2. 3 Structure Parameters

Response Modification, R	8
System Overstrength, Ω	3
Deflection Amplification, Cd	5.5
Occupancy Importance, I	1

2.4. Structural Loading

Structural Loading is a weight or force that acts on structural system. It is divided into 5 types, which are dead load, additional dead load, live load, wind load, and earthquake load. This load must be resisted by the structure according to the design and strength. Therefore, the analysis of load is essential to check the maximum load that exerts on the structure. The load is regulated based on SNI 1727:2013.

2.4.1. Load Combination

Load combination is defined according to SNI 1727:2013 article 2.3.2. The load combinations are explained as follows:

Comb 1: 1.4D

Comb 2: 1.2D + 1.6L + 0.5Lr

Comb 3.1: 1.2D + 1.6Lr + L

Comb 3.2: 1.2D + 1.6Lr + 0.5W

Comb 3 Service: D + ADL + L

Comb 4: 1.2D + W + L + 0.5Lr

Comb 5: 0.9D + W

Comb 6: 1.2D + L + Ex + Ey

Comb 7: 0.9D – Ey + Ex

2.4.2. Dead Load (Self Weight)

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

2.4.3. Additional Dead Load

Additional dead load is an additional load due to the use of non-structural components (architectural) that are attached to and burden the main structure of the building. The additional dead load in this structure are roof, ceramics, and ceiling.

2.4.4. Live Load

Live load is the load that occurs as a result of the use of the building structure. The live load can come from people/goods that can move from place to place. Apartment is included in the category of residential according to SNI 1727:2013 table 4.1, so the live load is 4.79 kN/m2.

2.4.5. Wind Load

Wind load is a type of load that must be resisted by the roof structure. It can act in two directions, in which windward and leeward. The procedure of wind load calculation is presented in SNI 1727:2013 chapter 26.

Table 2. 4 Wind Load Parameter

Parameter	**Value**	**Unit**	**SNI 1727:2013**
Building Width, B	96.0	m	
Building Length, L	28.0	m	
Wall Height	23.0	m	
Soil to Roof Height	25.2	m	
Effective Height, h	23.7	m	
L/B	0.29		
h/L	0.85		
Roof Angle, θ	15.0	º	
Roof Type	Pitched		
Base Wind Velocity, V	32.00	m/s	Chapter 26.5.1
Coefficient Factor of Wind Direction, Kd	0.85		Table 26.6.1
Exposure Category	B		Chapter 26.7
Coefficient Factor of Topography, Kzt	1.00		Table 26.6.1
Coefficient Factor of Wind Blow, G	0.85		Chapter 26.9
Coefficient of Internal Pressure, (GCpi)	0.18		Table 26.11-1
α	7		Table 26.9.1
Zg	365.76	m	Table 26.9.1
Zmin	9.14	m	Table 26.9.1

Coefficient Exposure of Velocity Pressure, Kz	0.92		Table 27.3.1
Velocity pressure, q_z	490.70319	N/m2	Eq. 27.3-1

The coefficient of external pressure Cp* is determine in SNI 1727:2013 figure 27.4-1. The coefficient is divided for wall and roof surface. Plus (+) sign shows the wind comes to the surface and minus (-) sign shows the wind goes off the surface.

Table 2. 5 Wind Load Coefficient Pressure

Coefficient of External Pressure, Cp*	
Wall Surface	Cp*
Wall side of windward	0.80
Wall side of leeward	-0.50
Edge wall	-0.70
Roof Surface	Cp*
Windward side	-0.88
Leeward side	-0.56

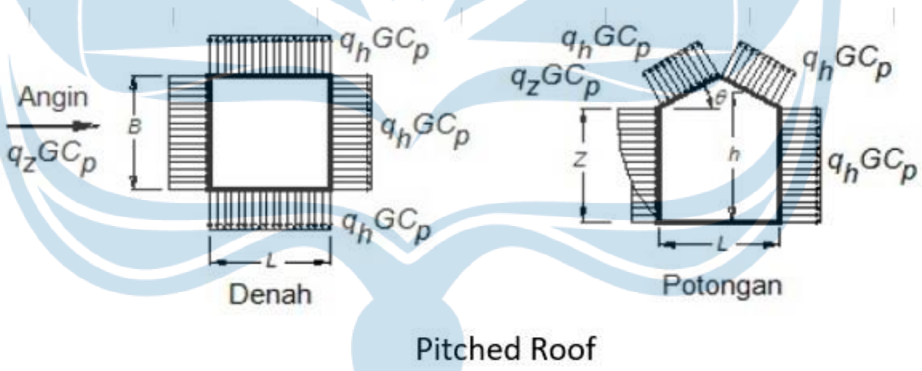


Figure 2. 3 Pitched Roof Wind Load (Source SNI 1727:2013 figure 27.4-1)

The wind pressure is calculated for roof using SNI 1727:2013 eq. 27.4-1 and is shown in Table 2. 6.

Table 2. 6 Wind Load on Roof Surface

Roof Pressure, P			
Roof Wind, Windward	-456	N/mm2	Chapter 27.4.2 & Eq. 27.4.1
Roof Wind, Leeward	-322	N/mm2	Chapter 27.4.2 & Eq. 27.4.1

$P = qGC_p - q_i(GC_{pi})$ (SNI 1727:2013 eq. 27.4-1) ....Eq. 2.1

For $Cp > 0, (-GC_{Pi})$
For $Cp < 0, \left(GC_{pi}\right)$

2.4.6. Earthquake Load

Earthquake load is a dynamic load that moves in lateral motion in the form of vibration. As a dynamic load it can move in two directions, forward and backward. In SNI 1726:2019 table 16, the allowed analysis of earthquake is based on the irregularity of structure. It can be analyzed by three methods, which are static equivalent, response spectrum analysis, and time history analysis. Static equivalent analysis considers earthquake load in static form, meanwhile response spectrum analysis and time history analysis consider earthquake as dynamic load. In this project, the analysis use response spectrum and static equivalent that will be inputted to ETABS modeling as earthquake load.

a. Response Spectrum Analysis

To determine the spectral response of the MCER earthquake acceleration mapped on the soil surface, an amplification factor is required in the period of 0.2 seconds (Fa) and 1 seconds (Fv). The amplification factor is determined based on the site class and ground acceleration parameters. Amplification factor in 0.2 second period (Fa) determined by site class and earthquake acceleration spectral response parameters with value of 1. The MCER is mapped for a 0.2 second (Ss) period. While the amplification factor at 1 second period (Fv) is determined by site class and spectral response earthquake acceleration is mapped for a period of 1 second (S1), the value is 1.7543. Determination of site coefficients (Fa and Fv) are based on SNI 1726:2019 table 6 and 7, respectively and can be calculated by linear interpolation.

Furthermore, the value of the Fa and Fv is used to determine the response spectrum parameters acceleration in short period (SMS) and 1 second period (SM1) which can be calculated using SNI 1726:2019 eq. 7 and 8.

$S_{MS} = Fa \times Ss = 1.28\ g$ (SNI 1726:2019 eq. 7) ....Eq. 2.2
$S_{M1} = Fv \times S1 = 0.9573\ g$ (SNI 1726:2019 eq. 8) ....Eq. 2.3

Next based on the SMS and SM1 values, the design spectral acceleration parameters for short period 0.2 second (SDS) and period 1 second (SD1) need to be set1to construct the response spectrum curve. SDS and SD1 values are calculated1using SNI 1726:2019 eq. 9 and 10.

$SDS = \frac{2}{3} \times SMS = 0.8533\ g$ (SNI 1726:2019 eq. 9) ....Eq. 2.4

$SD1 = \frac{2}{3} \times SM1 = 0.6382\ g$ (SNI 1726:2019 eq. 10) ....Eq. 2.5

Based on the response spectra parameters calculated previously, the design of spectra response curve is based on response spectrum acceleration and (Sa) and period (T). The response spectrum acceleration is calculated using SNI 1726:2019 eq. 11, 12, and 13. The period is calculated using SNI 1726:2019 figure 3. T_L is a long transition period that the value can be taken from SNI 1726:2019 figure 20.

$S_a = S_{DS}\left(0.4 + 0.6\left(\frac{T}{T_0}\right)\right)$ (SNI 1726:2019 eq. 11)

$S_a = \frac{S_{D1}}{T}$ (SNI 1726:2019 eq. 12)... eq. 2.6

$S_a = \frac{S_{D1} \times T_L}{T^2}$ (SNI 1726:2019 eq. 13)... eq. 2.7

$T_0 = 0.2 \times \frac{S_{D1}}{S_{DS}}$ (SNI 1726:2019 figure 3) .... eq. 2.8

$T_s = \frac{S_{D1}}{S_{DS}}$ (SNI 1726:2019 figure 3).. eq. 2.9

$T_L = 6\ s$ (SNI 1726:2019 figure 20) .... eq. 2.10

The response spectrum design calculation can be seen in Table 2. 7 and response spectrum curve can be seen in Figure 2. 4. The calculation from Table 2. 7 will be inputted to ETABS Model as seismic dynamic load.

Table 2. 7 Response Spectrum Design

T (s)	Note	Sa (g)	Note
0	0	0.341333	SDS*(0.4+0.6*T/T0)
0.1496	T0	0.8533	SDS
0.748	Ts	0.8533	SDS
0.848	Ts + 0.1	0.7527	SD1/T
0.948	Ts + 0.2	0.6733	SD1/T
1.048	Ts + 0.3	0.6090	SD1/T
1.148	Ts + 0.4	0.5560	SD1/T
1.248	Ts + 0.5	0.5114	SD1/T
1.348	Ts + 0.6	0.4735	SD1/T
1.448	Ts + 0.7	0.4408	SD1/T
1.548	Ts + 0.8	0.4123	SD1/T
1.648	Ts + 0.9	0.3873	SD1/T
1.748	Ts + 1	0.3651	SD1/T
1.848	Ts + 1.1	0.3454	SD1/T
1.948	Ts + 1.2	0.3276	SD1/T
2.048	Ts + 1.3	0.3116	SD1/T
2.148	Ts + 1.4	0.2971	SD1/T
2.248	Ts + 1.5	0.2839	SD1/T

T (s)	Note	Sa (g)	Note
2.348	Ts + 1.6	0.2718	SD1/T
2.448	Ts + 1.7	0.2607	SD1/T
2.548	Ts + 1.8	0.2505	SD1/T
2.648	Ts + 1.9	0.2410	SD1/T
2.748	Ts + 2	0.2323	SD1/T
2.848	Ts + 2.1	0.2241	SD1/T
2.948	Ts + 2.2	0.2165	SD1/T
3.048	Ts + 2.3	0.2094	SD1/T
3.148	Ts + 2.4	0.2027	SD1/T
3.248	Ts + 2.5	0.1965	SD1/T
3.348	Ts + 2.6	0.1906	SD1/T
3.448	Ts + 2.7	0.1851	SD1/T
3.548	Ts + 2.8	0.1799	SD1/T
3.648	Ts + 2.9	0.1750	SD1/T
3.748	Ts + 3	0.1703	SD1/T
3.848	Ts + 3.1	0.1659	SD1/T
3.948	Ts + 3.2	0.1617	SD1/T
4.048	Ts + 3.3	0.1577	SD1/T
4.148	Ts + 3.4	0.1539	SD1/T
4.248	Ts + 3.5	0.1502	SD1/T
4.348	Ts + 3.6	0.1468	SD1/T
4.448	Ts + 3.7	0.1435	SD1/T
4.548	Ts + 3.8	0.1403	SD1/T
4.648	Ts + 3.9	0.1373	SD1/T
4.748	Ts + 4	0.1344	SD1/T

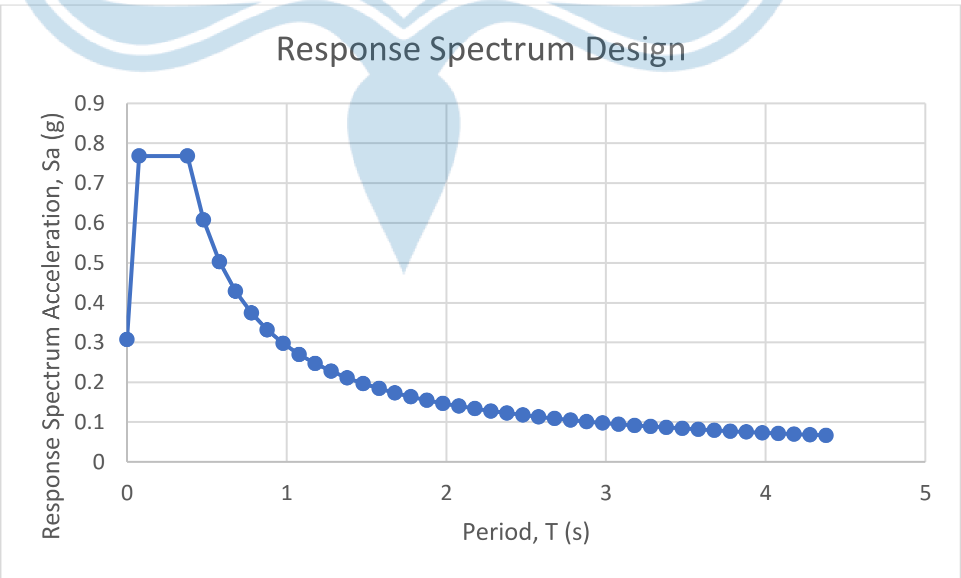


Figure 2. 4 Response Spectrum Curve

b. Static Equivalent Analysis

Static equivalent load is divided into two directions, which are X direction and Y Direction. This load considers earthquake as static load that acts differently on the height of building. The taller the height, the bigger the force and displacement as in Figure 2. 5. The parameter of earthquake static load in X direction and Y direction can be seen in Table 2. 8 and Table 2. 9, respectively. This parameter will be inputted to ETABS Model as static equivalent load.

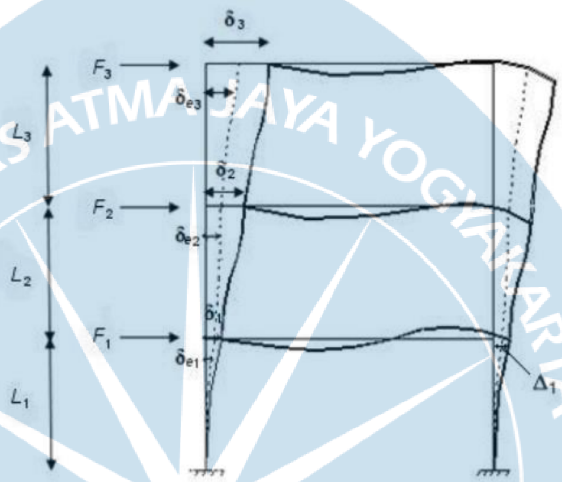


Figure 2. 5 Static Equivalent Load (Source: SNI 1726:2019 figure 10)

Table 2. 8 Static Equivalent Parameter in X Direction

Description	**Value**
Direction and Eccentricity	X Dir, X Dir ± Eccentricty
0.2 Sec Spectral Accel, S_S	1.28
1 Sec Spectral Accel, $S1$	0.5457
Long-Period Transition Period, T_L	6
Site Class	D
Fa	1
Fv	1.7543
SDS	0.8533
SD1	0.6382
Response Modification, R	8
System Overstrength, Ω	3
Deflection Amplification, Cd	5.5
Occupancy Importance, I	1

Table 2. 9 Static Equivalent Load Parameter in Y Direction

Description	**Value**
Direction and Eccentricity	X Dir, X Dir ± Eccentricty
0.2 Sec Spectral Accel, S_S	1.28
1 Sec Spectral Accel, $S1$	0.5457

Description	Value
Long-Period Transition Period, T_L	6
Site Class	D
Fa	1
Fv	1.7543
SDS	0.8533
SD1	0.6382
Response Modification, R	8
System Overstrength, Ω	3
Deflection Amplification, Cd	5.5
Occupancy Importance, I	1

2.5. Structural Modelling

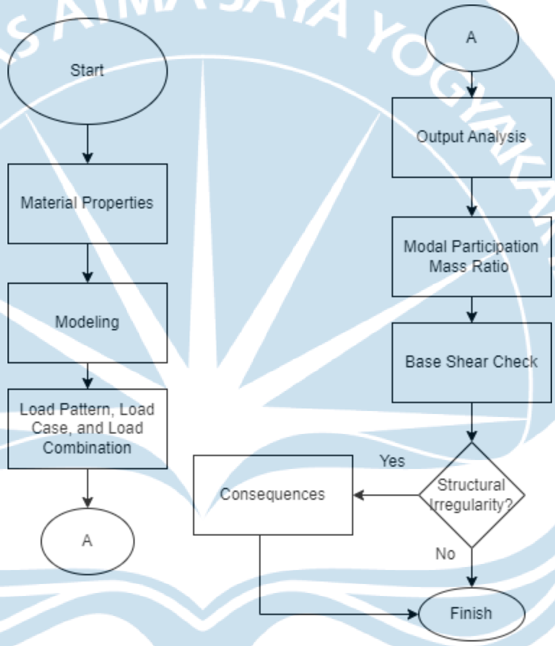


Figure 2. 6 Structural Modelling Flowchart

The procedure of structural modelling is shown in Figure 2. 6. The modeling uses ETABS Software to analyze the internal force and displacement of the building due to several loads that is put in combination. Structural modeling is helpful to determine the internal forces that occur in structural elements and the behavior of the structure due to the working load. The results of the structural modeling are used as the basis for designing the dimensions section of structural elements required.

The structure parts that are modeled in the software are beam, tie beam, column, and slab. The properties of this structure are presented in Table 2. 10 Material Properties

Table 2. 10 Material Properties

Materials	
Fc’	30 Mpa
Fy	400 Mpa
Fu	550 MPa
Beam	
Width, B	500 mm
Height, H	500 mm
Concrete Cover	40 mm
Tie Beam	
Width, B	350 mm
Height, H	350 mm
Concrete Cover	40 mm
Column	
Width, B	650 mm
Height, H	650 mm
Concrete Cover	40 mm
Longitudinal Bar	16D25
Confinement	D13

Based on SNI 2847:2019 table 6.6.3.1.1(a) floor slabs are recommended as a shell, while beams and columns are recommended as frame elements. The structural modeling can accommodate the effects of steel damage when an earthquake occurs, namely by reducing the moment of inertia of the cross section of the structural elements. The moment of inertia in the slab is reduced to 25% of the moment initial inertia. In beam structural elements, the moment of inertia is reduced to 35% of the initial moment of inertia. In addition, the torque is also reduced to 25% to balance the reduction value against the inertia of the structural elements. While in the column, the moment of inertia is reduced to 70% of the initial moment of inertia. The comparation of SNI 2847:2019 table 6.6.3.1.1(a) and ETABS is presented in Table 2. 11. The left view, right view, front view, and plan view of building model can be seen in Figure 2. 7, Figure 2. 8, Figure 2. 9, and Figure 2. 10 respectively.

Table 2. 11 Comparation of SNI and ETABS

SNI 2847:2019 table 6.6.3.1.1(a)			**ETABS**	
Beams	Shear Effective Stiffness	0.4EcAw	V22=V33	0.4
	Torsional Effective Stiffness	0.25	Torsional Constant	0.25

SNI 2847:2019 table 6.6.3.1.1(a)			**ETABS**	
	Flexure Effective Stiffness	0.35Ig	I22=I33	0.35
Columns		0.7Ig	I22-I33	0.7
Walls	Uncracked	0.7Ig	shell - f11, f22	0.7
	Cracked	0.35Ig	shell - f11, f22	0.35
Slab	Flat Plates and Flat Slabs	0.25Ig	membrane	0.25
			shell - f11, f22, f12, m11, m22, m12	0.25

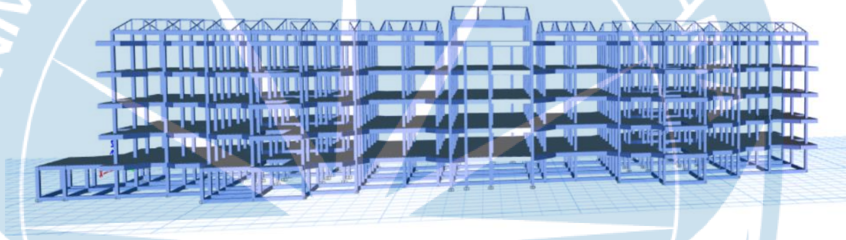


Figure 2. 7 Building Model Left View

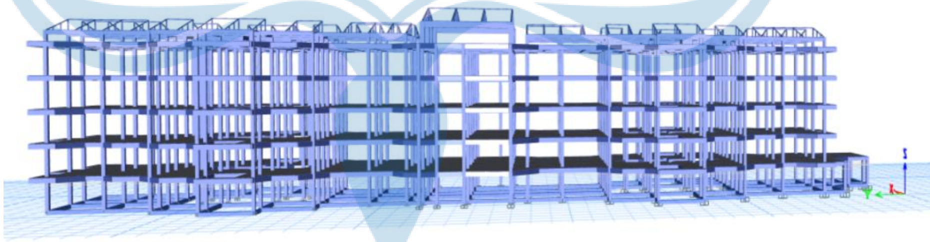


Figure 2. 8 Building Model Right View

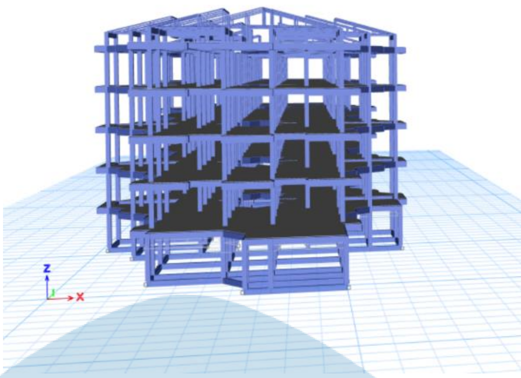


Figure 2. 9 Building Model Front View

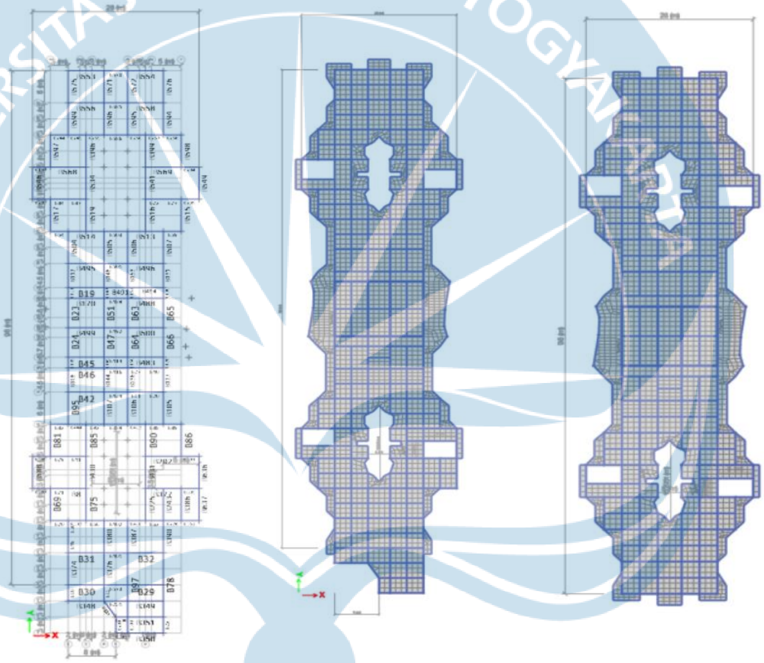


Figure 2. 10 Building Model Plan View

According to SNI 1726:2019 articles 7.7.2 the mass source must include the dead load, additional load, and at least 25% live load of the structures. This can be seen in Table 2. 12.

Table 2. 12 Mass Source

Load Pattern	**Multiplier**
Dead	1
ADL	1
Live	0.25

2.6. Evaluating Structural Systems Output and Structural Irregularities

2.6.1. Modal Mass Ratio Check

Based on the results of structural modeling, the capital participation mass ratio is presented in Table 2. 13. The number of modes (modes) required to determine the natural vibrational variation for the structure must be sufficient to reach a combined mass participation up to 100% of the actual mass of each x and y direction of the response considered by the model, in accordance with Article 7.9.1 of SNI 1726:2019. Based on the results of structural modeling, it is found that in both directions involving 60 modes of vibration the total mass participation mass ratio in both X and Y directions is very close to 100%.

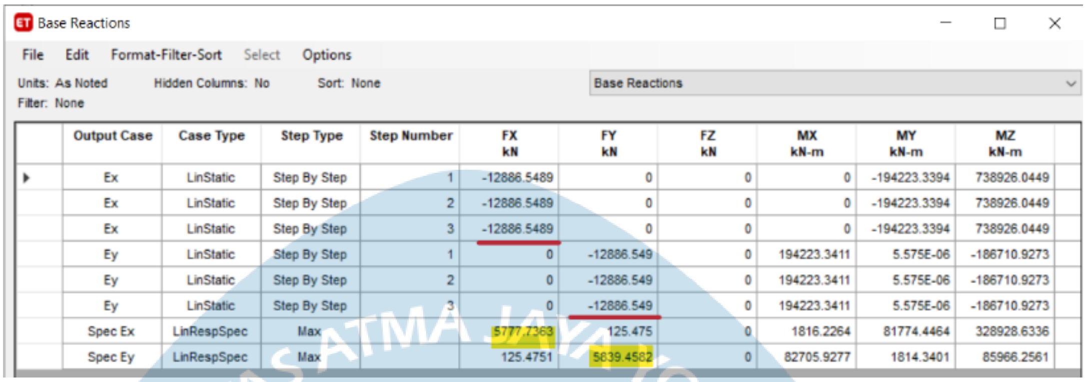
Table 2. 13 Modal Participation Mass Ratio

Mode	Period (sec)	UX	UY	SumUX	SumUY
1	1.461	0.2233	0.0034	0.2233	0.0034
2	1.435	0.011	0.7793	0.2343	0.7827
3	1.426	0.5733	0.025	0.8076	0.8076
4	0.516	0	0.0015	0.8076	0.8091
5	0.454	0.0206	0.0001	0.8282	0.8092
6	0.445	0.0613	0.0341	0.8896	0.8433
7	0.444	0.0294	0.0767	0.919	0.92
8	0.421	0.0004	0	0.9194	0.92
9	0.321	0.000002823	0.00001081	0.9194	0.92
10	0.246	0.000002931	0.00000363	0.9194	0.92
11	0.242	0.047	0.001	0.9664	0.921
12	0.241	0.0009	0.0461	0.9673	0.9671
13	0.221	0.0003	0.0000277	0.9676	0.9672
14	0.211	0	0.00001156	0.9676	0.9672
15	0.21	7.423E-07	0.0001	0.9676	0.9673
16	0.209	0	0.000004497	0.9676	0.9673
17	0.208	0	0.0013	0.9676	0.9686
18	0.202	0	0	0.9676	0.9686
19	0.202	0.000003573	0.000001848	0.9676	0.9686
20	0.186	0.0001	5.868E-07	0.9677	0.9686
21	0.182	0	0.000002376	0.9677	0.9686
22	0.167	0.005	0.00001138	0.9727	0.9686
23	0.162	0	0.00002054	0.9727	0.9686
24	0.16	0.0102	0.000002278	0.9829	0.9687
25	0.157	0.00002709	0.0218	0.9829	0.9905
26	0.156	0.004	0.0003	0.987	0.9908
27	0.154	0.003	0.00003246	0.9899	0.9908
28	0.154	0.0007	0.0007	0.9907	0.9915

29	0.139	0.000002203	0	0.9907	0.9915
30	0.138	0.0006	0	0.9913	0.9915
31	0.137	0.000002025	0	0.9913	0.9915
32	0.137	5.383E-07	9.492E-07	0.9913	0.9915
33	0.137	0.00001128	0	0.9913	0.9915
34	0.137	0	0	0.9913	0.9915
35	0.137	0.000007956	0	0.9913	0.9915
36	0.137	0.0001	0	0.9914	0.9915
37	0.136	0	0.0001	0.9914	0.9916
38	0.136	0	0	0.9914	0.9916
39	0.133	0	0.0001	0.9914	0.9917
40	0.133	0	0	0.9914	0.9917
41	0.132	0.000002845	0	0.9914	0.9917
42	0.13	0.00002224	0.000002183	0.9914	0.9918
43	0.129	0.0004	0.00000125	0.9919	0.9918
44	0.127	0.000004437	0	0.9919	0.9918
45	0.127	0	0.0002	0.9919	0.9919
46	0.127	0.000002086	8.481E-07	0.9919	0.9919
47	0.127	0	0.00004718	0.9919	0.992
48	0.125	0.00003059	0	0.9919	0.992
49	0.125	0.00004628	0	0.992	0.992
50	0.124	0.0006	0	0.9926	0.992
51	0.123	0.0016	0.00001888	0.9941	0.992
52	0.123	0.000006219	0	0.9942	0.992
53	0.122	0.0002	0.00001434	0.9944	0.992
54	0.121	0	0.0002	0.9944	0.9922
55	0.12	0.0001	0.0021	0.9945	0.9943
56	0.12	0.0001	0.0039	0.9946	0.9982
57	0.118	0.0037	0.0000108	0.9982	0.9982
58	0.115	0	0.0002	0.9982	0.9984
59	0.114	0.00003188	0	0.9983	0.9984
60	0.113	0	0.000001843	0.9983	0.9984

2.6.2. Load Combination Base Shear Check

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


Base Reactions

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None Base Reactions

Filter: None

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Ex	LinStatic	Step By Step	1	-12886.5489	0	0	0	-194223.3394	738926.0449
Ex	LinStatic	Step By Step	2	-12886.5489	0	0	0	-194223.3394	738926.0449
Ex	LinStatic	Step By Step	3	-12886.5489	0	0	0	-194223.3394	738926.0449
Ey	LinStatic	Step By Step	1	0	-12886.549	0	194223.3411	5.575E-06	-186710.9273
Ey	LinStatic	Step By Step	2	0	-12886.549	0	194223.3411	5.575E-06	-186710.9273
Ey	LinStatic	Step By Step	3	0	-12886.549	0	194223.3411	5.575E-06	-186710.9273
Spec Ex	LinRespSpec	Max		5777.7363	125.475	0	1816.2264	81774.4464	328928.6336
Spec Ey	LinRespSpec	Max		125.4751	5839.4582	0	82705.9277	1814.3401	85966.2561

Figure 2. 11 Old Base Reaction

From the result of base shear, the response spectrum variance is still less than 85%, so scaling is required. To scale the load is by dividing the equivalent lateral force in x direction by the response spectrum load in X direction and the same with the force in Y direction.

Table 2. 14 Base Reaction Scaling

Ex	12886.55	kN	Ey	12886.55	kN
Spec Ex	5777.736	kN	Spec Ey	5839.458	kN
Scale =Ex/Spec Ex	2.23038		Scale = Ey/Spec Ey	2.206806	

Where:

Ex = earthquake load due to static equivalent load in X direction
Ey = earthquake load due to static equivalent load in Y direction
Spec Ex = earthquake load due to response spectrum in X direction
Spec Ey = earthquake load due to response spectrum in Y direction

The scale can be inputted in the load cases for response spectrum force by multiplying the old scale with the new scale. The old scale fact of Spec Ex and Spec Ey is presented in Figure 2. 12, the new scale fact of Spec Ex and Spec Ey is presented in Figure 2. 13, the new base shear can be seen in Figure 2. 14.

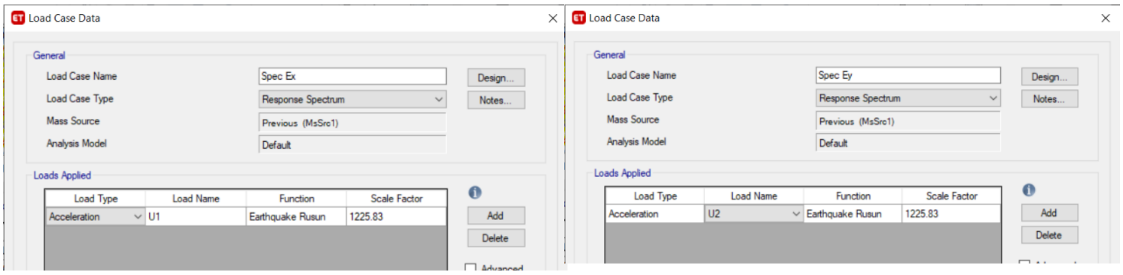


Figure 2. 12 Old Scale Factor

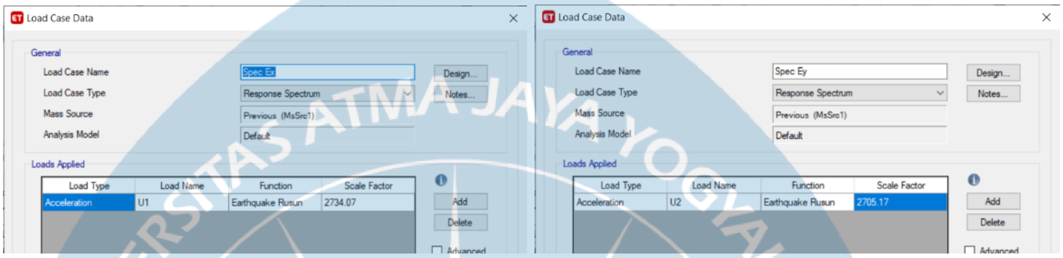
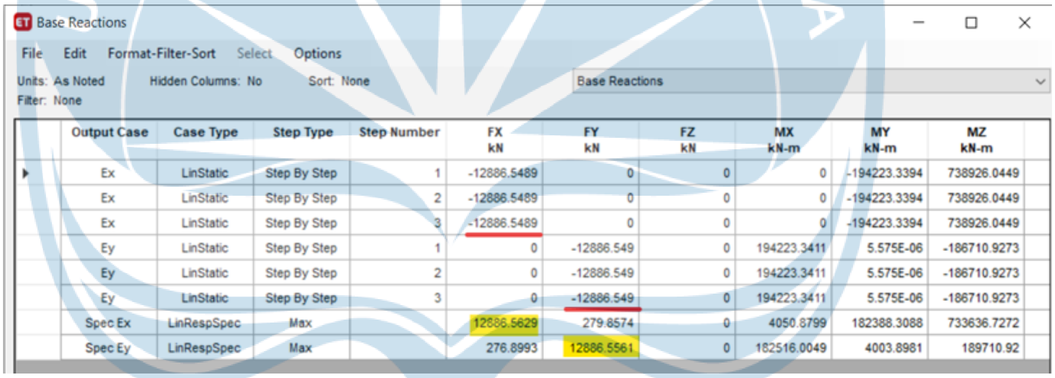


Figure 2. 13 New Scale Factor


Base Reactions

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None Base Reactions

Filter: None

Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Ex	LinStatic	Step By Step	1	-12886.5489	0	0	0	-194223.3394	738926.0449
Ex	LinStatic	Step By Step	2	-12886.5489	0	0	0	-194223.3394	738926.0449
Ex	LinStatic	Step By Step	3	-12886.5489	0	0	0	-194223.3394	738926.0449
Ey	LinStatic	Step By Step	1	0	-12886.549	0	194223.3411	5.575E-06	-186710.9273
Ey	LinStatic	Step By Step	2	0	-12886.549	0	194223.3411	5.575E-06	-186710.9273
Ey	LinStatic	Step By Step	3	0	-12886.549	0	194223.3411	5.575E-06	-186710.9273
Spec Ex	LinRespSpec	Max		12886.5629	279.8574	0	4050.8799	182388.3088	733636.7272
Spec Ey	LinRespSpec	Max		276.8993	12886.5561	0	182516.0049	4003.8981	189710.92

Figure 2. 14 New Base Reaction

2.6.3. Structural Irregularity

In the design process, the structure should be classified as a regular structure or irregular by referring to Article 7.3.2 of SNI 1726:2019. Structural irregularities are divided into horizontal irregularities and vertical irregularities. Further types and explanations of horizontal and vertical irregularities respectively are presented in more detail in SNI 1726:2019 table 13 and table 14.

The following are the results of calculations and checking of horizontal irregularities of the structure:

1.a. Torsional Irregularity is defined to exist if the maximum story drift (calculated torque including unexpected torque) at an end of the structure transverse to the axis is more than

1.2 times the average story drift at both ends of the structure. The torsional irregularity requirements in the reference articles apply only to structures whose diaphragms are rigid (rigid) or semi-rigid (semi-rigid). The data of maximum drifts per average drifts in the considered story level is presented in **Error! Reference source not found.**.

Table 2. 15 Torsional Irregularity Check

Level	**X Direction**		**Y Direction**	
	Δmax/Δavg	**Check**	**Δmax/Δavg**	**Check**
5	1.006	OK	1.002	OK
4	1.011	OK	1.002	OK
3	1.021	OK	1.005	OK
2	1.035	OK	1.018	OK
1	1.267	H.1a	1.035	OK
	Ex		Ey	

Based on checking the torsional irregularity, it is found that the maximum ratio story drift in the X direction for story level 1 is bigger than 1.2. Therefore, this structure has torsional irregularity 1.a.

1.b. Excessive Torsional Irregularity is defined to exist if the maximum story drift (calculated torque including unexpected torque) at an end of the structure transverse to the axis is more than 1.4 times the average story drift at both ends of the structure (see Figure 2. 15). The excessive torsional irregularity requirements in the reference articles apply only to structures whose diaphragms are rigid (rigid) or semi-rigid (semi-rigid). Based on checking for excessive torsional irregularities, it was found that the maximum floor deviation in the X and Y directions is less than 1.4 times the average floor deviation so that there is no type 1b horizontal irregularity in the structure under review.

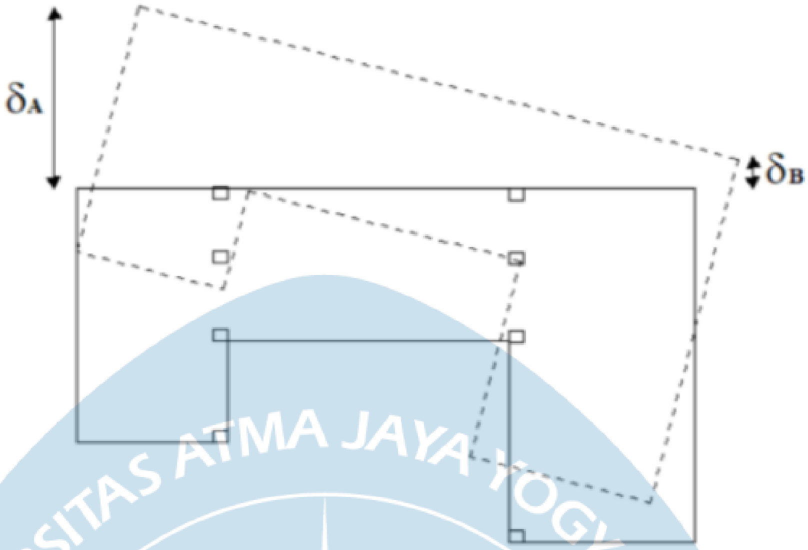


Figure 2. 15 Horizontal Irregularity 1. a and 1. B

2. Inner Angle Irregularity, defined to exist if both projections structural plan from the inside angle greater than 15% of the floor plan structure in the specified direction (see Figure 2. 16)

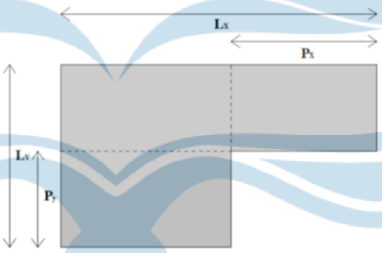


Figure 2. 16 Horizontal Irregularity 2

Based on checking the irregularity of the interior angle, the results obtain that the structural plan projections in x direction from the interior angle is larger than 15% dimensions of the structure plan in the specified direction but in Y direction is less than 15%. In SNI, the type 2 irregularity is defined if both cases meet the condition. Horizontal irregularity type 2 check is presented in Table 2. 16 and the condition of model is presented in Figure 2. 17. So, there is no type 2 horizontal irregularity in the structure under consideration.

Table 2. 16 Horizontal Irregularity Type 2 Check

Description	**Value**	**Unit**
Lx	28	m
Px	8	m
Ly	96	m
Py	6	m
Px/Lx	0.2857	
Py/Ly	0.0625	
Check	OK	

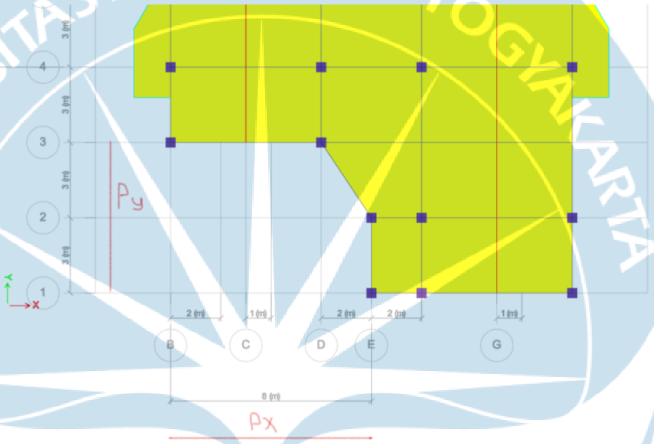


Figure 2. 17 Structure Horizontal Irregularity 2 Condition

3. Diaphragmatic discontinuity irregularities, defined as present if present diaphragm with sudden discontinuity or variation in stiffness, including those with truncated or exposed areas greater than 50% of the gross diaphragm area surrounding it, or changes in effective diaphragm stiffness of more than 50% from lower story to upper story (see Figure 2. 18).

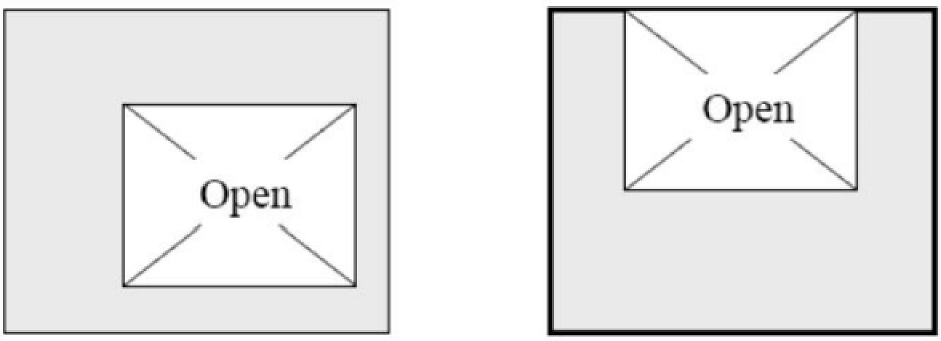


Figure 2. 18 Horizontal Irregularity 3

Based on checking the irregularity of the interior angle, the results obtained are that there is no type 3 horizontal irregularity in the structure reviewed. As can be seen in Figure 2. 19 there are 4 opening of 6 m x 3.5 m size and 2 openings of 8 m x 15 m. The whole area of building is 96 m x 28 m. The check of interior opening is presented in Table 2. 17. If the ratio is less than 50% of the building area, the condition is okay.

Table 2. 17 Structure Opening Check

A_{total}	2640	m^2
$A_{opening}$	324	m^2
Check	OK	
Ratio	0.12	<0.5

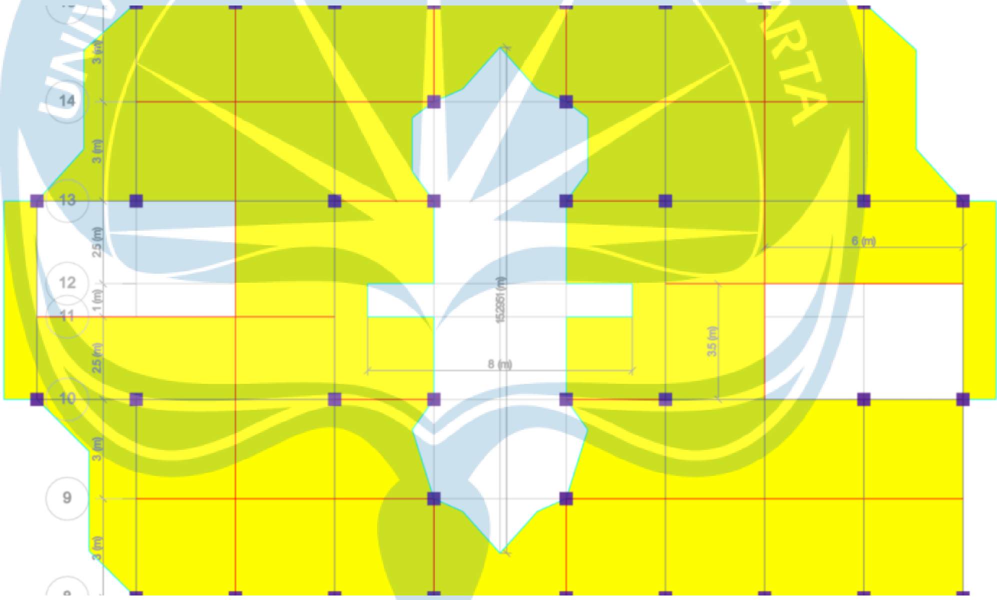


Figure 2. 19 Structure Opening

4. The Transverse Displacement Irregularity with respect to the plane, defined to exist if there is a discontinuity in the path of the lateral resistance, such as transverse to the plane of the vertical element (see Figure 2. 20).

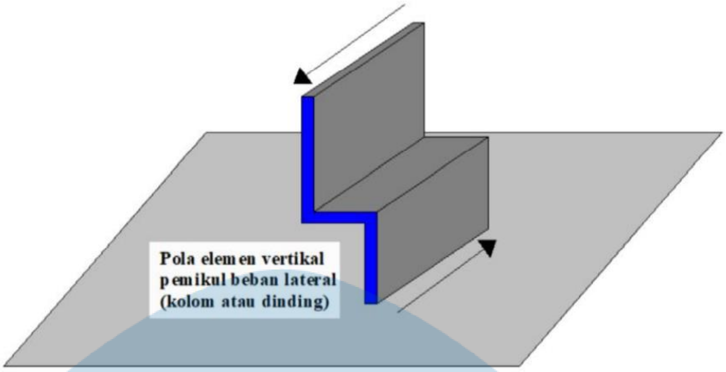


Figure 2. 20 Horizontal Irregularity 4

Based on checking the irregularity of the interior angle, the result is there is no type 4 horizontal irregularity in the structure reviewed.

5. Nonparallel system irregularities are defined to exist if the elements vertical lateral restraints are not parallel or symmetrical about the axes main orthogonal seismic resisting system.

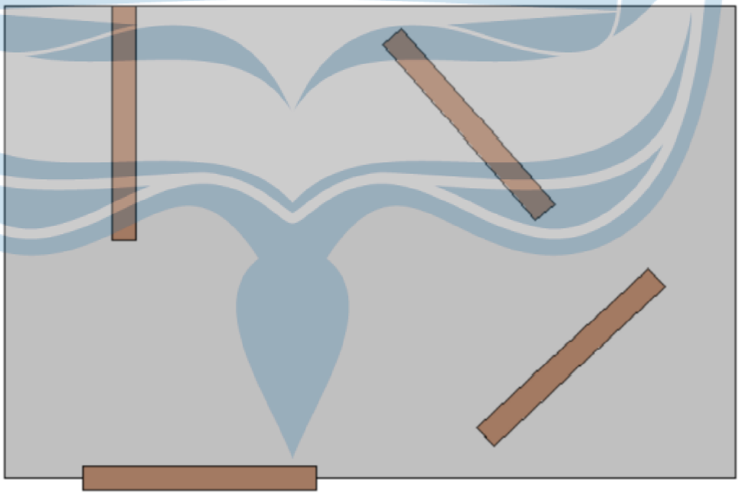


Figure 2. 21 Horizontal Irregularity 5

Based on checking the irregularity of the interior angle, the results obtained are that there is no type 5 horizontal irregularity in the structure reviewed.

Following are the results of calculations and checking of vertical irregularity of the structure:

1.a. Soft-grade stiffness irregularity, defined to exist if any a grade where the lateral stiffness is less than 70% of the lateral stiffness story above or less than 80% of the average stiffness of the 3 stories above (see Figure 2. 22). Based on irregularity checking stiffness of the soft level using ETABS, the result is that there is no vertical irregularity of type 1a in the structure under consideration.

Kx<0.7K(x+1) .... eq. 2.11
Kx<0.8[K(x+1) + K(x+) + K(x+3)]/3 .... eq. 2.12

Table 2. 18 Soft-grade Stiffness Irregularity Check

Story	**Direction X**		**Direction Y**	
	Stiffness (K) kN/m	**Check**	**Stiffness (K)** kN/m	**Check**
5	304434.1	OK	331177.8	OK
4	320201.4	OK	336970.8	OK
3	323407.6	OK	338394.8	OK
2	339289.8	OK	352885.7	OK
1	615684.6	OK	562611.2	OK

1.b. Excessive soft level stiffness irregularity, defined to exist if1there is a level where the lateral stiffness in Table 2. 18 is less than 60% of the stiffness lateral story level above or less than 70% of the average stiffness 3 levels above it (see Figure 2. 22). Based on checking excessive soft level stiffness irregularity, the results obtained that there is no type 1b vertical irregularity in the structure reviewed.

Kx<0.6K(x+1) .... eq. 2.13
Kx<0.7[K(x+1) + K(x+) + K(x+3)]/3 .... eq. 2.14

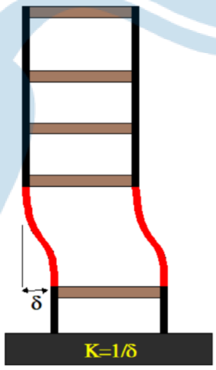


Figure 2. 22 Vertical Irregularity 1a and 1b

2. Heavy (mass) irregularity, defined to exist if all levels are effective more than 150% effective rate nearby. A roof that is lighter than the floor below does not need to be reviewed (see Figure 2. 23). Based on checking the weight (mass) irregularity in Table 2.

19 Mass Irregularity Check, it was found that there is no type 2 vertical irregularity in the structure under consideration.

Mx>1.5M(x+1).. eq. 2.15
Mx>1.5M(x-1)... eq. 2.16

Table 2. 19 Mass Irregularity Check

Story	Mass kg	Check
5	2271285.39	OK
4	2349745.83	OK
3	2348571.12	OK
2	2348571.12	OK
1	2455281.89	OK

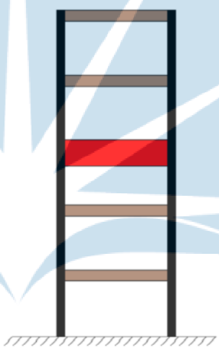


Figure 2. 23 Vertical Irregularity 2

3. A vertical geometric irregularity, defined to exist if the dimension1horizontal seismic resisting system at all levels more than 130% the horizontal dimensions of the nearby level seismic restraint system (see Figure 2. 24). Based on geometric irregularity checking vertically in Table 2. 20, the result is that there is no vertical irregularity type 3 on the structure under review.

Lx>1.3L(x+1)... eq. 2.16
Lx>1.3L(x-1)... eq. 2.17

Table 2. 20 Vertical Geometric Irregularity Check

Story	L mm	Check
5	400	OK
4	400	OK

Story	L	Check
	mm	
3	400	OK
2	400	OK
1	400	OK

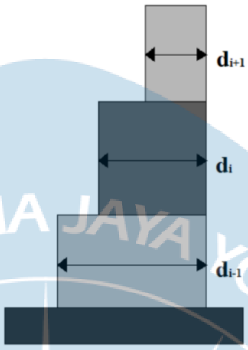


Figure 2. 24 Vertical Irregularity 3

4. Discontinuity of the plane direction in the irregularity of the force-resisting element vertical lateral, defined to exist if the shift in the direction of the element plane1lateral restraint is greater than the length of the element or there is a reduction1stiffness of the retaining elements in the story below (see Figure 2. 25). Based on checking the irregularity of the discontinuity of the direction of the plane1in the irregularity of the vertical lateral force resisting element, we get1the result that there is no type 4 vertical irregularity in the structure reviewed.

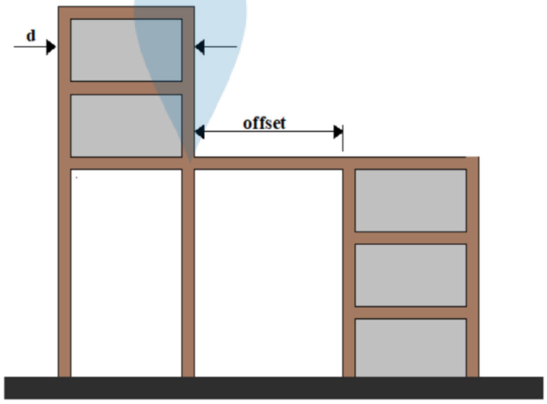


Figure 2. 25 Vertical Irregularity 4

5.a. The discontinuity in the level lateral strength irregularity, defined to exist1if the lateral strength of the story is less than 80% of the lateral strength of the story above it. Strong lateral level is the total lateral strength of all seismic resisting elements1share the level shear for the direction under consideration (see Figure 2. 26). Based on checking the irregularity of the discontinuities in the level lateral strength irregularities, the results show in Table 2. 21 that there is no vertical irregularity of type 5a in the structure under consideration.

Table 2. 21 Discontinuity Check

Story	Direction X		Direction Y	
	Strength kN	Check	Strength kN	Check
5	2734.059	OK	2734.059	OK
4	4768.63	OK	4768.63	OK
3	6283.111	OK	6283.111	OK
2	7282.812	OK	7282.812	OK
1	7796.597	OK	7796.597	OK

5.b. Discontinuity in excessively graded lateral strength irregularity, defined to exist if the lateral strength level is less than 65% lateral strength level above it. The lateral strength of the story is the total strength of all elements1seismic restraints sharing the story shear for the direction under consideration (see Figure 2. 26). Based on discontinuity irregularity checking in excessive degree of lateral strength irregularity, obtained1the result that there is no type 5b vertical irregularity in the structure reviewed.

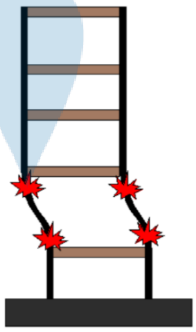


Figure 2. 26 Vertical Irregularity 5a and 5b

2.7. Consequence of Irregularity Horizontal Irregularity

2.7.1. Consequence of Irregularity Horizontal Irregularity

This structure has horizontal irregularity 1.a of torsional irregularity. As the consequence, in the article 11.3.4 for torsional irregularity 1.a or 1.b, this structure must consider the accidental eccentricity consisting of the assumed displacement of the centre of mass each from the actual location by a distance equal to 5% of the diaphragm dimension from structure parallel to the direction of mass shift. The scale for eccentricity can inputted in the ETABS to the load cases of earthquake spectrum X and earthquake lateral force X as 0.05. Even though, the structure does not have horizontal irregularity but in the seismic class C, D, E, and F it is better to consider the 5% eccentricity for both X and Y direction.

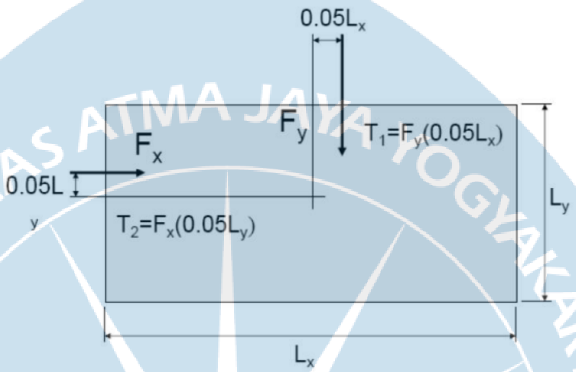


Figure 2. 27 Consequence of Irregularity Horizontal Irregularity

a. Redundant Factor

The redundant factor (ρ) for this structure is taken to be 1.0 because this structure as a seismic design category D structure does not have 1.b irregularity or moment resisting frame that loss of moment resistance at beam-to-column connection at moment at both ends of a single beam resulting in more of the strong reduction rate of 33%. Further information can be seen in table SNI 1726:2019 table 15.

Therefore, based on SNI 1726:2019 table 20 the permission displacement for all structure in risk category II for this structure is 0.02x20 m = 0.4 m.

b. Unexpected torque moment magnification factor

Structures designed for seismic design category C, D, E, or F, where torsional irregularities of type 1a or 1b occur as defined must have an effect calculated by multiplying Mta at each level with torque magnification factor (Ax and Ay) in 1726:019 chapter 7.8.4.31. The calculation of torque magnification factor in X direction and in Y direction is presented in Table 2. 22 and Table 2. 23.

$$A_x = \left(\frac{\delta max}{1.2\delta_{avg}}\right)^2 \quad \text{eq. 2.18}$$

δmax = maximum displacement in the x (mm) level calculated assuming Ax = 1mm

δavg = average displacement at the furthest points of the structure at level x calculated assuming Ax = 1 mm

Table 2. 22 Torque Magnification Factor in X direction

Story Level	UxA (mm)	UxB (mm)	Umax (mm)	Uavg (mm)	Ax = $\frac{\mathrm{Umax}^2}{\mathrm{U_{avg}}}$
5	0.07711	0.081955	0.081955	0.079534	0.74
4	0.06809	0.072928	0.072928	0.070507	0.74
3	0.05305	0.057903	0.057903	0.055475	0.76
2	0.03324	0.038235	0.038235	0.035738	0.79
1	0.01128	0.01604	0.01604	0.01366	0.96

Table 2. 23 Torque Magnification Factor in Y Direction

Story Level	UyA (mm)	UyB (mm)	Umax (mm)	Uavg (mm)	Ay =$\frac{\mathrm{Umax}^2}{\mathrm{U_{avg}}}$
5	75.672	75.639	0.081955	0.079534	0.74
4	67.404	67.37	0.072928	0.070507	0.74
3	53.237	53.198	0.057903	0.055475	0.76
2	34.598	34.708	0.038235	0.035738	0.79
1	13.791	14.307	0.01604	0.01366	0.96

Because the horizontal irregularity 1.a happens, so the torque magnification factor is multiplied with the unexpected torque moment (M_{ta}) of first floor. In ETABS Model this can be calculated by multiplying eccentricity of 0.05 with the torque magnification factor.

As in SNI 1726:2019 table 16, the last consequence of horizontal irregularity 1.a is to determine the right analysis procedure of the structure characteristic based on the seismic design. From the table 16 can be seen that this structure cannot be analysed by lateral equivalent force. The analysis that is chosen for this structure is mode spectrum response analysis.

From SNI 1726:2019 article 17.9.1.11, the analysis should be carried out to determine the natural vibrational variance for the structure. For mode spectrum response analysis. analysis must include enough variance to obtain mass participation of variance combined by 100% of the mass of the structure. In order to achieve this provision, for a variety of single rigid body with a period of 0.05 seconds, it is permitted to take all variances with periods under 0.05 seconds. From ETABS the modal mass ratio can be checked. It is seen that up to mode number 60 the total mass participation ratio is very close to 1 or 100%.

2.8. Consequence of Vertical Irregularity

Because this structure does not have any vertical irregularity, so there is no consequence of vertical irregularity.

2.9. Roof Structure Design

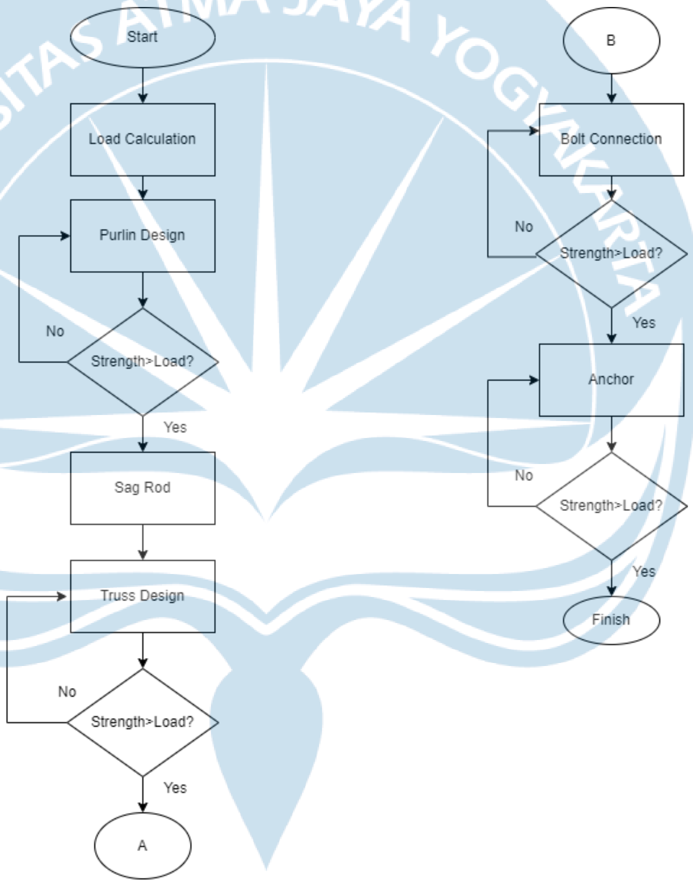


Figure 2. 28 Roof Structure Design Flowchart

Roof design consists of purlin design, truss design, connection, and anchorage. The main procedure of roof structure design is first doing preliminary design, then calculate the loading that acts on the structure, next calculate the internal forces using ETABS Software, and last to check the strength and displacement of structure. The procedure of roof structure design is shown in Figure 2. 28.

2.9.1. Purlin

The purlin uses steel C125x50x20x2. The dimension of steel can be seen in Table 2. 24 and the properties can be seen in Table 2. 25. Tributary area where the load acts on purlin is calculated in Table 2. 26 and shown in Figure 2. 29. Figure 2. 29 shows the plan view of roof where purlin holds the loading than transferred by the roof cover.

Table 2. 24 Purlin Dimension

Description	**Value**	**Unit**
Purlin Type	C125x50x20x2	
H	125	mm
B	50	mm
C	20	mm
t	2	mm
L	3.7	m

Table 2. 25 Purlin Properties

Description	**Value**	**Unit**	**Note**
Purlin weight	3.95	kg/m	
Ix	1200000.00	mm4	Inertia of x
Iy	180000.00	mm4	Inertia of y
Sx	19300.00	mm3	elastic section modulus of x
Sy	5500.00	mm3	elastic section modulus of y
A	504.00	mm2	Gross Area
Fy	240.00	mpa	Yield Strength
Fu	370.00	mpa	Rupture Strength
rx	48.90	mm	radius of gyration of x
ry	19.10	mm	radius of gyration of y
Cx	16.90	mm	Centroid of x
xo	41.50	mm	Shear Center of x
xy	0.00	mm	Shear Center of y
J	6720000.00	mm4	Torsional Constant
Cw	675000000.00	mm6	Warping Constant
E	200000.00	mpa	Modulus of Elasticity
Lb	1850	mm	Unbraced Length due to Sag Rod
Zx	22874.63813	mm3	Plastic section modulus of y
Zy	7564.791813	mm3	Plastic section modulus of x

Table 2. 26 Purlin Tributary Area

Tributary Area			
Description	**Value**	**Unit**	**Note**
Tributary Length (Lt)	2.07	m	
Top Purlin	7.659	m2	$Lt \times L$
Middle Purlin	15.318	m2	$2 \times Lt \times L$
Bottom Purlin	7.659	m2	$Lt \times L$
α	15	°	Truss Angle

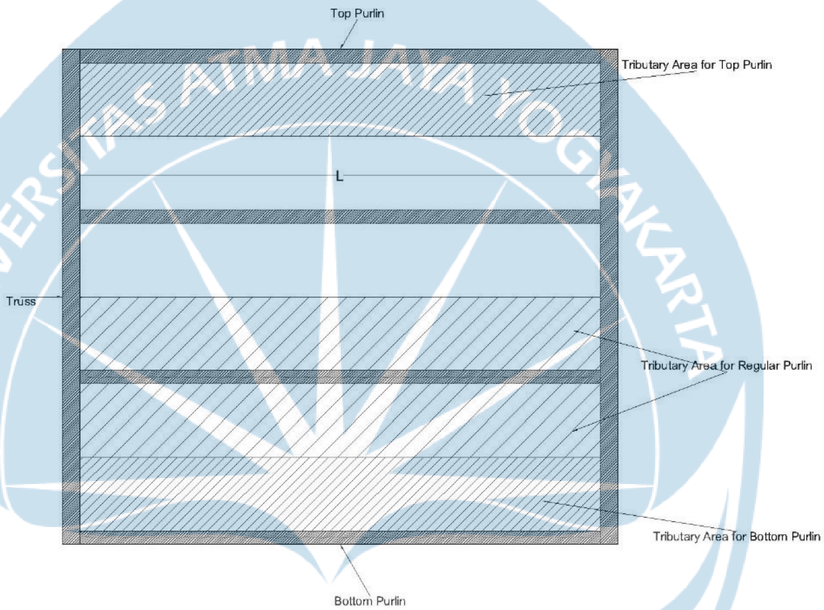


Figure 2. 29 Purlin Tributary Area

a. Loading

The loading of purlin for top and bottom can be seen in Table 2. 27 and the loading for middle purlin can be seen in Table 2. 28.

Table 2. 27 Top and Bottom Purlin Load

Top Purlin and Bottom Purlin			
Description	**Value**	**Unit**	**Note**
Roof Weight	2.5	kg/m2	Metal
Ceiling Weight	18	kg/m2	
Roof (ADL)	0.1915	kN	Roof Weight×Tributary
Ceiling (ADL)	1.3786	kN	Ceiling Weight×Tributary
Purlin (DL)	0.1462	kN	Purlin Weight×Length
Total Dead Load	1.7162	kN	Roof+Ceiling+Purlin
Live Load (LL)	1	kN	SNI

Top Purlin and Bottom Purlin			
Description	**Value**	**Unit**	**Note**
Windward Wind Load (Wt)	-0.456	kN/m2	(+) comes to roof (-) goes off roof
Leeward Wind Load (Wh)	-0.322	kN/m2	(+) comes to roof (-) goes off roof
Windward Wind Load (Wt)	-3.4926	kN	(Wt)×Tributary
Leeward Wind Load (Wh)	-2.4689	kN	(Wh)×Tributary
Take Wind Load (WL)	-3.4926	kN	Max from Wt and Wh

Table 2. 28 Middle Purlin Load

Middle Purlin			
Description	**Value**	**Unit**	**Note**
Roof Weight	2.5	kg/m2	Metal
Ceiling Weight	18	kg/m2	
Roof (ADL)	0.3830	kN	Roof Weight×Tributary
Ceiling (ADL)	2.7572	kN	Ceiling Weight×Tributary
Purlin (DL)	0.1462	kN	Purlin Weight×Length
Total Dead Load	3.2863	kN	Roof+Ceiling+Purlin
Live Load (LL)	1	kN	SNI
Windward Wind Load (Wt)	-0.456	kN/m2	(+) comes to roof (-) goes off roof
Leeward Wind Load (Wh)	-0.322	kN/m2	(+) comes to roof (-) goes off roof
Windward Wind Load (Wt)	-6.9851	kN	(Wt)×Tributary
Leeward Wind Load (Wh)	-4.9378	kN	(Wh)×Tributary
Take Wind Load (WL)	-6.9851	kN	Max from Wt and Wh

The load in purlin can be projected into X and Y direction. Figure 2. 30 Load Projection of D, ADL, and LLshows the load projection of dead load, additional dead load, and live load. Figure 2. 31 shows the load projection of wind load.

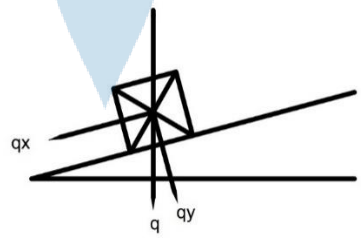


Figure 2. 30 Load Projection of D, ADL, and LL

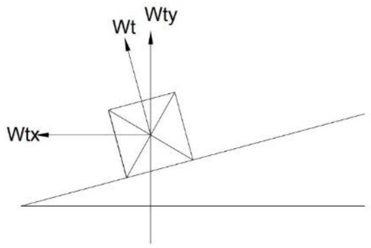


Figure 2. 31 Load Projection of WL

The load combination is taken from SNI 1727:2013 as follows

Combination 1: 1.4D

Combination 2: 1.2D + 0.5Lr

Combination 3: 1.2D + 1.6Lr + 0.5W

Combination 4: 1.2D + W + 0.5Lr

Combination 5: 0.9D + W

The maximum loading and moment of top and bottom purlin is shown in Table 2. 29. For middle purlin is shown in Table 2. 30

Table 2. 29 Maximum Loading and Moment of Top and Bottom Purlin

Top and Bottom Purlin	**X**	**Comb.**	**Y**	**Comb.**
Load (kN)	0.6624	2	2.5595	2
Moment (kNm)	0.3064	2	1.1838	2

Table 2. 30 Maximum Loading and Moment of Middle Purlin

Middle Purlin	**X**	**Comb.**	**Y**	**Comb.**
Load (kN)	1.1908	1	4.6008	1
Moment (kNm)	0.5507	1	2.1279	1

b. Profile Check

Because the purlin uses steel material, the profile must be checked as compact, non-compact, or slender material. The calculation is presented in

Table 2. 31. The flange is shown to be non-compact because $\lambda_p \leq \lambda \leq \lambda_r$ and the web is compact because $\lambda \leq \lambda_p$. Therefore, if one is compact and the other is non-compact, the profile is non-compact.

Table 2. 31 Profile Check for Compact, Non-Compact, and Slender

	λ	λp	λr	**Note**
Flange	25	10.9697	28.86751346	Non-compact
Web	62.5	108.542	164.5448267	Compact

Where the formula of λ, λ_p, and λ_r is presented in Table 2. 32.

Table 2. 32 Formula forλ, λ_p, and λ_r

Element	λ	λ_p	λ_r
Flange	$\frac{b_f}{2t_f}$ (I shape) $\frac{b_f}{t_f}$ (Channels)	$0.38\sqrt{\frac{E}{F_y}}$	$1.0\sqrt{\frac{E}{Fy}}$
Web	$\frac{h}{t_w}$	$3.76\sqrt{\frac{E}{F_y}}$	$5.7\sqrt{\frac{E}{F_y}}$

The profile must also be checked for lateral torsional buckling. This can be checked from the value of length Lb, Lp, and Lr (see Table 2. 33 and Table 2. 34). In Y direction load, the profile is inelastic lateral torsional buckling and in X direction load, the profile has no lateral torsional buckling. Therefore, the moment strength is different based on this condition.

Table 2. 33 Purlin Lb, Lp, and Lr for Y Direction

Lb	1850	mm
Lp	2484.453678	mm
Lr	745917.3565	mm

Table 2. 34 Purlin Lb, Lp, and Lr for X Direction

Lb	1850	mm
Lp	2484.453678	mm
Lr	745917.3565	mm

Where:

Lb = Unbraced length = L/2 .... eq. 2.19

$$Lp = 1.76r_y\sqrt{\frac{E}{F_y}} \quad \text{eq. 2.20}$$

$$Lr = 1{,}95r_{ts}\frac{E}{0{,}7F_y}\sqrt{\frac{J\,c}{S_x h_0}+\sqrt{\left(\frac{J\,c}{S_x h_0}\right)^2+6{,}76\left(\frac{0{,}7F_y}{E}\right)^2}} \quad \text{eq. 2.21}$$

c. Moment Strength

Moment strength of purlin as steel beam is determined by the condition of lateral torsional buckling. It can be inelastic, elastic, or no torsional buckling. The moment calculation shows in Table 2. 35 and Table 2. 36 for Y direction and X direction, respectively. It shows the factored moment strength (φMn) is bigger than the maximum moment (Mu) that acts in the purlin.

Table 2. 35 Moment Design Calculation for Y Direction Load

Description	Mn	Unit	Note
-	-		
Inelastic LTB	7117316.489	Nmm	$L_p < L_b \le L_r$
The flange is non-compact	3728060.753	Nmm	
Take φMn	3.355254678	kNm	0.9×smallest Mn
Mu for Top and Bottom Purlin	1.183758365	kNm	φMn>Mu Safe
Mu for Middle Purlin	2.1279	kNm	φMn>Mu Safe

Where:

Mn for Inelastic LTB = $C_b\left[M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{L_b-L_p}{L_r-L_p}\right)\right] \le M_p$; Cb = 1.3 (lateral bracing in the middle) .... eq. 2.22

Mp = $F_y.Z_x$ .... eq. 2.23

Table 2. 36 Moment Design Calculation for X Direction Load

Description	Mn	Unit	Note
-	-		
No LTB	1815550.035	Nmm	$L_b \le L_p$
The flange is non-compact	1116653.316	Nmm	
Take φMn	1.004987984	kNm	0.9×smallest Mn
Mu for Top and Bottom Purlin	0.306381179	kNm	φMn>Mu Safe
Mu for Middle Purlin	0.5507	kNm	φMn>Mu Safe

Where:

Mn for No LTB = $F_y.Z_x$

d. Shear Strength

Besides moment, shear strength must be checked. The shear strength is determined by the value of depth-thickess ratio (h/tw), ratio of critical web stress to shear yield stress (C_{v1}), resistance factor (φv), safety factor (Ωv), and shear factor (k_v). The calculation of shear design is presented in Table 2. 37 Purlin Shear Design Calculation

Table 2. 37 Purlin Shear Design Calculation

Description	Value	Unit	Note
h/tw	62.5		(a)
k_v	5.34		
1.1*√(kv/Fy)	73.37915235		(b)
Conclusion	(a)<=(b)		
C_{v1}	1		
φv	0.9		
Ωv	1.67		
Aw	250	mm2	H×t
Vn	36000	N	0.6Fy×Aw×Cv1
φVn	32.4	kN	
Vu of Top and Bottom Purlin	0.591879183	kN	φVn>Vu Safe
Vu of Middle Purlin	1.063927497	kN	φVn>Vu Safe

e. Displacement Check

Purlin acts like steel beam, so due to the loading it is subjected to displacement. According to Segui (2015), the maximum deflection that is still appropriate for steel beam can be taken as 1 inch or 25.5 mm. The deflection calculation is presented in Table 2. 38 and Table 2. 39 for Y direction and X direction, respectively.

Table 2. 38 Deflection at Y Direction

Description	Value	Unit	Note
δmax of Top and Bottom Purin	11.25392502	mm	Safe
δmax of Middle Purlin	20.22939922	mm	Safe
δallowed	25.5	mm	1 inch

Table 2. 39 Deflection at X Direction

Description	Value	Unit	Note
δmax of Top and Bottom Purlin	2.91274885	mm	Safe
δmax of Middle Purlin	5.2358772	mm	Safe
δallowed	25.5	mm	1 inch

Where:

δMax for Point Load acts on Beam = $\frac{PL^3}{48EI}$ .... eq. 2.24

2.9.2. Sag Rod

To provide alignment and lateral support for purlin, sag rod is used. Sag rod can help purlin from sagging due to lateral load and moment. On the top of truss, the most to purlin is attached with tie rod that has the same function. The calculation of sag rod and tie rod is presented in

Table 2. 40. The sag rod and tie rod will use the smallest diameter that is allowed in SNI 1729:2015 which is 12.7 mm.

Table 2. 40 Sag Rod and Tie Rod Calculation

Sag Rod Calculation			
T	0.662445793	kN	
Ab required	3.182922729	mm2	T/(0.75×0.75×Fu)
Diam. Required	2.013112785	mm	
Use Thread Rod diameter	12.7	mm	SNI 1729:2015 J3.4
Ab	126.6768698	mm2	
Tie Rod at Ridge			
P	0.662450051	kN	
Ab required	3.18294319	mm2	P/(0.75×0.75×Fu)
Diam. Required	2.013119256	mm	
Use Thread Rod diameter	12.7	mm	SNI 1729:2015 J3.4
Ab	126.6768698	mm2	

2.9.3. Truss

The truss uses steel profile 2Lx90x90x7. The properties of steel truss can be seen in Table 2. 41. The load on truss is the recap from load of purlin and the self-weight of truss itself. It is shown in Table 2. 42. Figure 2. 33 shows the truss model in ETABS, with

Figure 2. 34, Figure 2. 35, and Figure 2. 36 show the load act on truss for super dead load, live load, and wind load, respectively.

Table 2. 41 Truss Profile

2L x 90 x 90 7			
Steel Type	BJ37		
Fy	240	Mpa	
Fu	370	Mpa	
Ag for 1 angle	1222	mm2	
Ag for 2 angles	2444	mm2	
b	90	mm	
h	90	mm	
t	7	mm	
E	200000	Mpa	
r (radius of gyration	27.6	cm	
dbolt	16.00	mm	
dhole	18	mm	SNI 1729:2015 Table J3.3M
nbolt at edge for 1 column	1		
Cx	24.6	mm	
Cx (eccentricity center of gravity)	49.2	mm	2×Cx for 2L

Table 2. 42 Load Recap on Truss

Load Recap on Truss		
SDLmid	3.29	kN
SDLcorner	1.72	kN
L	1.00	kN
Wmid	-6.99	kN
Wcorner	-3.49	kN

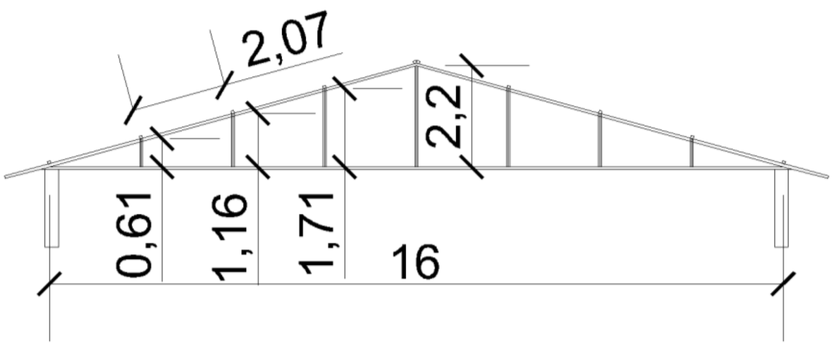


Figure 2. 32 Truss Preliminary Design

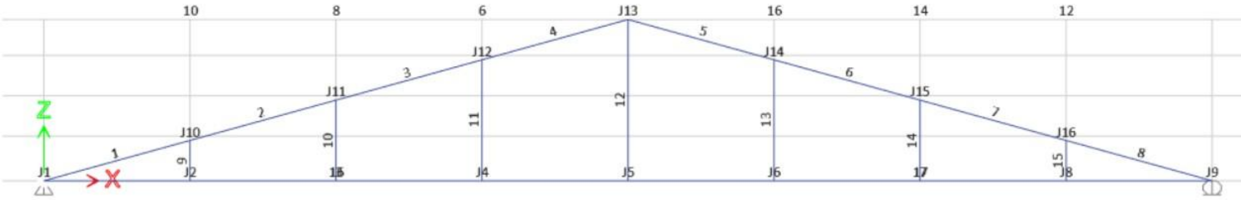


Figure 2. 33 Truss Model in ETABS

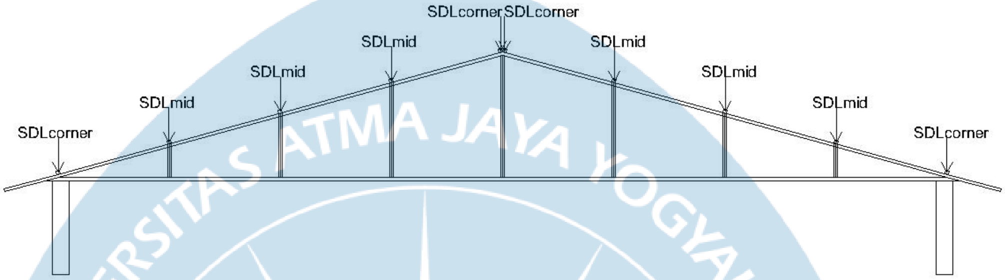


Figure 2. 34 Super Dead Load on Truss

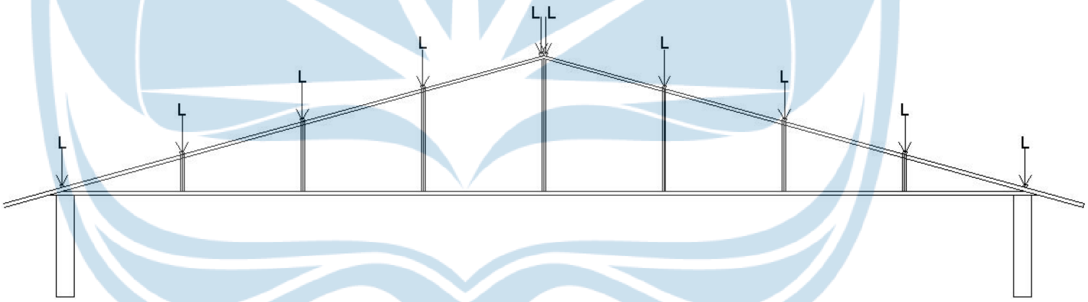


Figure 2. 35 Live Load on Truss

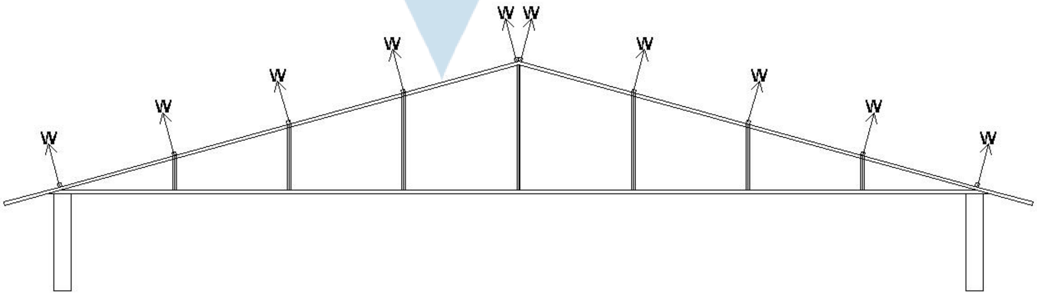


Figure 2. 36 Wind Load on Truss

a. ETABS Model Output

From ETABS, the output of axial force and displacement are analysed to check the requirement of truss strength. The axial force output is shown in Table 2. 43 and the displacement of each joint is shown in Table 2. 44.

Table 2. 43 Truss Axial Force Output

Member	Axial Force (kN)	Combination
1	-56.4922	Comb1
2	-51.7206	Comb1
3	-44.4211	Comb1
4	-37.8958	Comb1
5	-37.8958	Comb1
6	-44.4211	Comb1
7	-51.7206	Comb1
8	-56.4922	Comb1
9	-1.669	Comb1
10	-1.3069	Comb1
11	-4.0039	Comb1
12	8.0794	Comb1
13	-1.4298	Comb1
14	-1.3069	Comb1
15	-1.669	Comb1
16	56.4922	Comb1
17	56.4922	Comb1
Maximum	56.4922	

Table 2. 44 Truss Displacement

Joints	Displacement (mm)	Combination
J1	0	
J2	-14.279	Comb1
J3	-20.004	Comb1
J4	-15.302	Comb1
J5	-5.935	Comb1
J6	-15.302	Comb1
J7	-20.004	Comb1
J8	-14.279	Comb1
J9	0	
J10	-14.884	Comb1
J11	20.007	Comb1
J12	15.307	Comb1

Joints	Displacement (mm)	Combination
J13	-5.899	Comb1
J14	-15.307	Comb1
J15	-20.007	Comb1
J16	-14.281	Comb1
Maximum	20.007	

The maximum displacement shows to be smaller than 1 inch or 25.5 mm, so the truss is safe.

b. Tension Member Check

Truss consists of tension and compression member. These two members must be checked differently. For tension member, the tension strength is determined by effective length factor (K), net area (An), reduction factor for shear lag (Ue), effective area (Ae), and safety factor (φ). The member with maximum tension load is checked and safe from yield and rupture (see

Table 2. 45).

Table 2. 45 Tension Member Check

Check for Tension Member			
Pu max	56.4922	kN	
L	8000	mm2	
K	1		
Slenderness	289.8550725		KL/r
	KL/r<300 OK		
An	2192	mm2	Ag-n×dh×t×2 (2 for double angle)
Ue	0.99385		SNI 1729:2015 Table D3.1
Ae	2178.5192	mm2	An×Ue
Yield Strenth φPn	527.904	kN	0.9×Fy×Ag
	Safe		
Rupture Strenth φPn	604.539078	kN	0.75×Fu×Ae
	Safe		

c. Compression Member Check

For compression member, every member is checked. Th member compression strength is determined by effective length factor (K), steel modulus elasticity (E), steel yield strength (Fy), critical elastic buckling stress (Fe), critical compressive stress (Fcr), and safety factor (φ). All members show to be safe from compression load (see Table 2. 46).

Table 2. 46 Compression Member Check

Check for Compression Member								
Member	**Axial Force (kN)**	**Length (mm)**	$\frac{KL}{r}$	$4.71\sqrt{\frac{E}{Fy}}$	**Fe**	**Fcr**	**φPn**	**Note**
1	56.4922	2070	75.00	135.966	350.919	180.257	396.494	Safe
2	51.7206	2070	75.00	135.966	350.919	180.257	396.494	Safe
3	44.4211	2070	75.00	135.966	350.919	180.257	396.494	Safe
4	37.8958	2070	75	135.966	350.919	180.257	396.494	Safe
5	37.8958	2070	75	135.966	350.919	180.257	396.494	Safe
6	44.4211	2070	75	135.966	350.919	180.257	396.494	Safe
7	51.7206	2070	75	135.966	350.919	180.257	396.494	Safe
8	56.4922	2070	75	135.966	350.919	180.257	396.494	Safe
9	1.669	610	22	135.966	4040.99	234.108	514.943	Safe
10	1.3069	1160	42	135.966	1117.46	219.367	482.519	Safe
11	4.0039	1710	62	135.966	514.228	197.412	434.228	Safe
13	1.4298	1710	63	135.966	514.228	197.412	434.228	Safe
14	1.3069	1160	42	135.966	1117.46	219.367	482.519	Safe
15	1.669	610	22.1	135.966	4040.99	234.108	514.943	Safe

Where:

$$Fe = \frac{\pi^2 E}{\frac{KL^2}{r}} \quad \text{..... eq. 2.25}$$

$$Fcr\begin{cases} \frac{KL}{r} < 4.71\sqrt{\frac{E}{Fy}}, Fcr = 0.658Fy^{\frac{Fy}{Fe}} \\ \frac{KL}{r} > 4.71\sqrt{\frac{E}{Fy}}, Fcr = 0.877Fe \end{cases} \quad \text{..... eq. 2.26}$$

d. Slenderness Check

The last is to check the slenderness of truss. Since the truss is not slender, no more checking is required (see Table 2. 47).

Table 2. 47 Slenderness Check

λ	12.85714286	
λr	12.99038106	SNI 1279:2015 Table B4.1.a
λ<λr Not Slender		

Where:

$$\lambda = \frac{h}{tw} \quad \text{eq. 2.27}$$

$$\lambda_r = 0.45\sqrt{\frac{E}{Fy}} \quad \text{eq. 2.28}$$

2.9.4. Connection

Connection of truss will use bolt connection. The checking uses member with biggest axial force, which in joint J1 (see Figure 2. 37 and Table 2. 48 Joint with Maximum Axial Force). The properties of bolt can be seen in Table 2. 49 Bolt Properties.

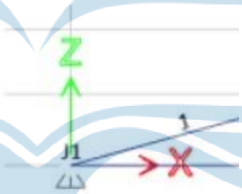


Figure 2. 37 Joint with Maximum Axial Force

Table 2. 48 Joint with Maximum Axial Force

Joint:	J1	
Member	Axial Force	
1	-56.4922	kN
16	56.4922	kN

Table 2. 49 Bolt Properties

Description	Value	Unit	Note
Bolt Type	Group A (A325)		
nbolt at edge for 1 column	1		
nbolt at other holes	1		
Total number of Bolt (n)	2		

Description	Value	Unit	Note
db	16.00	mm	
dh	18	mm	
Gusset Plate Type	BJ37		
Gusset Plate Thickness	12	mm	
Fy of Gusset Plate	240	Mpa	
Fu of Gusset Plate	370	Mpa	
Member thickness	7	mm	
Fy of Member	240	Mpa	
Fu of Member	370	Mpa	
Fu of Bolt	830	Mpa	A325
Nominal bolt shear strength, Fnv	372	Mpa	SNI 1729:2015 Table J3.2
Nominal bolt tensile strength, Fnt	620	Mpa	SNI 1729:2015 Table J3.2

a. Connection Strength

Bolt connection strength must be checked for shear strength, slip critical strength, bearing strength, yielding and rupture, and block shear strength. The total bolt that is preliminary designed is 2 bolts. The checking for 2 bolts strength to resist shear, slip critical, bearing force, yielding and rupture, and block shear are presented in table Table 2. 50, Table 2. 51, Table 2. 52, Table 2. 53, and Table 2. 54, respectively. The calculation shows that 2 bolts is enough to hold the member axial forces. Therefore 2 bolts are used as connection.

Table 2. 50 Double Shear Strength

Double Shear Strength for 1 bolt			
Ab	201.0619298	mm2	
Rn	149.5900758	kN	Fnv×Ab×2 for double shear, Fnv×Ab for one shear
Threads not in the shear plane			
Fnv(X)	466.875	Mpa	0.625×Fu×0.9
Rn	93.87078849	kN	
Threads in the shear plane			
Fnv(N)	373.5	Mpa	0.8×Fnv(X)
Rn	75.09663079	kN	
Used Shear Strength for 1 bolt			
Smallest Rn	75.09663079	kN	
φ	0.75		
φRn	56.32247309	kN	
φRn*n	112.6449462	kN	φRn×n>Pu Safe

Table 2. 51 Slip Critical Strength

Slip Critical Strength for 1 Bolt			
μ	0.3		coefficient of static friction for class A
Du	1.13		ratio of mean actual bolt pretension to minimum pretension
hf	1		1 filler=1.0, 2 fillers=0.85
Tb	91		Table J3.1M
ns	1		number of shear plane
φ	1		
φRn	30.849	kN	φ×μ×Du×hf×Tb×ns (Eq. J3.4)
φRn*n	61.698	kN	φRn×n>Pu Safe

Table 2. 52 Bearing Strength

Description	Value	Unit	Note
Bearing Strength			
minimum spacing	42.6667	mm	SNI 1729:2015 Section J3.3
actual spacing	45	mm	
ℓe minimum edge	22	mm	SNI 1729:2015 Table J3.4M
Actual ℓe minimum edge	25	mm	
Tension Member for 1 bolt			
t	7	mm	Member thickness
Fu	370	Mpa	Ultimate Strength
Rn for All holes	99.456	kN	2.4×d×t×Fu(member)
dh	18	mm	
ℓc near edge	16	mm	ℓe-h/2
Rn for edge holes	49.728	kN	1.2×ℓc×t×Fu
ℓc for other holes	27	mm	s-h
Rn for other holes	83.916	kN	1.2×ℓc×t×Fu
Used Rn for edge hole	49.728	kN	
Used Rn for other hole	83.916	kN	
nbolt at edge for 1 column	1		
nbolt at other holes	1		
Rn	133.644	kN	n(edge)×Rn(edge)+n(other)×Rn(other)
φ	0.75		
φRn	100.233	kN	
φRn*n	200.466	kN	φRn×n>Pu Safe
Gusset Plate for 1 bolt			
t	12	mm	
Fu	370	Mpa	
Rn for All holes	170.496	kN	2.4×d×t×Fu(member)
dh	18	mm	
ℓc near edge	16	mm	ℓe-h/2
Rn for edge holes	85.248	kN	1.2×ℓc×t×Fu

Description	Value	Unit	Note
ℓc for other holes	27	mm	s-h
Rn for other holes	143.856	kN	1.2×ℓc×t×Fu
Used Rn for edge hole	85.248	kN	
Used Rn for other hole	143.856	kN	
nbolt at edge for 1 column	1		
nbolt at other holes	1		
Rn	229.104	kN	n(edge)×Rn(edge)+n(other)×Rn(other)
φ	0.75		
φRn	171.828	kN	
φRn*n	343.656	kN	φRn×n>Pu Safe

Table 2. 53 Yielding and Rupture Bolt Strength

Description	Value	Unit	Note
Check Tension on Member			
Yielding			
Fy	240	Mpa	
Ag	2444	mm2	
Pn	586.56	kN	Fy×Ag
Yield Strenth φPn	527.904	kN	0.9×Pn
φRn×n	1055.808	kN	φRn×n>Pu Safe
Rupture			
Fu	370	Mpa	
Pu max	56.4922	kN	
L	8000	mm2	
K	1		
r (radius of gyration)	27.6	cm	
Slenderness	289.855072 5		KL/r
	KL/r<300 OK		
dh	18	mm	
t	7	mm	
Cx	24.6	mm	
Cx (eccentricity center of gravity)	49.2	mm	2×Cx for 2L
An	2192	mm2	Ag-n×dh×t×2 (2 for double angle)
Ue	0.99385		SNI 1729:2015 Table D3.1
Ae	2178.5192	mm2	An×Ue
Rupture Strenth φPn	604.539078	kN	0.75×Fu×Ae
φRn×n	1209.07815 6	kN	φRn×n>Pu Safe

Table 2. 54 Block Shear Strength

Description	Value	Unit	Note
Block Shear Strength			
For Tension Member			
t	7	mm	
bolt rows number	1		
Lgv	70	mm	(ℓc(edge) + s)×bolt row = Gross shear length
Agv	490	mm2	Lgv×t = Gross shear area
Lgn	25	mm	Gross net length
Agn	175	mm2	Lgn×t = Gross tension area
Lnv	43	mm	Net Shear Length
Anv	301	mm2	Lnv×t = Net shear area
Lnt	16	mm	Net Tension area
Ant	112	mm2	Net Tension area
Fu	370	Mpa	
Fy	240	Mpa	
Ubs	1		For uniform tension stress in angles, gusset plates, and coped beams
Rn	108.262	kN	0.6×Fu×Anv+Ubs×Fu×Ant
Rn upper limit	112	kN	0.6×Fy×Agv+Ubs×Fu×Ant
Used Rn	108.262	kN	
φRn*n	216.524	kN	φRn×n>Pu Safe
For Gusset Plate			
t	12	mm	
bolt rows number	1		
Lgv	70	mm	(ℓc(edge) + s) × bolt row = Gross shear length
Agv	840	mm2	Lgv×t = Gross shear area
Lgn	25	mm	Gross net length
Agn	300	mm2	Lgn×t = Gross tension area
Lnv	43	mm	Net Shear Length
Anv	516	mm2	Lnv×t = Net shear area
Lnt	16	mm	Net Tension area
Ant	192	mm2	Net Tension area
Fu	112.645	Mpa	
Fy	56.322	Mpa	
Ubs	1		For uniform tension stress in angles, gusset plates, and coped beams
Rn	56.503	kN	0.6×Fu×Anv+Ubs×Fu×Ant
Rn upper limit	50.014	kN	0.6×Fy×Agv+Ubs×Fu×Ant
Used Rn	50.014	kN	
φRn*n	100.029	kN	φRn×n>Pu Safe

2.9.5. Anchor

An anchor is a connection between steel and concrete. This connection is used to connect steel truss to the ring balk. The load from truss will be transferred to concrete beam from anchor. Therefore, anchor must be checked from several failure. This check will focus on compressive strength and shear strength of anchor. The anchor strength for shear strength is 68.7 kN, compression strength is 1740.375 kN, spalling shear strength is 70.04 kN, pry out strength is 113.49 kN, and breakout strength is 28.37 kN. The calculations are presented in Table 2. 57, Table 2. 58, Table 2. 60, Table 2. 61, and Table 2. 62, respectively.

Table 2. 55 Anchor Properties

Type	**Steel Headed Stud**			
Dmn	17.5	mm	>2.5*t(member)	SNI 1729:2015 Section I8.1
Actual D	18	mm		
Lmin	70	mm	>4*D	SNI 1729:2015 Section I8.2
Actual L	144	mm	h/d>8 =>h=8*D	Section I8.3 when subjected to shear and tension

Table 2. 56 Reaction and Maximum Load of Truss

Load			
Ru	18.83	kN	Reaction
Pu	56.4922	kN	Member Load

Table 2. 57 Anchor Shear Strength

Description	**Value**	**Unit**	**Note**
Shear Strength			
Asa	240.5281875	mm2	$0.25\pi D^2$
fc'	30	Mpa	
Ec	25742.9602	Mpa	4700×√(fc')
Rg	1		
Rp	1		
Fu	448	Mpa	Minimum fu (65 ksi)
0.5×Asa×√(fc'×Ec)	105688.0422	N	
Rg×Rp×Asa×Fu	107756.628	N	
Qnv	105.6880422	kN	SNI 1729:2015 Eq I8.1
φ	0.65		
φQnv	68.69722745	kN	

Table 2. 58 Anchor Compressive Strength

Description	**Value**	**Unit**	**Note**
Failure due to Compression on Concrete			
fc'	30	Mpa	
Ac	105000	mm2	
Pc	2677.5	kN	SNI 2847:2019 Table 17.3.1.1
φ	0.65		
φPc	1740.375	kN	φPc>Ru Safe

Table 2. 59 Anchor Dimension Properties for Shear Checking

Description	**Value**	**Unit**	**Note**
Failure due to Shear on Concrete			
ha = 1.5ca1	150	mm	SNI 2847:2019 Sec 17.5.2.4
ca1	100	mm	
ca2	45	mm	
maximum s	300	mm	SNI 2847:2019 Sec 17.2.1.1
take s	100	mm	
hef	144	mm	SNI 2847:2019 Fig R2.2

Table 2. 60 Anchor Spalling Shear Strength in Concrete

Description	**Value**	**Unit**	**Note**
Concrete spalling			
Asa	240.5281875		$0.25\pi D^2$
Fu	448	Mpa	
Vsa	107756.628	N	
φ	0.65		SNI 2847:2019 Sec 17.3.3
φVsa	70.04180821	kN	

Table 2. 61 Anchor Pry Out Shear Strength in Concrete

Description	**Value**	**Unit**	**Note**
Concrete pry out			
kcp	2		Sec 17.5.3.1
Ψec,N	1		Sec 17.4.2.4
Ψed,N	0.8389		Sec 17.4.2.5
Ψc,N	1.25		Sec 17.4.2.6
Ψcp,N	1		Sec 17.4.2.7 (Cast in type)
Anc	150176	mm2	Fig R17.4.2.1b
Anco	196992	mm2	Fig R17.4.2.1a
kc	10		
λ	1	normal	Sec 17.2.6 & 19.2.4
fc'	30	Mpa	

Description	Value	Unit	Note
Nb	94646.458	N	Eq. 17.4.2.2
Ncpg	75660.771	N	Ncpg=Ncbg Sec 17.5.3.1b
Vcpg	151321.542	N	
ϕ	0.75		Sec 17.3.3 Cond. A
ϕVcpg	113.491	kN	

Table 2. 62 Anchor Breakout Strength in Concrete

Description	Value	Unit	Note
Concrete breakout			
Ψec,V	1		Sec. 17.5.2.5
Ψed,V	1		Sec. 17.5.2.6
Ψc,V	1.4		Sec. 17.5.2.7
Ψh,V	1		Sec. 17.5.2.8
Avc	60000	mm2	Case 3 Fig R17.5.2.1.b
Avco	45000	mm2	
L	144	mm	anchor length
D	18	mm	anchor diameter
fc'	30	Mpa	
ca1	100	mm	
λ	1	normal	Sec 17.2.6 & 19.2.4
Vb1	21133.24207		Eq. 17.5.2.2a
Vb2	20265.73463		Eq. 17.5.2.2b
Take Vb	20265.73463		
Vcb	37.82937131	kN	Eq 17.5.2.1.b
ϕ	0.75		Sec 17.3.3 Cond. A
ϕVcb	28.37202848	kN	

a. Number of Anchor

The total number of anchors is calculated by comparing the smallest strength of anchor and the maximum load. It is shown that the smallest shear strength is 28.37 kN that holds the anchor from breakout and the maximum load is 56.49 kN. Therefore, the required anchor are 2 anchors for one side. Since the truss use double angle profile, for two sides the total anchor are 4.

Table 2. 63 Anchor Number

Recap Shear Strength		
φVsa	70.04180821	kN
ϕVcpg	113.4911568	kN

Recap Shear Strength		
ϕVcb	28.37202848	kN
Smallest ϕV	28.37202848	kN
n anchor	1.991123054	
n use	2	1 side
	4	2 sides

2.9.6. Conclusion

The purlin that is used here is steel C125x150x150x2 with length of 3.7 meter. Between the purlin lengths, a sag rod of diameter 12.7 mm is used to prevent purlin from sagging. The truss uses steel 2Lx90x90x7. The connection uses 2 bolts to connect each member on truss. The anchors use 4 anchors to connect steel truss and concrete ring balk.

2.10. Beam Design

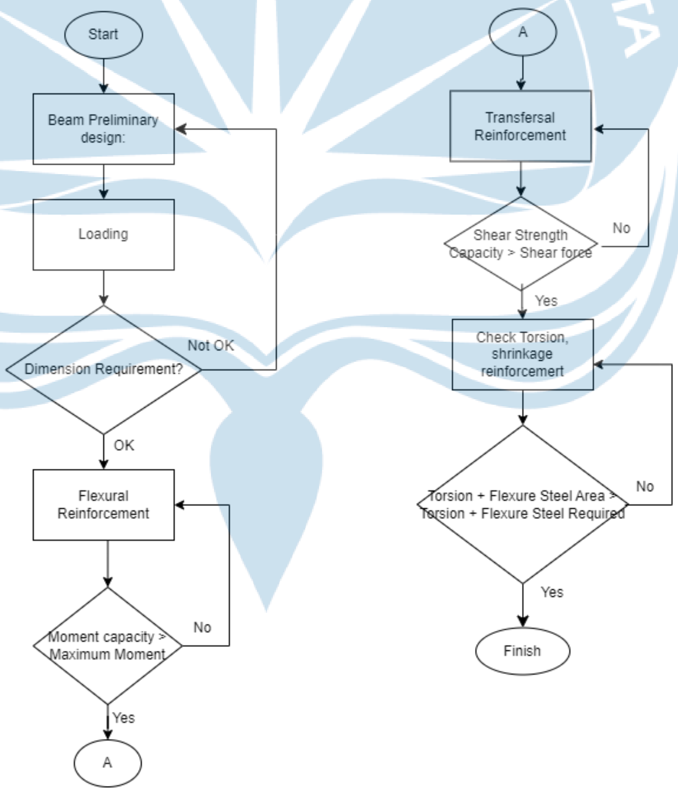


Figure 2. 38 Beam Design Flowchart

Beam uses reinforced concrete with dimension and properties is shown in Table 2. 64 According to SNI 2847:2019 table R18.2, beam for building with earthquake-resistant special moment structural system (SRPMK) must be designed as in SNI

2847:2019 article 18.6. In designing beam, the dimension of column beside the beam must be considered to calculate the clear length of beam. The dimension of column is presented in Table 2. 65. This calculation shows the beam with the longest length 5.5 meter and use normal concrete beam with λ is 1. The procedure of beam design is shown in Figure 2. 38.

Table 2. 64 Beam Properties

Beam	AB	Unit	Note
fc'	30	Mpa	
fy	400	Mpa	
bw	500	mm	beam width
hb	500	mm	beam height
Lb	5500	mm	beam length
cc	40	mm	cover
d_b	25	mm	longitudinal rebar diameter
d_{bt}	25	mm	middle longitudinal rebar diameter
d_c	13	mm	confinement diameter
lo	1000	mm	plastic hinge region/support
non lo	3400	mm	non plastic hinge region/span

Table 2. 65 Column Dimension

Left Column			
bcl	650	mm	left column width
hcl	650	mm	left column height
Lcl	4000	mm	left column length
Right Column			
bcr	650	mm	right column width
hcr	650	mm	right column height
Lcr	4000	mm	right column length

2.10.1. Loading

The beam loading is obtained from ETABS output from the internal forces of preliminary design model. The maximum axial load (Pu), negative moment at support, positive moment at support, and maximum moment at span are presented in Table 2. 66.

Table 2. 66 Beam Load

Description	Value	Unit	Note
Pu	283.8319	kN	ETABS Output
Mu at Support	-224.0967	kNm	ETABS Output
	199.6064	kNm	ETABS Output
Mu at Span	165.3817	kNm	ETABS Output

2.10.2. Dimension Requirement

According to SNI 2847:2019 the beam for SRPMK must meet several dimension requirements. This dimension requirement is calculated in Table 2. 67. The beam with 500x500 mm2 has meet the dimension requirement.

Table 2. 67 Dimension Requirements

Dimension Requirements			
d	434.5	mm	clear distance = hb – cc – dc - db/2
ln	4850	mm	clear length
Check ln>=4d	OK		SNI 2847:2019 chapter 18.6.2.1a
bw	500	mm	
0.3hb	150	mm	
Check bw>=0.3h	OK		SNI 2847:2019 chapter 18.6.2.1b
Check bw>=250mm	OK		SNI 2847:2019 chapter 18.6.2.1b
bc	650	mm	
3/4hc	487.5	mm	
min[bc;3/4hc]	487.5	mm	Minimum between bc or 3/4hc
Check if bw<=bc+2(min[bc;3/4hc])	OK		SNI 2847:2019 chapter 18.6.2.1c

2.10.3. Moment Strength and Longitudinal Reinforcement

Beam must hold bending moment from load. In order to increase the moment strength, longitudinal reinforcement is required. The condition must be checked for support area where it is two times beam height from column surface for negative moment and positive moment. Then, in the span area beam must be checked from ultimate moment. The maximum ratio of steel must meet the SNI 2847:2019 chapter 18.6.3.1. The calculation of moment strength is presented in Table 2. 68

Table 2. 68 Moment Strength and Flexural Reinforcement

Description	Value	Unit	Note
$\rho = \frac{0.85fc'}{fy}\left[1-\sqrt{1-\frac{4Mu}{1.7\phi fc'bd^2}}\right]$			
At Support with Negative Moment			
Mu	224.0967	kNm	
φ	0.9		
0.85fc'/fy	0.06375		
4Mu/(1.7×φfc'bd^2)	0.206887		
ρ	0.006976231		
ρmax	0.025		SNI 2847:2019 chapter 18.6.3.1
ρmin	0.0035		SNI 2847:2019 chapter 9.6.1.2
ρmin<ρ<ρmax	OK		
ρuse	0.006976231		
As	1515.586267	mm2	ρbd
req n steel	3.08752699		As/(0.25πdb^2)
Use nlong steel	4		
As use	1963.495408	mm2	
a	61.59985595	mm2	(Asfy)/(0.85fc'b)
Mn	317065295.1		(Asfy)(d-a/2)
φMn	285.3587656	kNm	
Check φMn>Mu	Safe		
At Support with Positive Moment			
Mu	199.6064	kNm	
φ	0.9		
0.85fc'/fy	0.06375		
4Mu/(1.7*φfc'bd^2)	0.184277453		
ρ	0.006172683		
ρmax	0.025		SNI 2847:2019 chapter 18.6.3.1
ρmin	0.0035		SNI 2847:2019 chapter 9.6.1.2
ρmin<ρ<ρmax	OK		
ρuse	0.006172683		
As	1341.015421	mm2	ρbd
req n steel	2.731894183		As/(0.25πdb^2)
Use nlong steel	3		
As use	1472.621556	mm2	
a	46.19989196	mm2	(Asfy)/(0.85fc'b)
Mn	242334635.1		(Asfy)(d-a/2)
φMn	218.1011716	kNm	
Check φMn>Mu	Safe		
At Support with Positive Moment			
Mu	199.6064	kNm	

Description	Value	Unit	Note
φ	0.9		
0.85fc'/fy	0.06375		
4Mu/(1.7×φfc'bd^2)	0.184277453		
ρ	0.006172683		
ρmax	0.025		SNI 2847:2019 chapter 18.6.3.1
ρmin	0.0035		SNI 2847:2019 chapter 9.6.1.2
ρmin<ρ<ρmax	OK		
ρuse	0.006172683		
As	1341.015421	mm2	ρbd
req n steel	2.731894183		As/(0.25πdb^2)
Use nlong steel	3		
As use	1472.621556	mm2	
a	46.19989196	mm2	(Asfy)/(0.85fc'b)
Mn	242334635.1		(Asfy)(d-a/2)
φMn	218.1011716	kNm	
Check φMn>Mu	Safe		
At Span			
Mu	165.3817	kNm	
φ	0.9		
0.85fc'/fy	0.06375		
4Mu/(1.7×φfc'bd^2)	0.152681069		
ρ	0.005068171		
ρmax	0.025		SNI 2847:2019 chapter 18.6.3.1
ρmin	0.0035		SNI 2847:2019 chapter 9.6.1.2
ρmin<ρ<ρmax	OK		
ρuse	0.005068171		
As	1101.060078	mm2	ρbd
req n steel	2.243061172		As/(0.25πdb^2)
Use nlong steel	3		
As use	1472.621556	mm2	
a	46.19989196	mm2	(Asfy)/(0.85fc'b)
Mn	242334635.1		(Asfy)(d-a/2)
φMn	218.1011716	kNm	
Check φMn>Mu	Safe		

Table 2. 69 Check Requirements

Requirements	Note	Reference
1. At least 2 longitudinal reinforcement steel	OK	SNI 2847:2019 chapter 18.6
2. φMn (+) $_{left}$>=0.5φMn (-) $_{left}$	OK	SNI 2847:2019 chapter 18.6
φMn (+) $_{right}$>=0.5φMn (-) $_{right}$	OK	SNI 2847:2019 chapter 18.6
3. φMn(+) or φMn(-)>=1/4*Max(φMn)	OK	SNI 2847:2019 chapter 18.6

2.10.4. Shear Strength and Transversal Reinforcement

According to SNI 2847:2019 chapter 18.6.1.1, beam in SRPMK must be designed for moment and shear. According to SNI 2847:2019 chapter 18.6.5.2, SRPMK beam's shear strength (Vc) must assumed to 0 if shear force due to earthquake load (Veq) is bigger than 50% beam shear strength (Vu) due to beam's probability moment (Mpr). In this condition, transversal reinforcement must be placed in order to increase the shear strength of the reinforced beam. Probability moment (Mpr) according to SNI 2847:2019 chapter 18.6.5.1 is a maximum moment that works at joint's face and beam. The moment must be considered with the increase of yield stress that is not least than 1.25fy according to SNI 2847:2019 R18.6.5. The calculation of shear strength and transversal reinforcement is presented in Table 2. 70.

Table 2. 70 Beam Shear Strength and Transversal Reinforcement

Description	**Value**	**Unit**	**Note**
Transversal Reinforcement			
At support			
For 4D25			
a	76.9999	mm	As×1.25fy/(0.85fc'b)
Mpr(-)	388.7722	kNm	As×1.25fy(d-a/2)
For 3D25			
a	57.7499	mm	As×1.25fy/(0.85fc'b)
Mpr(+)	298.6661	kNm	As×1.25fy(d-a/2)
Shear Strength			
qu	51.6058	kN/m	
Vleft	266.8839	kN	(Mpr(-) +Mpr(+))/ln+qu×ln/2
Vright	16.5958	kN	(Mpr(+) +Mpr(-))/ln-qu×ln/2
Take Vu	266.8839	kN	Ma× Vleft or Vright
V due to earthquake only	141.7399	kN	
50% Total V	133.4420	kN	
Vc	0.0000	kN	Veq>0.5Vu, then Vc=0
φ	0.7500		
Vs	355.8452	kN	Vu/φ
0.66×√(fc')bw.d	785.3520	kN	
Check if Vs<0.66√(fc').bw.d	OK		
Confinement legs	2		
Av use of dc=13 mm	132.7323	mm2	
s	129.6568	mm	legs×As×fy×d/Vs
	125		

Description	Value	Unit	Note
d/4	108.625	mm	
6db	150	mm	
ma× 150 mm	150	mm	
Ma× confinement spacing at 2h=1000mm <	108.625	mm	min from d/4,6db, and 150mm
s use	100	mm	
L of confinement	900	mm	
Transversal Reinforcement use	D13-100	mm	Along 900 mm from support
First confinement with s=50 mm from support	50	mm	
At span			
Vu	220.4387	kN	
λ	1		
Vc	202.2876	kN	
φ	0.75		
Vs	91.6306	kN	Vu/φ-Vc
0.33×√(fc')bw.d	392.6760	kN	
legs	2		
Av use of dc=10 mm	132.7323	mm2	
s1	503.5188	mm	legs×As×fy×d/Vs
s2	217.2500	mm	d/2
s3	606.7762	mm	legs×Av×fy/(0.35bw)
s4	600	mm	
min s	217.2500	mm	
s use	200	mm	
Check s	OK		
Transversal Reinforcement use	D13-200	mm	

2.10.5. Torsion Check and Shrinkage Reinforcement

Beside moment and shear, beam can be subjected to torsion. In this check, shrinkage reinforcement or middle reinforcement is added to prevent the beam from shrinkage and torsion. As stated in PBI 1971 N.I.-2 chapter 8.16.2 that the maximum vertical spacing of concrete without longitudinal bar is 300 mm, so a middle reinforcement must be added if it is larger than 300 mm. The beam torsion check is in Table 2. 71, the torsion transversal reinforcement is presented in Table 2. 72, and the torsion longitudinal reinforcement is presented in Table 2. 73.

Table 2. 71 Torsion Check

Description	Value	Unit	Note
Acp	250000	mm2	
pcp	2000	mm2	
φ	0.75		Table 21.2.1
Tcr	10.6977062	kNm	φ*√fc'/12*(Acp2/pcp)
Tu	16.9776	kNm	
Check $Tu > \varphi \times \sqrt{fc'}/12 \times \left(\frac{Acp2}{pcp}\right)$	Torsion must be checked		Table 22.7.4.1
bb clear	407	mm	
hb clear	407	mm	
Aoh	165649	mm2	
ph	1628	mm	
Vu	266.8839	kN	
$\left(\frac{Vu}{bw \times d}\right)^2$	1.5091		
$\left(\frac{Tu \times ph}{1.7Aoh^2}\right)^2$	0.3511		
Ultimate Shear + Torsion	1.3639	Mpa	√((Vu/(bw*d))^2+((Tu*ph)/(1.7Aoh^2))^2) $\sqrt{(\left(\frac{Vu}{bw*d}\right)^2 + \left(\frac{Tu \times ph}{1.7Aoh^2}\right)^2}$
Torsion Capacity	3.4096	Mpa	φ(0.17×√fc'+0.66×√fc')
Check beam torsion strength	Safe		

Table 2. 72 Torsion Transversal Reinforcement

Description	Value	Unit	Note	Reference
$\phi\left(\frac{2A_oA_tf_{yv}cot\theta}{s}\right) \geq Tu$				
Ao	140801.65	mm2		
ϴ	45	o		SNI 2847:2019 chapter 22.7.6.1.2
φ2Ao × fyv × cotϴ	84480990			
At/s	0.200963554	mm2/m	$\frac{Tu}{\varphi 2Ao \times fyv \times cot\theta}$	SNI 2847:2019 chapter 22.7.6.1
s max 1	203.5	mm	ph/8	SNI 2847:2019 chapter 9.7.6.3.3
s max 2	300	mm	300 mm	SNI 2847:2019 chapter 9.7.6.3.3
s support	100	mm		
s span	200	mm		
check s suport	OK		s support >= s max?	

Description	Value	Unit	Note	Reference
check s span	OK		s span >= s max?	
A_{v+t} / s Support Use	2.6546	mm2/mm	n ×π/4 ×dc^2 / s	
A_{v+t} / s Span Use	1.3273	mm2/mm	n ×π/4 ×dc^2 / s	
A_v / s Support Need	2.0474	mm2/mm	(V_u Support / φ - V_c) / (f_y × d)	
A_v / s Span Need	0.5272	mm2/mm	(V_u Span / φ - V_c) / (f_y × d)	
A_{v+t} / s Support Need	2.4494	mm2/mm	2 × A_t / s + A_v / s	SNI 2847:2019 figure. R9.5.4.3
A_{v+t} / s Span Need	0.9291	mm2/mm	2 × A_t / s + A_v / s	SNI 2847:2019 figure. R9.5.4.3
A_{v+t} / s min 1	0.4245	mm2/mm	0.062 × (f_c')$^{0.5}$ × b / f_y	SNI 2847:2019 chapter 9.6.4.2
A_{v+t} / s min 2	0.4375	mm2/mm	0.35 × b / f_y	SNI 2847:2019 chapter 9.6.4.2
Check Shear+Torsion Support	OK		A_{v+t} / s Support Use>= A_{v+t} / s Need and Min ?	
Check Shear+Torsion Span	OK		A_{v+t} / s Span Use>= A_{v+t} / s Need and Min?	

Table 2. 73 Torsion Longitudinal Reinforcement

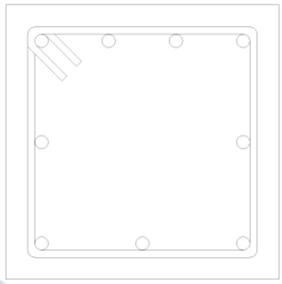
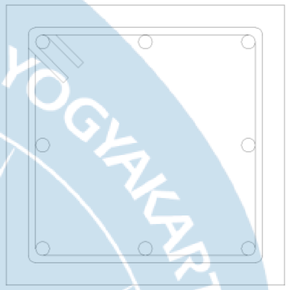
Description	Value	Unit	Note	Reference
d_b	25	mm		
d_b, min	8	mm	0.042 s	SNI 2847:2019 chapter 9.7.5.2
Chek d_b	OK			
As Support Top Need	1963.4954	mm2		
As Support Bottom Need	1341.0154	mm2		
As Span Top Need	1101.0601	mm2		
As Span Bottom Need	1101.0601	mm2		
A_l	327.1687	mm2	A_t / s × P_h	SNI 2847:2019 chapter 22.7.6.1
A_l min	1110.6030	mm2	0.42 × (f_c')$^{0.5}$ × A_{cp} / f_y - (A_t/s) × P_h	SNI 2847:2019 chapter 9.6.4.3

Description	Value	Unit	Note	Reference
$A_s + A_l$ Support need	4415.1139	mm2		
$A_s + A_l$ Span need	3312.7232			
n Support Top	4			
n Support Middle	2		multiply of 2	
n Support Bottom	3			
n Support Vertical	3		2 + n Middle / 2	
n Span Top	3			
n Span Middle	2		multiply of 2	
n Span Bottom	3			
n Span Vertical	3		2 + n Middle / 2	
Horizontal Support s	123	mm	$(b - 2c_c - 2d_s - d_b)$ / [min (n top, n bot) - 1]	
Vertical Support s	184.5	mm	$(h - 2c_c - 2d_s - d_b)$ / (n Vertical - 1)	
Horizontal Span s	123	mm	$(b - 2c_c - 2d_s - d_b)$ / [min (n top, n bot) - 1]	
Vertical Span s	184.5	mm	$(h - 2c_c - 2d_s - d_b)$ / (n Vertical - 1)	
Check Longitudinal Spacing Support	OK		s<=300 mm?	
Check Longitudinal Spacing Span	OK		s<=300 mm?	
$A_s + A_l$ Suport Use	4417.864669	mm2	(n top+n bot) $\times 0.25 \times \pi \times d_b^2$ + n mid$\times 0.25 \times \pi \times d_{bt}^2$	
$A_s + A_l$ Span Use	3926.990817	mm2	(n top+n bot) $\times 0.25 \times \pi \times d_b^2$ + n mid$\times 0.25 \times \pi \times d_{bt}^2$	
Check Flexure + Torsion Support	OK		$A_s + A_l$ Use>= $A_s + A_l$ Need?	
Check Flexure + Torsion Span	OK		$A_s + A_l$ Use>= $A_s + A_l$ Need?	

2.10.6. Conclusion

In conclusion the beam is 500x500 mm2 with a length of 5.5 meter. The concrete strength is fc’ 30 MPa and steel fy 400 MPa. The concrete cover is 40 mm. The recap for reinforcement is presented in Table 2. 74. With hook use 135° and length of 78 mm.

Table 2. 74 All Beam Reinforcement Recap

Reinforcement Recap			
At support	Top	4D25	
	Middle	2D25	
	Bottom	3D25	
	Confinement	D13-100	
At span	Top	3D25	
	Middle	2D25	
	Bottom	3D25	
	Confinement	D13-200	
Transversal Hook Length			
Hook Type	135	o	
l_{ext}	78	mm	

2.11. Tie Beam

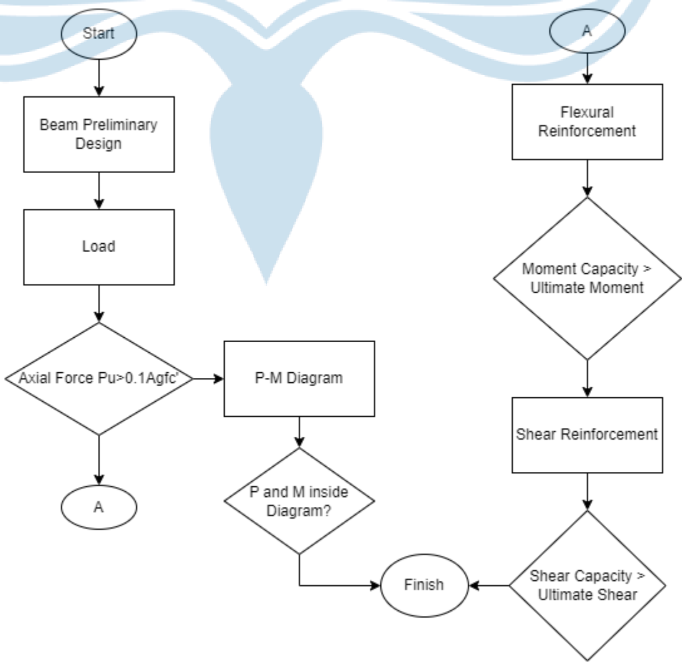


Figure 2. 39 Tie Beam Design Flowchart

Tie beam is a beam that is placed on the ground to transfer the load from column to foundation. The properties of tie beam can be seen in Table 2. 75. The procedure of tie beam design is shown in Figure 2. 39.

Table 2. 75 Tie Beam Properties

Description	Value	Unit	Eq.	Reference
NET Length of the Tie Beam, Ln	3000	mm		
Tie Beam Width, b	350	mm		
Tie Beam Height, h	350	mm		
Diameter of Longitudinal Reinforcement, db	19	mm		
Diameter of Stirrup Reinforcement, ds	10	mm		
Concrete Cover, cc	40	mm		
Effective Tie Beam Height, d	290.5	mm		
Concrete Compressive Strength, fc'	30	MPa		
Yield Strength of Longitudinal Reinforcement, fy	400	MPa		
Yield Strength of Transverse Reinforcement, fyv	400	MPa		
β1	0.8357		0.65 <= 0.85 - 0.05 * (fc' - 28) / 7 <= 0.85	SNI 2847:2019 Table 22.2.2.4.3
λ	1		Assuming not using lightweight concrete	
Minimum Width Requirements	OK		b >= min (Ln/20, 450 mm) ?	SNI 2847:2019 Article 18.13.3.2

2.11.1. Loading

The axial force that works on tie beam is in Table 2. 76. The loads that work on tie beam are from column, different settlement of foundation, and gravitational load. Because the axial force (Pu) is smaller than 10% of cross-sectional area times compressive strength, the axial strength does not need to be calculated but rather the flexural reinforcement must be considered.

Table 2. 76 Calculation of Axial Force

Description	Value	Unit	Eq.	Reference
Column Axial Force Due to Factored Gravity Load, Pg	1916.424 5	kN	Combination 1.2 D + 1.6 L	
Parameter of Spectral Response Acceleration in Short Period, SDS	1.5000	g	Input	
Tie Beam Axial Force, Pu	287.4637	kN	10% * SDS * Pg	SNI 1726:2019 Article 7.13.6.2
Axial Forces Should Be Calculated?	Not Required		Pu > 0.1 Ag fc'?	
Calculation of Internal Force Due to Differential Settlement				
Concrete Modulus of Elasticity, Ec	29725.41 00	MPa	4700 √fc'	SNI 2847:2019 Article 19.2.2
Cross-Section Inertia, Ig	12505208 33.3333	mm4	1/12 b h3	
Differential Settlement, Δ	5.4000	mm	Input (Ln/300 can be used if there is no data)	SNI 8460:2017 Article 9.2.4.3
Suppport Moment Due to Differential Settlement, Mdiff	133.8201	kNm	6 * Ec * Ig * Δ / Ln2	Hibbeler, R.C. "Structural Analysis"
Support Shear Force Due to Differential Settlement, Vdiff	89.2134	kN	dM/dx(x=L) = 12 Ec Ig Δ / Ln3	
Calculation of Internal Force Due to Gravitational Loads				
Reinforced Concrete Specific Gravity, BJc	23.5360	kN/m 3	Input	
Uniform Load Due to Self-Weight, qDL	2.8832	kN/m	BJc * b * h	
Level Height, hn	4.0000	m	Input	
Partition Wall Load per m2, qA,partition wall	2.5000	kN/m 2		
Uniform Weight Due to Partition Wall Load, qSIDL	10.0000	kN/m	qA,partition wall * hn	
Ultimate Uniform Load Due to Gravitational Loads, qD	18.0364	kN/m	1.4 (qDL + qSIDL)	
Support Ultimate Moment Due to Gravitational Loads, MD,sup	-13.5273	kNm	-1/12 * qD * Ln2	
Span Ultimate Moment Due to Gravitational Loads, MD,spa	6.7637	kNm	1/24 * qD * Ln2	

Description	Value	Unit	Eq.	Reference
Support Ultimate Shear Force Due to Gravitational Loads, VD,sup	27.0546	kN	qD * Ln / 2	
Span Ultimate Shear Force Due to Gravitational Loads, VD,spa	13.5273	kN	qD * Ln / 4	

2.11.2. Longitudinal Reinforcement

Since the axial load that works on tie beam is smaller than 0.1Agfc', the longitudinal reinforcement for flexure must be considered. The check will be determined by the moment at negative support, positive support, negative span, and positive span. The calculation of longitudinal reinforcement is presented in Table 2. 77.

Table 2. 77 Tie Beam Flexural Reinforcement

Description	Value	Unit	Eq.	Reference
Negative Support				
Amount of Support Negative Reinforcement, n	6.0000			
db	19.0000	mm		
Net Spacing for Each Reinforcement	46.2000	mm	(b - 2 cc - 2 ds - db) / (n - 1)	
Net Distance Check	OK		Net Spacing >= db and 25 mm?	SNI 2847:2019 Article 25.2.1
Number of Layers	2.0000			
As use	1701.1724	mm2	n *π/4 *db2	
As min,1	348.0606	mm2	(fc')0.5 / (4 * fy) * b * d	SNI 2847:2019 Article 9.6.1.2
As min,2	355.8625	mm2	1.4 / (4 × fy) × b × d	SNI 2847:2019 Article 9.6.1.2
Check As min	OK		As Use >= As min?	
a	76.2430	mm	As × fy / (0.85 × fc' × b)	SNI 2847:2019 Article 22.2.2.4.1
Mn	171.7357	kN-m	As × fy × (d - a/2)	SNI 2847:2019 Article 22.2.2.4.1
c	91.2310	mm	a / β1	SNI 2847:2019 Article 22.2.2.4.1
εs	0.0066		(d - c) / c × 0.003	SNI 2847:2019 Article 22.2.1.2, 22.2.2.1
ϕ	0.9000		0.65 <= 0.65 + (εs - 0.002) / 0.003 × 0.25 <= 0.9	SNI 2847:2019 Table 21.2.2

Description	Value	Unit	Eq.	Reference
ϕMn	154.5622	kN-m	ϕ × Mn	
Mu,support (-)	147.3474	kN-m	0.0000	
Check Capacity	OK		ϕMn > Mu ?	
Positive Support				
n	6.0000			
db	19.0000	mm		
Net Spacing for Each Reinforcement	46.2000	mm	(b - 2 cc - 2 ds - db) / (n - 1)	
Net Distance Check	OK		Net Spacing >= db and 25 mm?	SNI 2847:2019 Article 25.2.1
Number of Layers	2.0000			
As use	1701.1724	mm2	n ×π/4 ×db2	
As min,1	348.0606	mm2	(fc')0.5 / (4 × fy) × b × d	SNI 2847:2019 Article 9.6.1.2
As min,2	355.8625	mm2	1.4 / (4 × fy) × b × d	SNI 2847:2019 Article 9.6.1.2
Check As min	OK		As Use >= As min ?	
a	76.2430	mm	As × fy / (0.85 × fc' × b)	SNI 2847:2019 Article 22.2.2.4.1
Mn	171.7357	kN-m	As × fy × (d - a/2)	SNI 2847:2019 Article 22.2.2.4.1
c	91.2310	mm	a / β1	SNI 2847:2019 Article 22.2.2.4.1
εs	0.0066		(d - c) / c × 0.003	SNI 2847:2019 Article 22.2.1.2, 22.2.2.1
ϕ	0.9000		0.65 <= 0.65 + (εs - 0.002) / 0.003 × 0.25 <= 0.9	SNI 2847:2019 Table 21.2.2
ϕMn	154.5622	kN-m	ϕ × Mn	
Mu	133.8201	kN-m		
Check ϕMn > Mu	OK		ϕMn > Mu ?	
Negative Span				
n	3.0000			
db	19.0000	mm		
Net Spacing for Each Reinforcement	115.5000	mm	(b - 2 cc - 2 ds - db) / (n - 1)	
Net Distance Check	OK		Net Spacing >= db and 25 mm?	SNI 2847:2019 Article 25.2.1
Number of Layers	2.0000			

Description	Value	Unit	Eq.	Reference
As use	850.5862	mm2	n × π/4 ×db2	
As min,1	348.0606	mm2	(fc')0.5 / (4 × fy) × b × d	SNI 2847:2019 Article 9.6.1.2
As min,2	355.8625	mm2	1.4 / (4 × fy) × b × d	SNI 2847:2019 Article 9.6.1.2
Check As min	OK		As Use >= As min?	
Positive Span				
n	3.0000			
db	19.0000	mm		
Net Spacing for Each Reinforcement	115.5000	mm	(b - 2 cc - 2 ds - db) / (n - 1)	
Net Distance Check	OK		Net Spacing >= db and 25 mm?	SNI 2847:2019 Article 25.2.1
Number of Layers	2.0000			
As use	850.5862	mm2	n ×π/4 ×db2	
As min,1	348.0606	mm2	(fc')0.5 / (4 × fy) × b × d	SNI 2847:2019 Article 9.6.1.2
As min,2	355.8625	mm2	1.4 / (4 × fy) × b × d	SNI 2847:2019 Article 9.6.1.2
Check As min	OK		As Use >= As min?	
a	38.1215	mm	As × fy / (0.85 × fc' × b)	SNI 2847:2019 Article 22.2.2.4.1
Mn	92.3530	kN-m	As × fy × (d - a/2)	SNI 2847:2019 Article 22.2.2.4.1
c	45.6155	mm	a / β1	SNI 2847:2019 Article 22.2.2.4.1
εs	0.0161		(d - c) / c × 0.003	SNI 2847:2019 Article 22.2.1.2, 22.2.2.1
ϕ	0.9000		0.65 <= 0.65 + (εs - 0.002) / 0.003 × 0.25 <= 0.9	SNI 2847:2019 Table 21.2.2
ϕMn	83.1177	kN-m	ϕ × Mn	
Mu	6.7637	kN-m		
Check ϕMn > Mu	OK		ϕMn > Mu ?	

2.11.3. Transversal Reinforcement

The total shear strength of beam is determined by the shear strength of concrete itself (Vc) and the shear strength of steel (Vs). The shear strength will be checked at support and span. The calculation of shear strength and transversal reinforcement is presented in Table 2. 78.

Table 2. 78 Shear Strength and Transversal Reinforcement

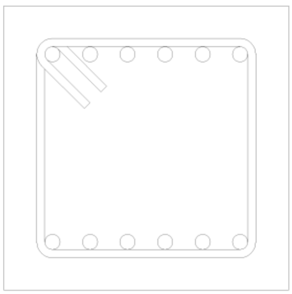
Description	**Value**	**Unit**	**Eq.**	**Reference**
Support				
Amount of Footing	2.0000			
Av	157.0796	mm2	n ×π/4 ×ds2	
Spacing	75.0000	mm	Input	
Vu	116.2680	kN		
ϕ	0.7500			SNI 2847:2019 Article 12.5.3.2, 21.2.4
Vu / ϕ	155.0240	kN		
Maximum Spacing Specifier Limit	183.7760	kN	0.33 × (fc')0.5 × b × d	SNI 2847:2019 Article 9.7.6.2.2
Max Spacing 1	145.2500	mm	d / 4 or d / 2	SNI 2847:2019 Article 9.7.6.2.2
Max Spacing 2	600.0000	mm	300 mm or 600 mm	SNI 2847:2019 Article 9.7.6.2.2
Check Spacing	OK			
Vs	243.3687	kN	Av × fy × d / s	SNI 2847:2019 Article 22.5.10.5.3
Boundary Vs	367.5520	kN	0.66 × (fc')0.5 × b × d	SNI 2847:2019 Article 22.5.1.2
Vc	94.6725	kN	0.17 × (fc')0.5 × b × d	SNI 2847:2019 Article 22.5.5.1
Vn	338.0412	kN	Vc + Vs	
ϕVn	253.5309			
Check Capacity	OK		ϕVn >= Vu?	
Span				
Amount of Footing	2.0000			
Av	157.0796	mm2	n ×π/4 ×ds2	
Spacing	100.0000	mm		
Vu	13.5273	kN		
ϕ	0.7500			SNI 2847:2019 Article 12.5.3.2, 21.2.4
Vu / ϕ	18.0364	kN		

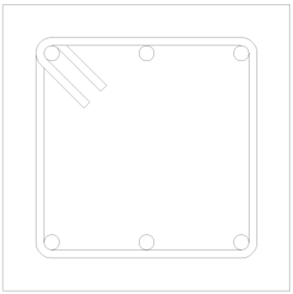
Description	Value	Unit	Eq.	Reference
Maximum Spacing Specifier Limit	183.7760	kN	0.33 × (fc')0.5 × b × d	SNI 2847:2019 Article 9.7.6.2.2
Max Spacing 1	145.2500	mm	d / 4 atau d / 2	SNI 2847:2019 Article 9.7.6.2.2
Max Spacing 2	600.0000	mm	300 mm or 600 mm	SNI 2847:2019 Article 9.7.6.2.2
Check Spacing	OK			
Vs	182.5265	kN	Av × fy × d / s	SNI 2847:2019 Article 22.5.10.5.3
Boundary Vs	367.5520	kN	0.66 × (fc')0.5 × b × d	SNI 2847:2019 Article 22.5.1.2
Vc	94.6725	kN	0.17 × (fc')0.5 × b × d	SNI 2847:2019 Article 22.5.5.1
Vn	277.1990	kN	Vc + Vs	
ϕVn	207.8993			
Check Capacity	OK		ϕVn >= Vu?	

2.11.4. Conclusion

The tie beam will use reinforced concrete 350x350 mm2 and the recap for tie beam is presented in Table 2. 79. The hook length is 75 mm using 135^{o} bending.

Table 2. 79 Tie Beam Reinforcement Recap

Reinforcement Recap		
Support Top	6D19	
Support Bottom	6D19	
Confinement	1D10-75	
Span Top	3D19	
Span Bottom	3D19	

Reinforcement Recap		
Confinement	1D10-100	
Transversal Hook Length		
Hook Type	135	o
l_{ext}	75	mm

2.12. Column Design

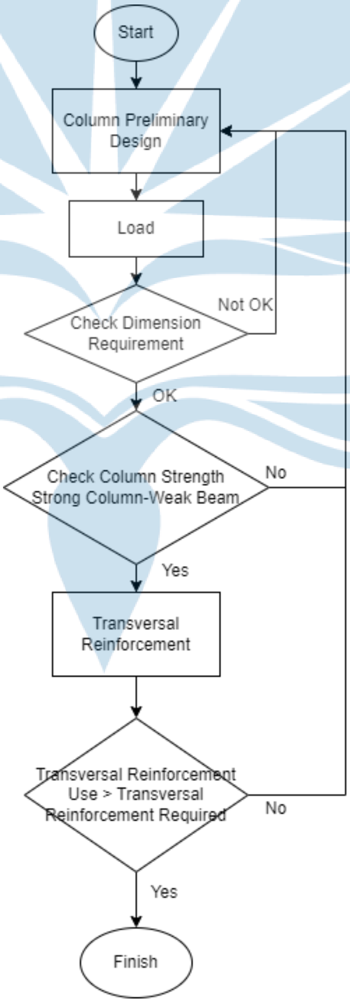


Figure 2. 40 Column Design Flowchart

The column for SRPMK building must meet the requirement from SNI 28147:2019 chapter 18.7. The preliminary design of column dimension is presented in Table 2. 80. The procedure to design a column is shown in Figure 2. 40.

Table 2. 80 Column Dimension and Properties

Description	Value	Unit	Note
Column	BC		
b	650	mm	
h	650	mm	
Longitudinal rebar	16	D	25
n of y rebar	5		number of rebar in y direction
n of x rebar	5		number of rebar in x direction
transversal rebar	10	mm	
L	4000	mm	
fc'	30	Mpa	
fy	400	Mpa	
cover	40	mm	
lo	650	mm	plastic hinge region/support
non lo	2600	mm	non plastic hinge region/span

2.12.1. Loading

The loadings of column are from dead load, live load, and earthquake load. These loads come from ETABS Output for upper column, designed column, and lower column. For moment calculation the moment is only from designed column at top edge column and bottom edge column. The shear load comes from maximum earthquake that happen in X or Y direction. The loading is presented in Table 2. 81. The load combination of column can be seen in Table 2. 82.

Table 2. 81 Column Loading

Loading	D	L	Qe
Axial Load (kN)			
Upper Col	1466.6924	1322.5082	1089.2903
Designed Col	1938.3739	1752.583	1469.1948
Lower Col	2411.3515	2183.9467	1804.4163
Moment (kNm)			
Top edge col	96.8718	95.9499	659.7767
Bottom edge col	96.8718	95.9499	659.7767
Shear (kN)	0	0	280.9338

Table 2. 82 Load Combination of Column

Load Combination				
Column	**Axial (kN)**	**Top Moment (kNm)**	**Bottom Moment (kNm)**	**Shear (kN)**
Upper column				
1.4D	2053.36936	-	-	-
1.2D+1.6L	3876.044	-	-	-
1.2D+1E+0.5L	3510.57528	-	-	-
0.9D+1E	2409.31346	-	-	-
Designed Column				
1.4D	2713.72346	135.62052	135.62052	0
1.2D+1.6L	5130.18148	269.766	269.766	0
1.2D+1E+0.5L	4671.53498	823.99781	823.99781	280.9338
1.2D-1E+0.5L	1733.14538	-495.55559	-495.55559	
0.9D+1E	3213.73131	746.96132	746.96132	280.9338
0.9D-1E	275.34171	-572.59208	-572.59208	
Lower column				
1.4D	3375.8921	-	-	-
1.2D+1.6L	6387.93652	-	-	-
1.2D+1E+0.5L	5790.01145	-	-	-
0.9D+1E	3974.63265	-	-	-

2.12.2. Dimension Check

The dimension of column must be checked to meet the requirements in SNI 2847:2019. The variables that must be considered is width-height ratio (b/h), steel area (Ast), column section area (Ag), axial force (Pu), concrete compressive strength (fc’). If the axial load that works on column is larger than 30% of column cross-section area times compressive strength, than the axial load must be considered and analyzed using SNI 2847:2019 Table 18.7.5.4. Since the maximum axial load in this column 5130.18 kN is larger than 0.3Agfc’ 3802.5 kN, so the axial load is considered.

Table 2. 83 Column Dimension Check

Check Dimension			
Description	**Value**	**Unit**	**Note**
Smallest dimension	650	mm	
Check if > 300 mm	OK		SNI 2847:2019 chapter 8.7.2.1a
b/h	1		
Check b/h>0.4	OK		SNI 2847:2019 chapter 8.7.2.1b
Ast	7853.981634	mm2	
Ag	422500	mm2	
0.01Ag	4225	mm2	
0.06Ag	25350	mm2	
Check 0.01Ag<Ast<0.06Ag	OK		SNI 2847:2019 chapter 18.7.4.1
Pu	5130.18148	kN	
0.3Agfc'	3802.5	kN	
Check Pu>0.3Agfc'	OK		SNI 2847:2019 table 18.7.5.4
ρg	0.018589306		16×π×0.25×25^2/Ag

2.12.3. Longitudinal Reinforcement

One of the requirements in SRPMK building for column in SNI 2847:2019 chapter 18.7.3.2 is that the column must be strong, and the beam must be weaker. This is to ensure that the resistance of building to withstand earthquake from strong to weak start from foundation->beam-column joint->column->beam. This condition achieves if $\sum Mnc \geq (1.2)\sum Mnb$. If this condition is achieved, the first possible failure is beam, which is less dangerous than the failure of column, beam-column joint, or foundation. The longitudinal reinforcement of column must be checked from the beam negative moment (Mnb_{left}), beam positive moment (Mnb_{right}), column top edge moment (Mnc_a), and column bottom edge moment (Mnc_b) as in Figure 2. 41 Moments act on Column. The calculation of bending moment strength is presented in Table 2. 84.

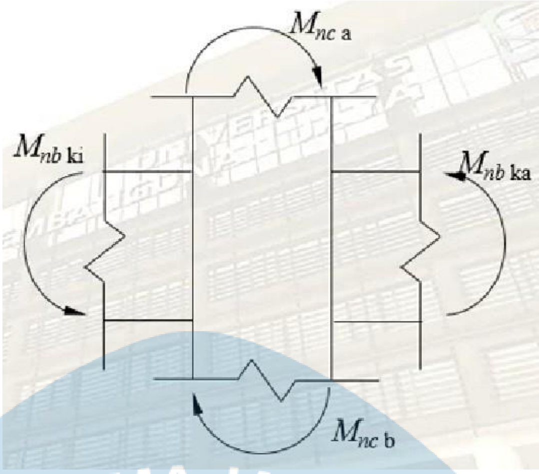


Figure 2. 41 Moments act on Column (Source: Setiawan. 2016)

Table 2. 84 Column Minimum Bending Strength

Check Minimum Bending Strength			
Description	**Value**	**Unit**	**Note**
fc'	30	Mpa	
fy	400	Mpa	
b	500	mm	
h	500	mm	
cover	40	mm	
db	25	mm	longitudinal rebar
dc	13	mm	confinement rebar
d for beam	434.5	mm	clear distance
top long. Rebar	4	D	25
As	1963.495408	mm2	
a	61.59985595	mm2	
Mnb_{left}	317.0652951	kNm	beam negative moment
bottom long. Rebar	3	D	25
As	1472.621556	mm2	
a	46.19989196	mm2	
Mnb_{right}	242.3346351	kNm	beam positive moment

The moment on column is obtained using SPColumn Software to check the P-M Interaction Diagram of column (see Figure 2. 42 Column P-M Interaction Diagram). The output of SPColumn is shown in Figure 2. 43.

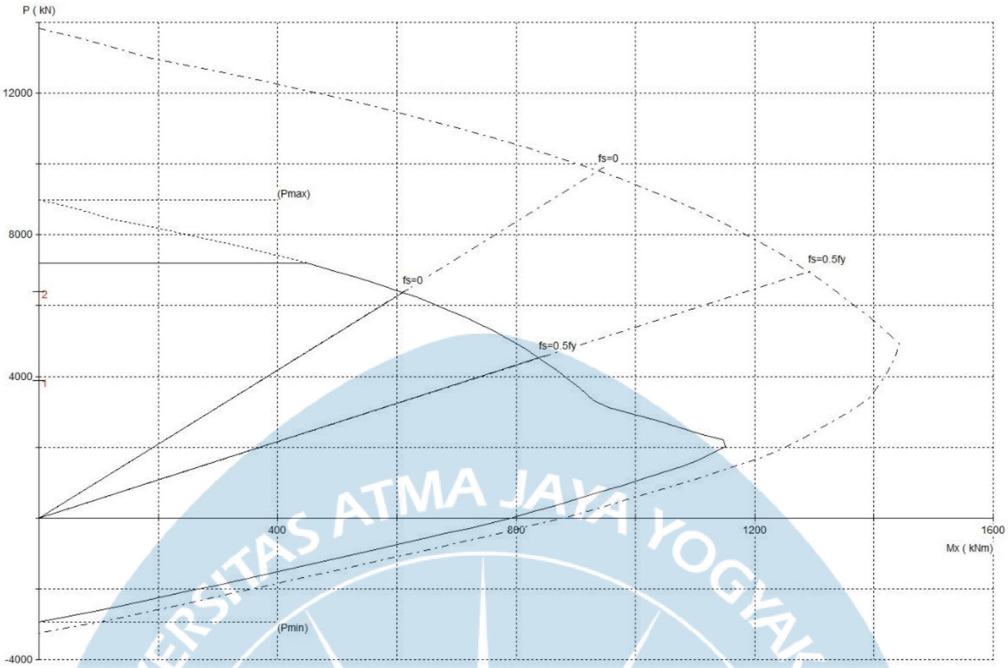


Figure 2. 42 Column P-M Interaction Diagram

Factored Loads and Moments with Corresponding Capacities

No	Pu	Mux	ϕMnx	ϕMn/Mu	NA Depth	dt Depth	εt	ϕ
	kN	kNm	kNm		mm	mm		
1	3876.04	0.00	890.40	999.999	396	588	0.00145	0.650
2	6387.94	0.00	607.13	999.999	589	588	-0.00001	0.650

Figure 2. 43 Factored Loads and Moments of Column

Table 2. 85 Column Top and Bottom Moment

Column			
Description	**Value**	**Unit**	**Note**
φ	0.65		
Pu upper col.	3876.044	kN	
φMnc_a	890.4	kNm	SPColumn Software P-M Diagram
Mnc_a	1369.85	kNm	
Φ	0.65		
Pu lower col.	6387.93652	kN	
φMnc_b	607.13	kNm	SPColumn Software P-M Diagram
Mnc_b	934.0461538	kNm	
Strong column-weak beam	OK Strong Column-Weak Beam	$(Mnc_a+Mnc_b) >= 1.2(Mnb_{left}+Mnb_{right})$?	SNI 2847:2019 chapter 18.7.3.2

From the calculation, it shows that the column and beam relationship is strong column-weak beam because ∑Mnc = 2303.89 kNm and (1.2)∑Mnb = 559.4 kNm.

2.12.4. Transversal Reinforcement

According to SNI 2846:2019 figure R18.7.5.2, all longitudinal rebar must be confined with cross tie reinforcement for SRPMK Column. The parameter of transversal reinforcement is presented in Table 2. 86 Transversal Reinforcement Parameters.

Table 2. 86 Transversal Reinforcement Parameters

Transversal Reinforcement			
Check Pu>0.3Agfc' is OK	All long. Rebar must be confined with cross tie 135° and 90°		SNI 2847:2019 figure R18.7.5.2 f
bc	570	mm	clear width of confinement
hc	570	mm	clear height of confinement
Ach	324900	mm2	Area of column without cover = bc×hc
Ag	422500	mm2	
xi at y	131.25	mm	Distance of x long. Rebar with confinement
xi at x	131.25	mm	Distance of y long. Rebar with confinement
hx	131.25	mm	
6db	150	mm	
hook length	150	mm	6db>=75 mm

Table 2. 87 Calculation of Confinement Steel Area

Ash/s.bc			
kf	1	>=1	fc'/175+0.6
kn	1.142857143		nl/(nl-2)
0.3(Ag/Ach-1)fc'/fyt	0.006759003		
0.09fc'/fyt	0.00675		
0.2kf×kn×Pu/(fytAch)	0.009022876		
Take Ash/s.bc	0.009022876		SNI 2847:2019 table 18.7.5.4

According to SNI 2847:2019 chapter 18.7.5.3 the maximum spacing must be decided from ¼ of smallest dimension of column, 6 times of longitudinal bar, and 150 mm. The spacing is chosen to be rounded down to 100 mm for safety reasons. Therefore, the required confinement steel area (Ash req) can be calculated as in Table 2. 88 Minimum Transversal Reinforcement Spacing. The confinement is chosen to have 5 legs and diameter of 13 mm with steel area (Ash use) bigger than required confinement steel area.

Table 2. 88 Minimum Transversal Reinforcement Spacing

Description	Value	Unit	Note
1/4 of smallest dimension	162.5	mm	
6db	150	mm	
s0	150	mm	SNI 2847:2019 eq. 18.7.5.3
maximum spacing	150	mm	
Take spacing	100	mm	
Ash/s	5.143039078	mm2/m	bc×Ash/s.bc
Ash required	514.3039078	mm	
v legs	5		
dc new	13	mm	
Ash use	530.9291585	mm2/m	Confinement steel area
Check Ash	OK		

The shear strength for SRPMK Column must be checked when the yield stress increase to 1.25fy. The new probability column moment ($Mprc_{top}$ and $Mprc_{bot}$) is calculated from P-M Interaction Diagram when fy increase 25% and maximum moment at the edge of diagram (see Figure 2. 44 New P-M Interaction Diagram when 1.25Fy). The calculation for transversal reinforcement at support is shown in Table 2. 89 Transversal Reinforcement Support.

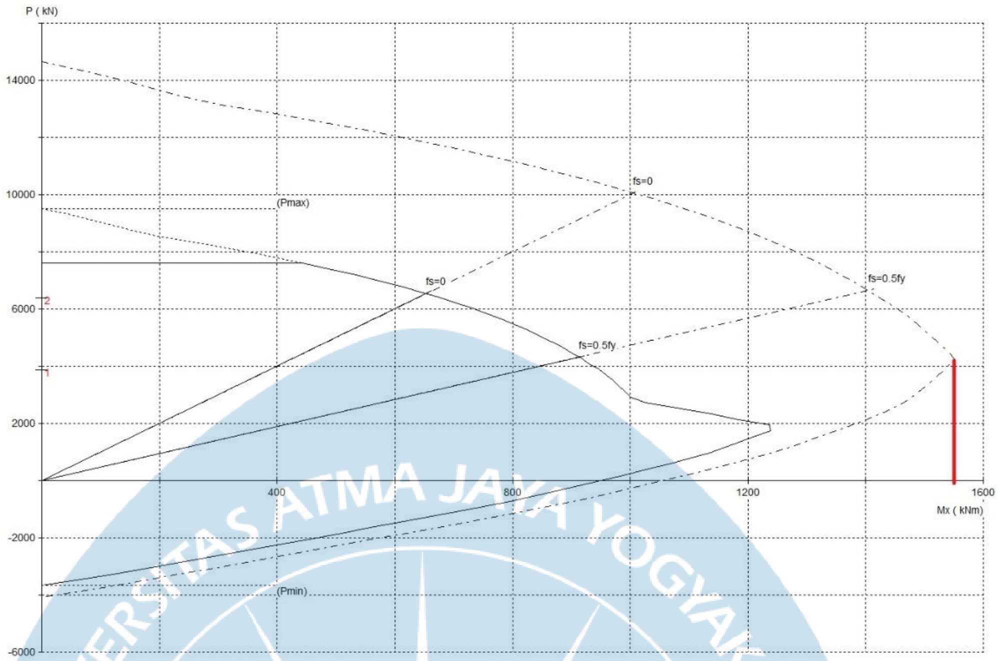


Figure 2. 44 New P-M Interaction Diagram when 1.25Fy

Table 2. 89 Transversal Reinforcement Support

Transversal Reinforcement Support			
Description	**Value**	**Unit**	**Note**
1.25fy	500	Mpa	
φ	1		
Mprc_top	1555	kNm	SPColumn Software P-M Diagram (Maximum Moment when 1.25fy)
Mprc_bot	1555	kNm	SPColumn Software P-M Diagram (Maximum Moment when 1.25fy)
lu	4	m	Column length
Ve	777.5	kN	(Mprc_top+Mprc_bot)/lu SNI 2847:2019 chapter 18.7.6.1.1.1
DFTop	0.5		Distribution factor = 0.5 if top and bottom section shape is the same
Dfbot	0.5		
Mprb_top	388.7722	kN	
Mprb_bot	298.6661	kN	
Max Ve	171.8596	kN	SNI 2847:2019 chapter 18.7.6.1.1.2
Max Ve must be bigger than	280.9338	kN	SNI 2847:2019 chapter 18.7.6.1.1.3
Ve use	280.9338	kN	
φ	0.75		
Vs	374.5784	kN	Assume Vc = 0
d for column	587.5	mm	new clear distance
Av/s	1.5940	mm	Vs/(fyt.d)
Av	159.3951	mm2	

Transversal Reinforcement Support			
Description	**Value**	**Unit**	**Note**
Check confinement Ash	OK Ash>Av		

Transversal support at span must check the shear strength of concrete first using SNI 2847:2019 eq. 22.5.6.1. Since concrete strength (Vc) is bigger than ultimate shear strength (Vu) the spacing can use half of clear distance (d). However, in SNI 2847:2019 chapter 18.7.5.5 the maximum spacing for confinement in span is 150 mm. The calculation of transversal reinforcement at span is presented in Table 2. 90 Transversal Reinforcement Span

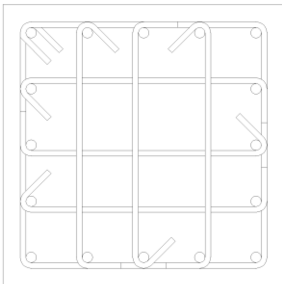
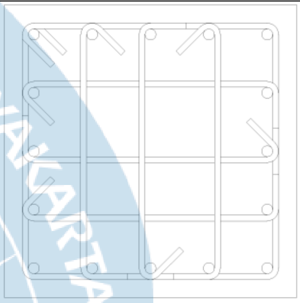
Table 2. 90 Transversal Reinforcement Span

Transversal Reinforcement Span			
lo	1200	mm	
Nu	275.3417	kN	smallest axial load on column
Ag	422500	mm2	
λ	1		
Vc	372.12654	kN	$Vc = 0.17\left(1 + \frac{Nu}{14Ag}\right)\lambda\sqrt{fc'}b_w d$ SNI 2847:2019 eq. 22.5.6.1
Check Vc>Vu	OK		
6db	150	mm2	
s=d/2	293.75	mm	
	250	mm	
Confinement s span	150	mm	SNI 2847:2019 chapter 18.7.5.5

2.12.5. Conclusion

The column will be use 650x650 mm2 with fc' 30 MPa and fy 400 MPa. The recap for reinforcement is presented in Table 2. 91 Recap for Column Reinforcement The hook will use cross tie where one part is bent 90^o and the other bent 135^o.

Table 2. 91 Recap for Column Reinforcement

Recap		
Longitudinal Reinforcement	16D25	
First 50mm confinement	2.5D13-50	
Support Confinement	2.5D13-100	
Span Confinement	2.5D13-150	
Transversal Hook Length		
Hook Type	Cross Tie	135° and 90°
l_{ext}	78	mm

2.13. Beam-Column Joint Design

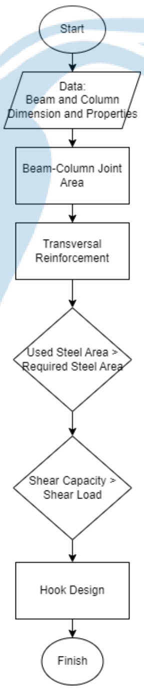


Figure 2. 45 Beam-Column Joint Design Flowchart

In SRPMK building, beam-column joint must be designed to have transversal reinforcement and development length of longitudinal rebar. The properties of beam column that have been decided in previous preliminary design is presented in Table 2. 92 Beam-Column Properties. Beam-column joint effective area is shown in Figure 2. 46 and Table 2. 93. The procedure of beam-column joint design is presented in Figure 2. 45.

Table 2. 92 Beam-Column Properties

Beam-Column Properties		
bb	500	mm
hb	500	mm
Lb	5500	mm
bc	650	mm
hc	650	mm
Lc	4000	mm
fc'	30	Mpa
fy	400	Mpa
λ	1	

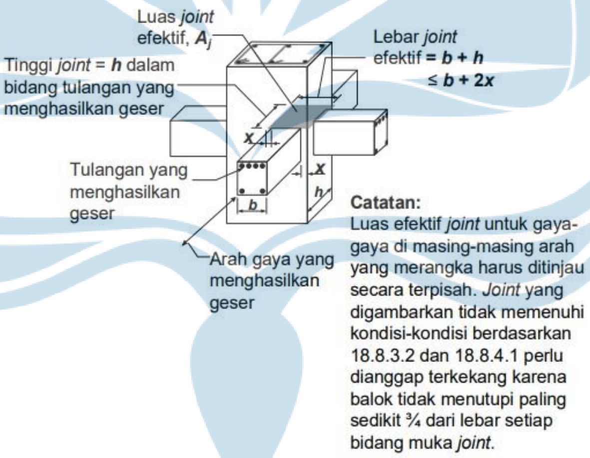


Figure 2. 46 Beam-Column Joint Effective Dimension

Table 2. 93 Beam-Column Joint Effective Dimension

Beam-Column Joint			
b	500	mm	SNI 2847:2019 chapter18.8.4.3
x	75	mm	
hj	650	mm	
b+h	1150	mm	
b+2x	650	mm	
bj	650	mm	
Aj	422500	mm2	SNI 2847:2019 figure 18.8.4

The beam is enough to restraint column if the beam width is larger than ¾ of column width. The confinement uses 3 legs so the steel area (Av) larger than required steel area (Ash).

2.13.1. Reinforcement

The reinforcement for beam-column joint is mainly designed for confinement, while the longitudinal reinforcement just the extension from column longitudinal reinforcement. However, both of transversal and longitudinal reinforcement must be checked. The calculation of reinforcement strength is presented in Table 2. 94.

Table 2. 94 Check Transversal Requirement

Check Transversal Requirement			
Description	**Value**	**Unit**	**Note**
lo	650	mm	SNI 2847:2019 figure 18.9.2.2
Confined by … beams	4	beams	
bb>=3/4bc	OK		
Ash/s	2.5715	mm2/mm	SNI 2847:2019 chapter 18.8.3.2 at plastic hinge
s use	100	mm	SNI 2847:2019 chapter 18.8.3.2
Ash required	257.1519	mm2	
n legs	3		
dc	13	mm	
Av	398.1969	mm2	
Check Av>Ash	OK		

The shear force of beam-column joint (V_{sway}) is the product of probability beam moment and column moment.

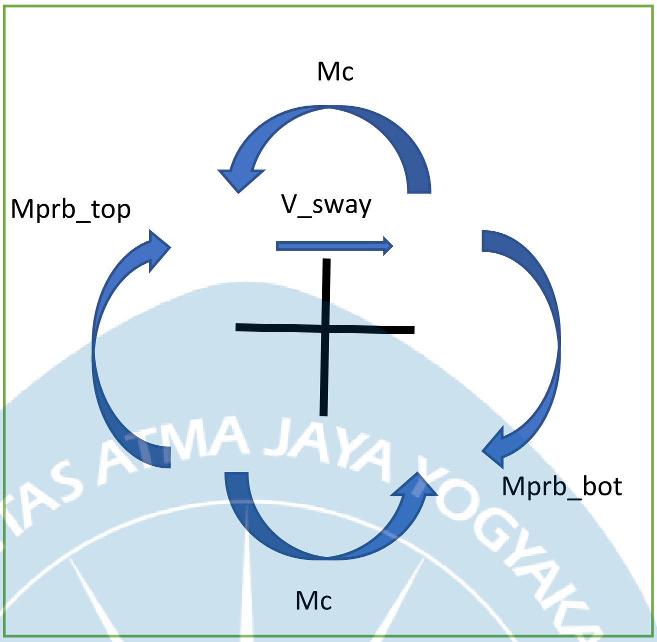


Figure 2. 47 Shear Force on Column due to Probability Moment of Beam and Column

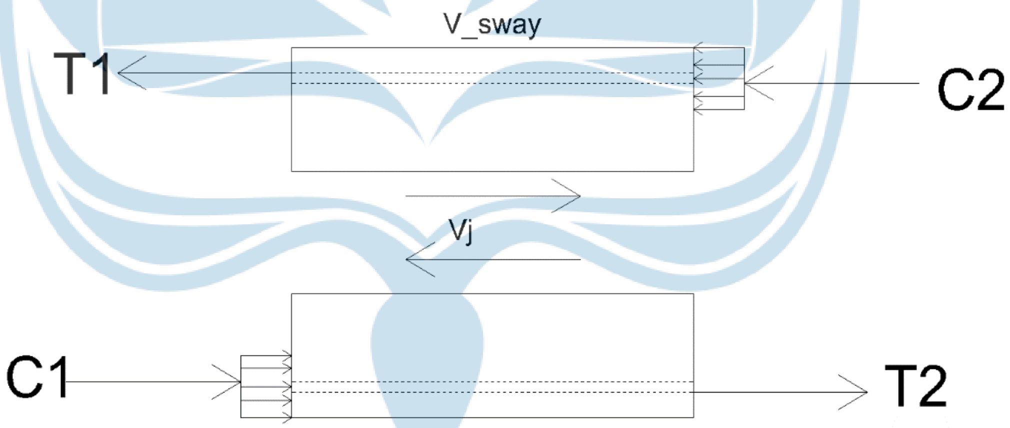


Figure 2. 48 Tension (T) and Compression on Beam-Column Joint

Table 2. 95 Shear Strength Check

Check Shear Strength			
$Mprb_{top}$	388.7722	kNm	
$Mprb_{bot}$	298.6661	kNm	
DF	0.5		Distribution Factor
Mc	343.7191	kNm	
V_{sway}	171.8596	kN	
fy	400	Mpa	
Top longitudinal rebar	4D25		

Check Shear Strength			
As	1963.4954	mm2	
T1	981.7477	kN	
C1	981.7477	kN	
Bottom longitudinal rebar	3D25		
As	1472.62156	mm2	
T2	736.310778	kN	
C2	736.310778	kN	
Vj	1546.19891	kN	max(T+C-Vsway)
Vn	3934.01727	kN	
φ	0.85		
φVn	3343.91468	kN	
Check φVn>Vj	OK		

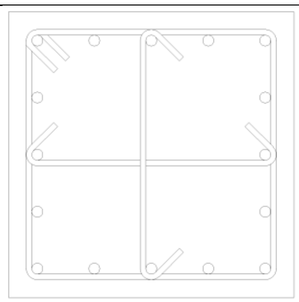
Table 2. 96 Hook Design

Hook Design			
Type	Standard 90°		SNI 2847:2019 chapter 18.8.5.1
For long. Rebar d10-d36			
db	25	mm	
8db	200	mm	
150mm	150	mm	
fy×db/(5.4×λ×√(fc'))	338.100344	mm	SNI 2847:209 eq. 18.8.5.1
ldh	338.100344	mm	
Take Development Length	340	mm	
12db	300	mm	

2.13.2. Conclusion

The confinement at beam-column joint, development length of longitudinal rebar, and hook length is presented in Table 2. 97 and Table 2. 98.

Table 2. 97 Reinforcement Recap on Beam-Column Joint

Recap		
Longitudinal	16D25	
Confinement	1.5D13-100	
Development Length, l_{dh}	340	mm

Recap		
Hook Length	300	mm

Table 2. 98 Transversal Hook Length

Transversal Hook Length		
Hook Type	135	o
l_{ext}	150	mm

According to SNI 2847:2019 figure R18.7.5.2 the spacing between each longitudinal bar in beam-column joint must be less than 350 mm. This spacing must be checked from drawing and presented in Table 2. 99.

Table 2. 99 Check Rebar Spacing

Check xi_y and xi_x		Note
xi_y	259.5	From Drawing
xi_x	259.5	From Drawing
Check xi_y	OK	
Check xi_x	OK	

2.14. Slab Design

This design is based on PBI'71 Slab Moment Table Method and SNI 2847:2019. The dimension of slab must be determined either as one way slab and two-way slab. One way slab is a slab that has a long span larger than two times of short span. Two way slab is a slab that has a long span shorter than two times the length of short span. Reinforcement in one way slab will be only placed in one direction, while reinforcement in two-way slab will be placed in two directions. The design of slab can be done as follows:

2.14.1. Determination of Dimension and Slab Type

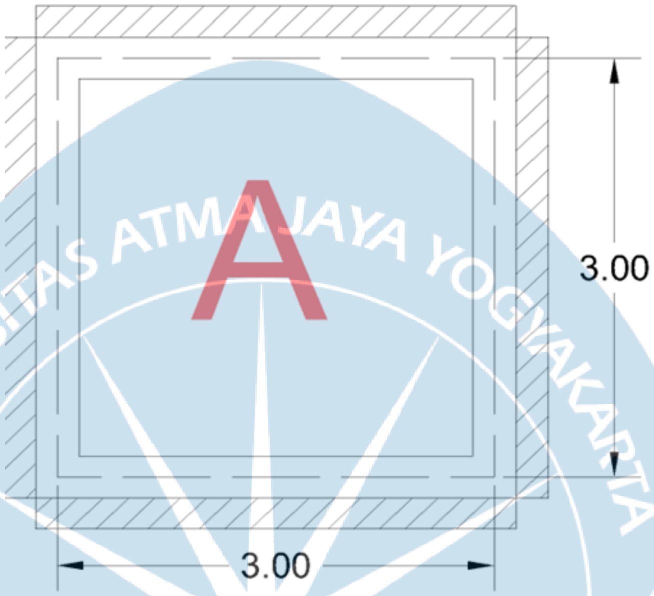


Figure 2. 49 Slab Type A

The slab will be checked for one way or two ways. Since the long span (Ly) is 3m and the short span (Lx) is also 3 m, the ratio of long span to the short span is one and the type is two-way slab. This slab is interior and is confined by beams on all four sides. The thickness of the slab is 150 mm. The concrete cover of slab according to SNI 2847:2019 table 20.6.1.3.1 is 20 mm. The beam is 500 x 500 mm2. Figure 2. 50 shows interior slab and beam cross section. The slab will use fc' 30 MPa and steel fy 400 MPa. The preliminary design for reinforcement diameter for short span (Dlx) is 10 mm, long span (Dly) is 13 mm, short support (Dtx) is 13 mm, and long support (Dty) is 13 mm. The load act on slab is presented in Table 2. 100 Slab Load

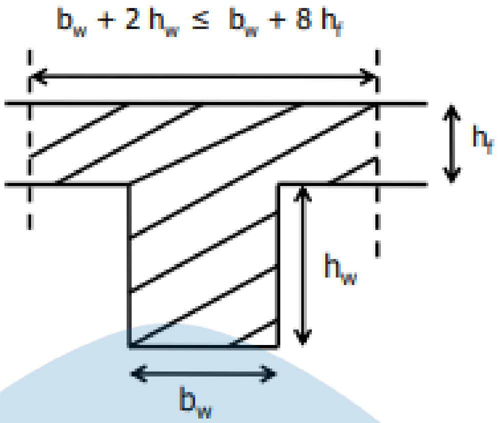


Figure 2. 50 Interior Slab-Beam Cross-section

Table 2. 100 Slab Load

Load		
Live Load	4.79	kN
Self-Weight	3.6	kN/m2
Reinforced	2.64	
Ceramic	0.2	kN/m2
Ceiling	0.18	kN/m2
Total DL	6.62	kN/m2
Comb Load 1.2D + 1.6L	15.608	kN/m2

2.14.2. Shear Strength

The shear force (Vu) that acts on the slab is $\frac{1.15 q_u l_n}{2} = 22.4365\ kN$, the clear distance from top slab to the middle of steel reinforcement in short span (dx) is $h - cc - \frac{D8}{2} = 124\ mm$, and the clear distance of long span steel reinforcement (dy) is $h - cc - D8 - \frac{D8}{2} = 112\ mm$. The shear strength of slab is $\phi 0.17 \lambda \sqrt{fc'} b_w d_x = 43.2975\ kN$ with $\phi = 0.75$ for shear. Because the shear strength is larger than the shear force, it is safe. The Figure 2. 51 Clear Distance Dx and Dy on Slab shows dx and dy.

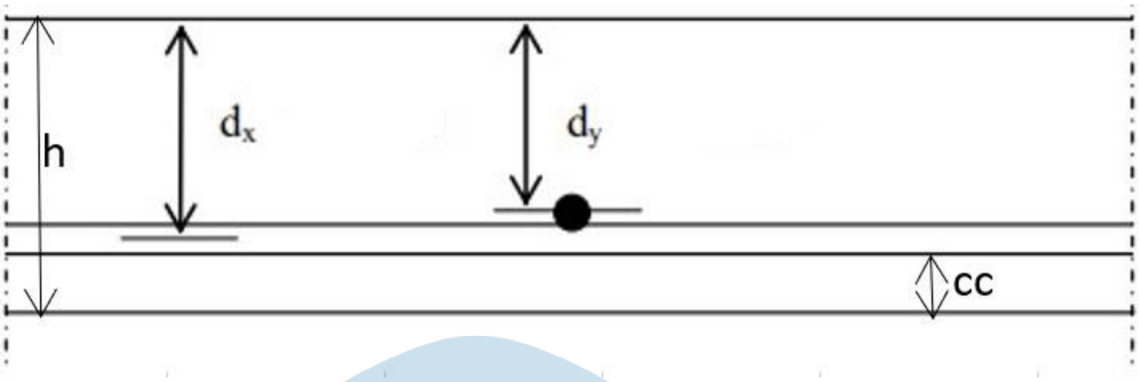


Figure 2. 51 Clear Distance Dx and Dy on Slab

2.14.3. Moment Strength

The moment load in the slab can be calculated by multiplying the moment load with the coefficient in PBI 1971 table 13.3.1. The coefficient of short span moment (mlx) is 21, long span moment (mly) is 21, short support moment (mtx) is 52, and long support moment (mty) is 52. The moment acts on slab is $M = \pm 0.01 q_u l^2 = 0.1405$ kNm. This moment will be multiplied by the coefficient moment previously.

To calculate the main reinforcement, the coefficient of flexural resistance (k) must be calculated first. The general eq. for k is $k = \frac{M_n}{bd^2} = \frac{Mu}{\phi bd^2}$, with Mu is mlx, mly, mtx, or mty and d is dx or dy. The value of k_{lx}, k_{ly}, k_{tx}, and k_{ty} is 0.4263, 0.5226, 1.0557, and 1.294, respectively.

The steel reinforcement is divided for span location (l) and support location (t). The length of steel in span location (bl) is $\frac{1}{2}b = 1500$ mm and for support location (bt) is $\frac{1}{4}b = 750$ mm. The ratio of steel (ρ) is $\frac{0.85fc'}{fy}\left(1 - \sqrt{1 - \frac{2k}{0.85fc'}}\right)$, with k is klx, kly, ktx, or kty. The value of $\rho_{lx}, \rho_{ly}, \rho_{tx}$, and ρ_{ty} is 0.001075, 0.00132, 0.002696, and 0.003322, respectively. The maximum steel ratio (ρmax) is $\frac{0.36fc'\beta1}{fy}$ with β1 value is from SNI 2847:2019 table 22.2.2.4.3 and the value of β1 is 0.8357. So, the maximum steel ratio is 0.003322. This is still bigger than the biggest steel ratio that is required. The area of steel required (As-req) is ρbd with b = 1000 mm and d is dx or dy. The steel ratio for As-reqlx, As-reqly, As-reqtx, and As-reqty is 133.2882 mm2, 147.8559 mm2, 334.3347 mm2, and 372.0204 mm2, respectively. Because SNI 2847:2019 table 8.4.1.1 provides the minimum steel area (Asmin), therefore the used steel area (As-use) cannot be smaller than the Asmin. The Asmin for fy<420 MPa is 0.2%Ag which is equal to 300 mm2.

Therefore, the As-uselx, As-usely, As-usetx, and As-usety is 300 mm2, 300 mm2, 334.3347 mm2, and 300 mm2, respectively.

In order to calculate the moment strength of steel, the value $a = \frac{Asfy}{0.85fc'fy}$ must be calculated and the moment strength can be calculated from $Asfy\left(d - \frac{a}{2}\right)$. The value of alx, aly, atx, and aty is 9.4118, 9.4118, 10.4889, and 11.6712, respectively. The factored moment strength (ϕMn) of $\phi Mnlx, \phi Mnly, \phi Mntx,$ and $\phi Mnty$ is 12.8838 kNm, 11.5878 kNm, 14.2183 kNm, and 14.2935 kNm, respectively with $\phi = 0.9$. This is still bigger than moment load mlx, mly, mtx, and mty which is 2.9499 kNm, 2.9499 kNm, 7.3045 kNm, and 7.3045 kNm, respectively.

2.14.4. Reinforcement

The spacing for reinforcement is calculated from $\frac{0.25\pi D^2 b}{As_{req}}$. The spacing for reinforcement in short span (slx), long span (sly), short support (stx), and long support (sty) is 261. 8 mm, 221.2 mm, 198.5 mm, and 178.4 mm. This spacing must be rounded down to make the steel arrangement easier to 50 mm down. From SNI 2847:2019 chapter 8.7.2.3 the maximum spacing is 3 times slab thickness (3h) and 450 mm. So, the used reinforcement for Dlx, Dly, Dtx, and Dty is D10-250, D13-200, D13-150, and D13-150, respectively.

2.14.5. Conclusion

The slab type A of 3000 x 3000 mm will use fc'30 MPa and fy 400 MPa with reinforcement as in Table 2. 101 Slab Reinforcement

Table 2. 101 Slab Reinforcement

Slab Reinforcement	
Dlx	D10-250
Dly	D13200-
Dtx	D13-150
Dty	D13-150

2.15. Stair Design

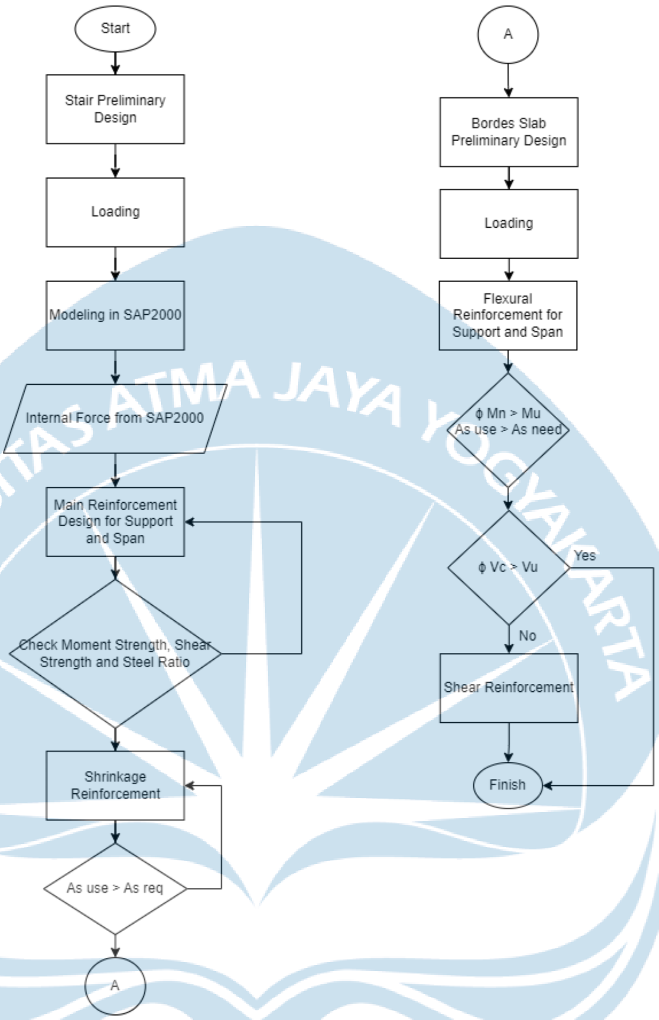


Figure 2. 52 Stair Design Flowchart

The stairs are typical for every floor in the building. In stair design, the design copes for slab (*landing slab*) and stair. The stair and landing slab slab will use reinforced concrete. The procedure of stair and landing slab slab design is shown in Figure 2. 52.

2.15.1. Stair Data

The figure of stair area and stair detail can be seen in Figure 2. 53 and Figure 2. 54.

- Height of each floor (Hlt) = 4 m
- Stair width = 3.5 m
- Landing slab width = 1.7 m
- Optrede (Op) = 0.2 m

- Antrede (An) = 0.25 m
- Number of stair (Ntg) = 20 stairs
- Stair length (Ltg) = 2.25 m
- Stair slope (α) $= \tan^{-1}\left(\frac{0.2}{0.25}\right) = 38.66°$
- Thickness of the stair = 0.15 m
- Thickness of landing slab = 0.3 m
- Stair concrete cover = 30 mm
- Landing slab concrete cover = 40 mm
- Fc' = 25 MPa
- Fy = 370 MPa
- Equivalent thickness of the stair (tt) $= \frac{0.5\times0.2\times0.25}{\sqrt{0.2^2+0.5^2}} = 0.078087\ m = 78.087\ mm$

Total equivalent of the stair (t') $= \frac{0.15+0.078087}{cos\alpha} = 0.292094$ m = 292.094 mm

- Weight of concrete = 24 kN/m^3
- Weight of tile = 21 kN/m^3

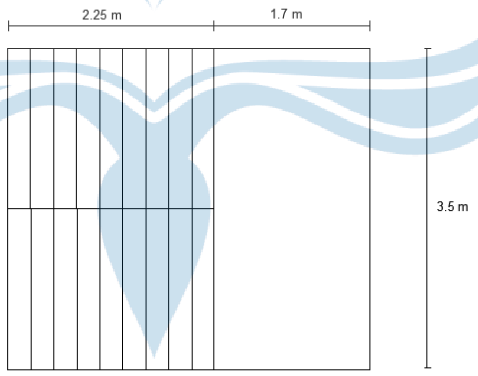


Figure 2. 53 Stair Area

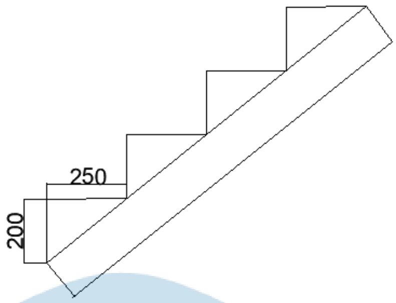


Figure 2. 54 Stair Detail

2.15.2. Loading

The load of stairs and landing slab slab are from self-weight, tiles, railing, and live load. The load can be seen in Table 2. 102 and Table 2. 103.

Table 2. 102 Stair Loading

Stair Loading (qtg)		
-Slab Weight	$\frac{292.094}{1000} \times 24 = 7.01$	kN/m
-Tiles and Plaster	$0.05 \times 21 = 1.05$	kN/m
-Railing Weight	1	kN/m
Total Dead Load (qd)	9.06	kN/m
Live Load (qL)	4.79	kN/m

Table 2. 103 Landing slab Loading

Landing slab Loading (qbd)		
-Slab Weight	$0.15 \times 24 = 3.6$	kN/m
-Tiles and Plaster	$0.05 \times 21 = 1.05$	kN/m
-Railing Weight	1	kN/m
Total Dead Load (qd)	5.65	kN/m
Live Load (qL)	4.79	kN/m

2.15.3. Reinforcement

The internal force of stairs is obtained by the help of SAP2000 Software. The maximum moment due to dead load is 14.62 kNm, maximum moment due to live load is 10.92 kNm, shear force due to dead load is 13.32 kN, and shear force due to live load is 9.02 kN. The maximum combination uses 1.2D + 1.6L, which for Mur = 35.468 kNm and Vur = 30.416 kN. The maximum moment is taken as 0.5Mur so Mu = 17.508 kNm for support location. Meanwhile for span location the maximum momen is taken as 0.8Mur so Mu = 28.01 kNm.

Calculation for reinforcement must begin from the calculation of nominal strength (Rn) which is $\frac{Mu}{0.9bd^2}$ with b = 1000 mm and d is stair thickness – concrete cover – 1/2 steel diameter. The minimum steel ratio (ρmin) is 0.0018, the maximum steel ratio (ρmax) is $0.75(\frac{0.85fc'\beta}{370})(\frac{600}{600+fy})$, and the required steel ratio (ρneed) is $\frac{0.85fc'}{fy}(1-\sqrt{1-\frac{2Rn}{0.85fc'}})$. The required steel area (Asneed) is ρbd. The required spacing is $\frac{0.25\pi^2 d^2 b}{As}$. The spacing need to be rounded down to 50 mm for easier arrangement. Last the Asuse $=\frac{0.25\pi d^2 b}{s}$ must be larger than Asneed.

The concrete shear strength (Vc) must be checked from shear force (Vu) with $\phi Vc = \phi\frac{1}{6}\sqrt{(fc')}bd$ with ϕ = 0.75. Last, the moment strength (Mn) must be checked and must be bigger than Mu. Factored nominal moment can be calculated with $\phi Mn = \phi Asfy(d-\frac{a}{2})$, where $a=\frac{Asfy}{0.85fc'fy}$, $c=\frac{a}{\beta 1}$, and $\epsilon_t = \frac{0.003(d-c)}{c}$. Because in landing slab span the concrete shear strength (Vc) is smaller than ultimate shear force (Vu), than steel shear strength must be calculated as $\frac{Vu}{\phi} - Vc$. The calculation for reinforcement is presented in Table 2. 104 Stairs and Landing slab Reinforcement Calculation

Table 2. 104 Stairs and Landing slab Reinforcement Calculation

Data	**Unit**	**Support**		**Span**		**Landing slab**	
		Main	**Shrinkage**	**Main**	**Shrinkage**	**Support**	**Span**
Mu	kNm	17.508	17.508	28.013	28.013	44.8617	30.8424
Dst	mm	13	8	13	8	13	13
d	mm	113.5	113.5	113.5	113.5	245.5	245.5
		0.8357	0.8357	0.8357	0.8357	0.8357	0.8357
Rn	kN/m2	1.51	1.51	2.41	2.41	0.785	0.5396
min		0.0018	0.0018	0.0018	0.0018	0.0018	0.0018
need		0.00424	0.002	0.00695	0.002	0.00216	0.00148
max		0.02265	0.02265	0.02265	0.02265	0.02265	0.02265
use		0.00424	0.002	0.00695	0.002	0.00216	0.0018
As need	mm2	480.9741	300	1042.607	300	544.8488	453.6
Spacing	mm	275.9656	167.619	127.308	167.619	369.0166	443.2582
S use	mm	250	150	100	150	350	400
As use	mm2	530.9292	335.238	1327.3229	335.238	574.4627	502.6548
		As us>As need	As us>As need	As us>As need	As us>As need	As us>As need	As us>As need

Data	Unit	Support		Span		Landing slab	
		Main	**Shrinkage**	**Main**	**Shrinkage**	**Support**	**Span**
a	mm	9.2444	-	23.1110	-	33.3414	8.7521
c		11.0619	-	27.6547	-	39.2250	10.2966
		0.0278	-	0.0093	-	0.0163	0.0704
		0.9000	-	0.9000	-	0.9000	0.9000
Mn	kNm	19.2495	-	45.0593	-	45.0176	47.3695
Mu	kNm	17.508	-	28.013	-	-	30.8424
		Mn>Mu		Mn>Mu	-	-	Mn>Mu
Vc	kN	94.5833	-	94.5833	-	-	52.5
Vc	kN	70.9375	-	70.9375	-	-	39.375
Vu	kN	30.416	-	30.416	-	-	41.65
		Vc>Vu	-	Vc>Vu	-	-	Vc<Vu
Vs	kN	-	-	-	-	-	3.0333
dc	mm	-	-	-	-	-	8
Av	mm2	-	-	-	-	-	50.2857
spacing	mm	-	-	-	-	-	100.263
S use	mm	-	-	-	-	-	100

2.15.4. Conclusion

The stairs and landing slab slab will use reinforced concrete with fc' 25 MPa and fy 370 MPa. The dimension of stairs optrede is 0.2 m, antrede is 0.25 m, stair width is 3.5 m, stair length is 2.25 m, stair slope is 38.66^o, the total number of stairs is 20, and the thickness of stair is 0.15 meter. The dimension of landing slab width is 1.7 m and thickness is 300 mm. The recap of reinforcement is presented in Table 2. 105 Stairs and Landing slab Reinforcement Recap

Table 2. 105 Stairs and Landing slab Reinforcement Recap

Recap		
Stairs Support	Main	D13-250
	Shrinkage	D8-150
Stairs Span	Main	D13-100
	Shrinkage	D8-150
Landing slab	Support	D16-350
	Span	D8-100