CHAPTER II

UPPER STRUCTURE

2.1 Introduction

The top structure is a structure that is above the ground consisting of roof plans, staircase plans, floor slab plans, beam and column plans. The top structure design usesreinforced concrete structures for beams, columns, stairs and slabs, while the roof usessteel structures.

The top structure planning begins with planning the roof structure with the by entering live load, dead load, and wind load according to the combination. Followedby beam and column planning using ETABS. By determining the size of the beams, columns and plates that are owned according to the conditions. The design using ETABS includes spectrum design data according to each area.

In the top structure planning, there are several references used, such as:

- SNI 2847:2019 concerning Structural Concrete Requirements for Buildings and Explanations
- SNI 1729:2020 concerning Specifications for Structural Steel Buildings
- SNI 1727:2020 concerning Minimum Design Load and Related Criteria forBuildings and Other Structures
- SNI 1726:2019 concerning Procedures for Planning Earthquake Resistancefor Building and Non-building Structures
- PBI (Indonesian Reinforced Concrete Regulation) 1971

2.2 Loaded

In top structure, it must be determined in advance the loading that occurs. There are 4 loads that occur, namely dead loads, live loads, earthquake loads and wind loads. Dead loads are loads that come from the self-weight of the structure and are permanent loads. Live load is a load that is dynamic or can change, for example humans and other objects in the building. Earthquake loads are loads that occur on structures that occur due to ground movements due to earthquakes.

2.2.1 Dead Load

Dead loads are loads that come from the own-weight of each structural element in the form of floor slabs, beams, columns, etc. which are part of the mainstructure. In structural modeling using software, the self-weight of the structure will be calculated automatically by the software based on the data on the density of the material and the dimensions of the structural elements entered in the software.

2.2.2 Live Load

Live load is the load that occurs as a result of the use of the building structure. The living burden can come from people/goods that can move from place to place. The church is included in the category of public space accordingto SNI 1727:2013 so that the living expenses are determined as follows : Live load = 4.79 kN/m2

2.2.3 Wind Load

In planning the wind load, it is necessary to know the location of the building, the function of the building, the number of floors, the height and width of the building. The building which functions as a church is located in Muntilan, Central Java. The building consists of 2 buldings with 4 floors in each building. The building height is 15 m and the building width is 60 m.

Wind loading refers to SNI 1727:2020 concerning Minimum Design Loads and Related Criteria for Buildings and Other Structures. To determine the loading, there are several steps that need to be taken in accordance with SNI 1727:2020.

The determination of the risk category can be seen in SNI 1727:2020. Buildings that function as houses of worship are included in important facilities so that they are included in risk category IV. The basic wind speed (V) for risk category IV according to the Indonesian Wind Map Book is 43.4 m/s.

2.2.4 Earthquake Load

The earthquake load starts from determining the class of the soil site first.Site classification can be searched using soil data in the form of SPT data. According to SNI 1726:2019, site classification is a seismic design classificationin the form of an amplification factor determined based on the soil profile. The soil profile is reviewed to a depth of 30 m from the land surface. Site class classification for SPT data can be performed by finding the average field standard penetration resistance ($\bar{N}$) using the formula:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

In determining the earthquake load, it is necessary to know the location of the building, the class of the site based on soil data, and the function of the building. Based on existing soil data, the class of the soil site is class SE or clay soil. This building functioned as a church. Based on table 3 of SNI 1726:2019, houses of worship are categorized as risk category IV.

Table 2. 1 Building and non-building risk categories

Jenis Pemanfaatan	Kategori Risiko
Gedung dan non gedung yang memiliki risiko rendah terhadap jiwa manusia pada saat terjadi kegagalan, termasuk, tapi tidak dibatasi untuk, antara lain: ▪ Fasilitas pertanian, perkebunan, perternakan, dan perikanan ▪ Fasilitas sementara ▪ Gudang penyiMPanan Rumah jaga dan struktur kecil lainnya	I
Semua gedung dan struktur lain, kecuali yang termasuk dalam kategori risiko I,III,IV, termasuk, tapi tidak dibatasi untuk: ▪ Perumahan ▪ Rumah toko dan rumah kantor ▪ Pasar ▪ Gedung perkantoran ▪ Gedung apartemen/ rumah susun ▪ Pusat perbelanjaan/ mall ▪ Bangunan industri	II
Gedung dan non gedung yang memiliki risiko tinggi terhadap jiwa manusia pada saat terjadi kegagalan, termasuk, tapi tidak dibatasi untuk: ▪ Bioskop ▪ Gedung pertemuan ▪ Stadion ▪ Fasilitas kesehatan yang tidak memiliki unit bedah dan unit gawat darurat ▪ Fasilitas penitipan anak ▪ Penjara ▪ Bangunan untuk orang jompo Gedung dan non gedung, tidak termasuk kedalam kategori risiko IV, yang memiliki potensi untuk menyebabkan daMPak ekonomi yang besar dan/atau gangguan massal terhadap kehidupan masyarakat sehari-hari bila terjadi kegagalan, termasuk, tapi tidak dibatasi untuk: ▪ Pusat pembangkit listrik biasa ▪ Fasilitas penanganan limbah	III

▪ Pusat telekomunikasi Gedung dan non gedung yang tidak termasuk dalam kategori risiko IV, (termasuk, tetapi tidak dibatasi untuk fasilitas manufaktur, proses, penanganan, penyimpanan, penggunaan atau tempat pembuangan bahan bakar berbahaya, bahan kimia berbahaya, limbah berbahaya, atau bahan yang mudah meledak) yang mengandung bahan beracun atau peledak di mana jumlah kandungan bahannya melebihi nilai batas yang disyaratkan oleh instansi yang berwenang dan cukup menimbulkan bahaya bagi masyarakat jika terjadi kebocoran.	
Gedung dan non gedung yang ditunjukkan sebagai fasilitas yang penting, termasuk, tetapi tidak dibatasi untuk: ▪ Bangunan-bangunan monumental ▪ Gedung sekolah dan fasilitas pendidikan ▪ Rumah sakit dan fasilitas kesehatan lainnya yang memiliki fasilitas bedah dan unit gawat darurat ▪ Fasilitas pemadam kebakaran, ambulans, dan kantor polisi, serta ▪ Fasilitas pemadam kebakaran, ambulans, dan kantor polisi, serta garasi kendaraan darurat	IV

Determination of site coefficients and parameters of the maximum earthquake acceleration spectral response considered risk-targeted (MCER), namely in the form of short period mapped earthquake acceleration spectral response parameters (Ss) and 1 second period mapped earthquake acceleration spectral response parameters can be seenon the official website of The Spektra Indonesia design from the Ministry of Public Worksand Public Housing found that Ss = 0.8863 g and S1 = 0.4205 g. This parameter has a different value depending on where the building is located.

Then, the amplification factor for short period vibration (Fa) can be determined using Table 6 of SNI 1726:2019, with the SE site class it is found that Fa = 1.191. The amplification factor representing the 1 second period (Fv) is determined using Table 7 sothat Fv = 2.359 is obtained.

Table 2.2 Site coefficient, Fa

Site Class	MCE$_R$ tearthquake acceleration spectral response parameters mapped In short periods, T = 0,2 second (S_s)				
	$S_s \le 0,25$	$S_s = 0,5$	$S_s = 0,75$	$S_s = 1,0$	$S_s \ge 1,25$
SA	0,8	0,8	0,8	0,8	0,8
SB	1,0	1,0	1,0	1,0	1,0
SC	1,2	1,2	1,1	1,0	1,0
SD	1,6	1,4	1,2	1,1	1,0
SE	2,5	1,7	1,2	0,9	0,9
SF	SS[b]				

Source: SNI 1726:2012

Notes:

(a) For values between Ss, linear interpolation can be performed

(b) SS = site that requires a specific geotechnical investigation and sitespecific response analysis, see Section 6.10.1

By knowing the amplification factor and the spectral response parameters for theearthquake acceleration, we can look for the parameters for the short period (SMS) and 1 second period (SM1) MCE spectral response acceleration with the formula:

$S_{MS} = S_s \times F_a$

$S_{M1} = S_1 \times F_v$

From the equation above, it is found that the value of SMS = 1.056 g and the valueof SM1 = 0.992 g. After the SMS and SM1 values are known, then the design spectral acceleration parameters for the short period (SDS) and 1 second period (SD1) are calculated using the following formula:

$$S_{DS} = \left(\frac{2}{3}\right) S_{MS} = 0.704 \text{ g}$$

$$S_{D1} = \left(\frac{2}{3}\right) S_{M1} = 0.661 \text{ g}$$

The SDS values obtained were 0.704 g and SD1 were 0.661 g. If the design spectral acceleration parameters are known, then the design response spectrum can be searched and graphed. There are several provisions based on SNI 1726:2019 Article 6.4 that must be followed to make a design response spectrum, including:

1. For periods smaller than T0, the design acceleration response spectrum (Sa) is found using the following equation:

$$S_a = S_{DS}\left(0{,}4 + 0{,}6\left(\frac{T}{T_0}\right)\right)$$

2. For periods greater than or equal to T0 and less than or equal to Ts, the design acceleration response spectrum (Sa) is the same as SDS.
3. For periods greater than Ts but smaller than or equal to TL (Long period transitionmap whose values are taken from Figure 20 SNI 1726:2019, the design acceleration spectral response (Sa) is calculated by the following formula:

$$S_a = \frac{S_{D1}}{T}$$

To find T (the fundamental vibration period of the structure) the following formula canbe used:

$$T_s = \frac{S_{DS}}{S_{D1}}$$

$$T_0 = 0{,}2\,T_s$$

So that the value of T_s = 1.065 seconds, T_0 = 0.213 seconds and the TL value is obtained according to the area where the building stands. For Muntilan, the TL value = 6 seconds

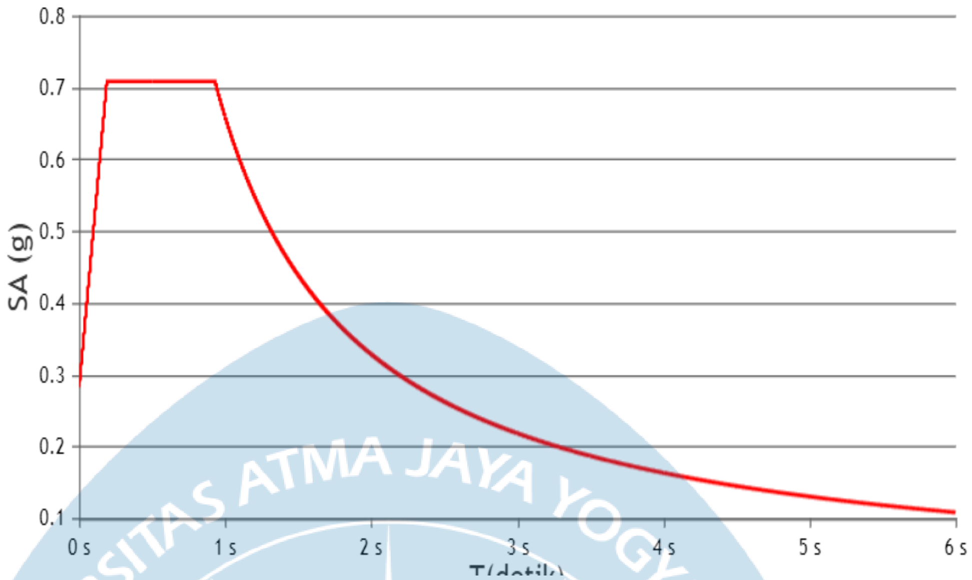


Figure 2.1 Graphic Design Spectrum Muntilan

2.3 Combine Load Plan

The ultimate load combination is determined based on Article 4.2.2 of SNI 1726:2019 Procedure for Calculation of Concrete Structures for Buildings, which areas follows:

1. 1,4DL
2. 1,2DL + 1,6LL + 0,5(Lr or R)
3. 1,2DL + 1,6(Lr atau R) + (1,0L or 0,5W)
4. 1,2DL + 1,0W + 1,0L + 0,5(Lr or R)
5. 1,2DL + 1,0E + 1,0LL
6. 0,9DL + 1,0W
7. 0,9DL + 1,0E

For load combinations number 5 and 7 which is a combination of earthquake loads, itis specifically regulated in Article 7.4 SNI 1726:2019 Earthquake Resistance Planning Standards for Building Structures, which are as follows:

1. $(1,2+0,2SDS)DL + 1,0LL \pm 0,3\rho Ex \pm 1,0\rho Ey$

2. $(1,2+0,2SDS)DL + 1,0LL \pm 1,0\rho Ex \pm 0,3\rho Ey$

3. $(0,9-0,2SDS)DL \pm 0,3\rho Ex \pm 1,0\rho Ey$

4. $(0,9-0,2SDS)DL \pm 1,0\rho Ex \pm 0,3\rho Ey$

While the combination of service loads is determined based on Article 4.2.3 SNI 1726:2019 Procedures for Calculation of Concrete Structures for Buildings, which are as follows :

1. DL

2. DL + LL

3. DL + (Lr or R)

4. DL + 0,75LL + 0,75(Lr or R)

5. DL + (0,6W or 0,7E)

6. DL + 0,75(0,6W or 0,7E) + 0,75LL + 0,75(Lr or R)

7. 0,6DL + 0,6W

8. 0,6DL + 0,7E

means,

DL = Dead load (self-weight of structure and additional dead load)

LL = Live load

Lr = Live load on roof structure

R = Rain load

W = Wind load

Ex = Earthquake load x direction

Ey = Earthquake load y direction

ρ = Redundancy factor

SDS = Design spectral acceleration parameters for a short period of 0,2 seconds

2.4 Strucure Modeling

Structural modeling is carried out to determine the internal forces that occur in structural elements and structural behavior due to workloads. The results of the structural modeling are used as the basis for designing the required cross-sectional dimensions of the structural elements. The structural model is carried out with severalidealizations. For example, floor slabs are idealized as shell elements, while beams and columns are idealized as truss elements.

The structural modeling carried out is able to accommodate the effects of steel damage during an earthquake, namely by reducing the moment of inertia of the cross-section of the structural elements. The moment of inertia of the plate is reduced to 25% of the initial moment of 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 by 25% to balance the reduction value against the inertia of the structuralelements. Whereas in the column, the moment of inertia is reduced to 70% of the initial moment of inertia.

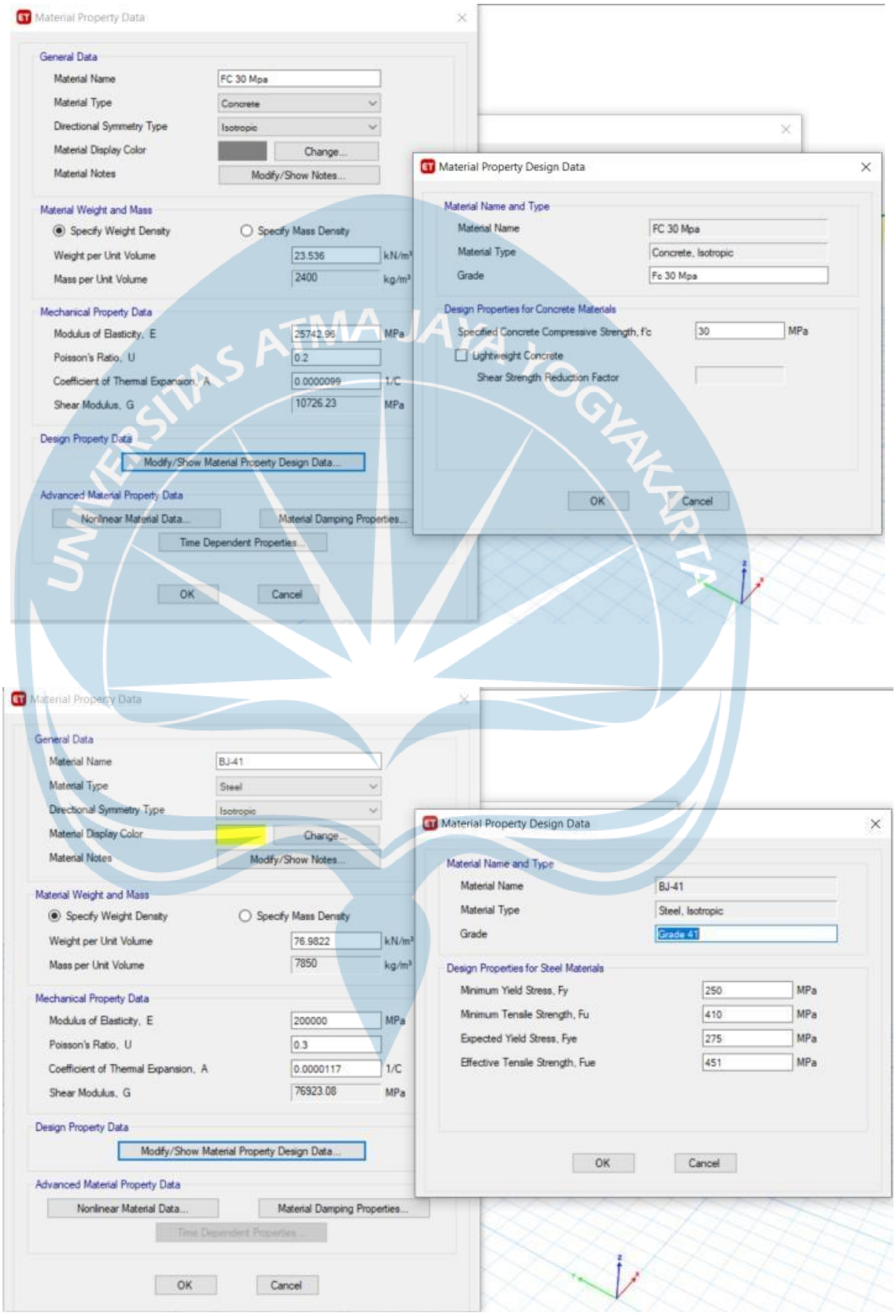
The structure of the restaurant is designed using a structural system in the form ofa special moment resisting frame structure. The structure is modeled in 3D models (3D Models) using software assistance.

2.4.1 Definition of Material

The materials used in the structural analysis are as follows:

Steel	; fy 280 MPa fu 370MPa
Concrete	; fc’ 30 MPa / K300
Reinforcing Steel	; fy 420 Mpa
Plain Reinforcement Steel	; fy 280 Mpa

Figure 2.2 Material Properties



2.4.2 Definition of Beam and Column Profile

The beam and column cross sections are defined as follows

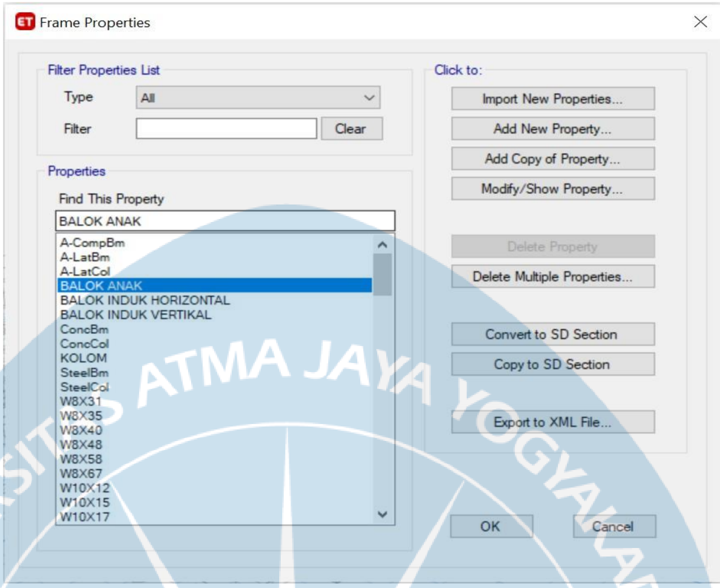


Figure 2.3. Section Properties

Example of Column Section Properties (H 300)

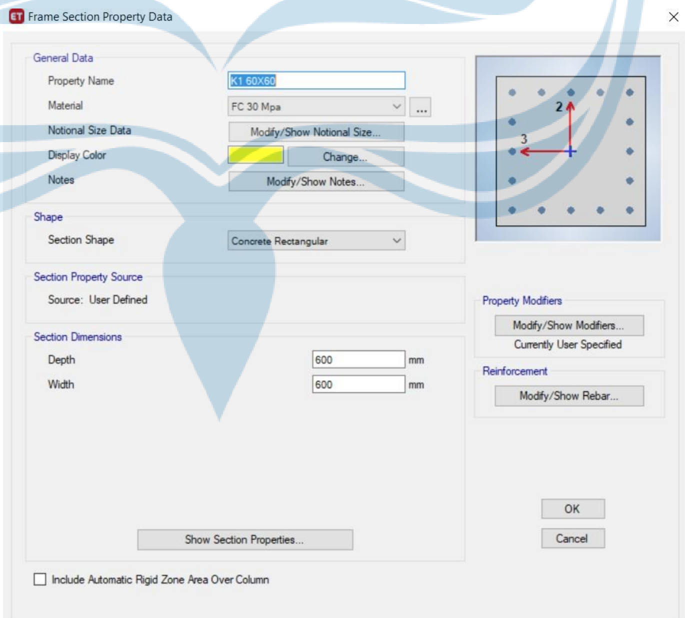


Figure 2.4. Section Properties Column K1

2.4.3 3D Modeling of Structure

After the material and section properties are complete, the next step is to create a3D model. The model accommodates all sizes of beams and columns, along withthe reinforcement planned to be installed as shown in Figure 4 below.

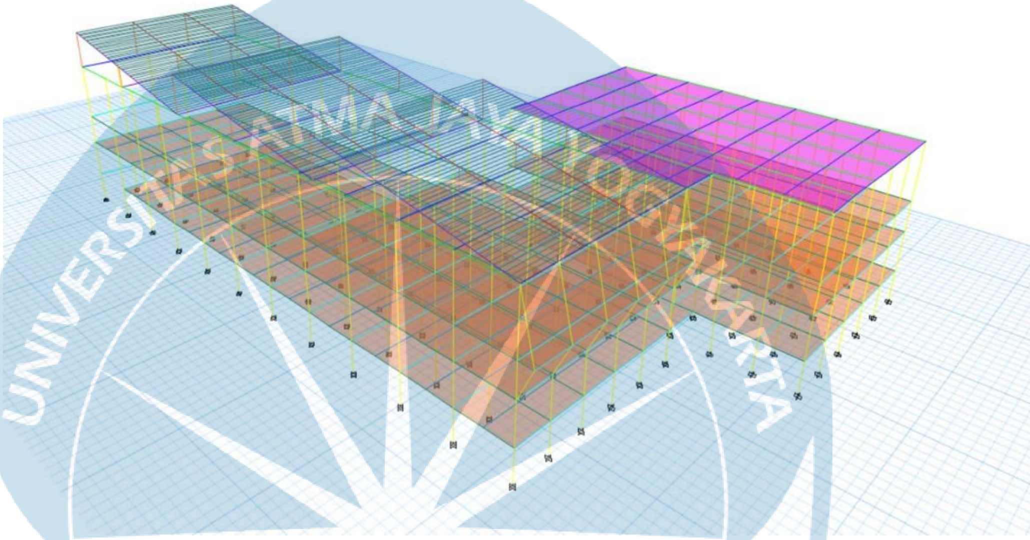


Figure 2.5. 3D Model of Building A (right) and Building B (left)

2.4.4 Giving Workload

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

- Dead load (self weight of steel, wall load),
- Additional dead load (ceramic load, pipe, ceiling etc.)
- Live load (working load), and
- Earthquake load. (response spectrum)

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

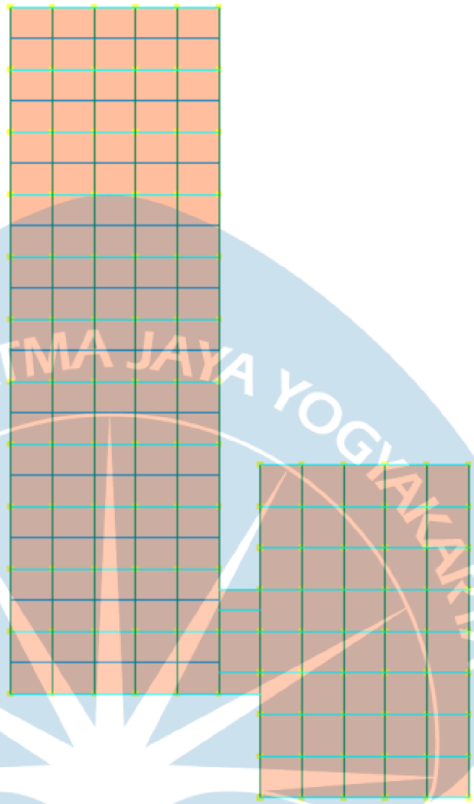


Figure 2.6. Loading of Floor Plates in 3D Model

2.4.5 Giving Earthquake Burden

Earthquake load is modeled in the program with the response spectrum function. Calculations and quantities can be seen in the Input data - response spectrum section. After obtaining the spectrum response graph, the graph is then input intothe program, as shown in Figure 6 below. After the earthquake load has entered, a combination of loading is carried out which allows several extreme loads to work together.

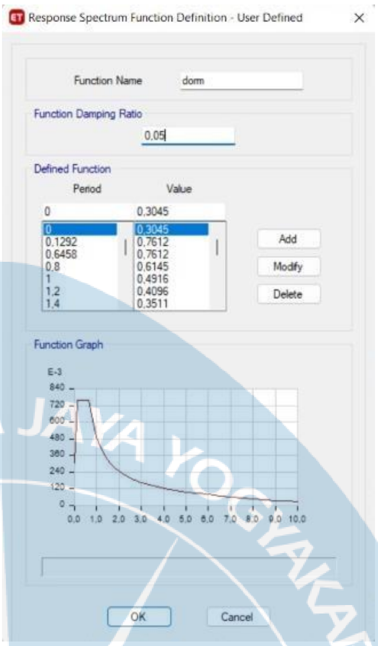


Figure 2.7. Earthquake Load Input

2.4.6 Combinations and Loading Factors

The load combination used refers to the 2012 Earthquake SNI, in this report the discussion of the load combination is carried out in the Data Input - Load Combination section.

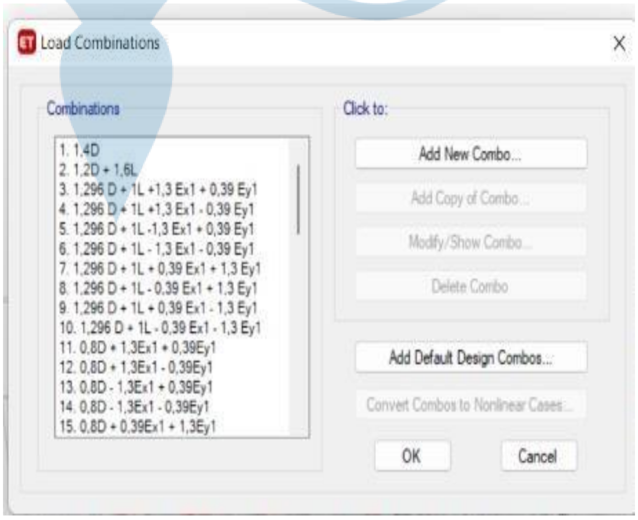


Figure 2.8. Load Combination Input

2.4.7 Running Program

After all the forces are installed, several treatments on the structure are carried out such as giving a mass source and a diaphragm, after which the programis run.

The result of running the program is in the form of internal forces acting on the beams and columns of the structure. This force is the key in analyzing the strength of the structure itself. The force obtained in the running results can be seen in Figure 2.

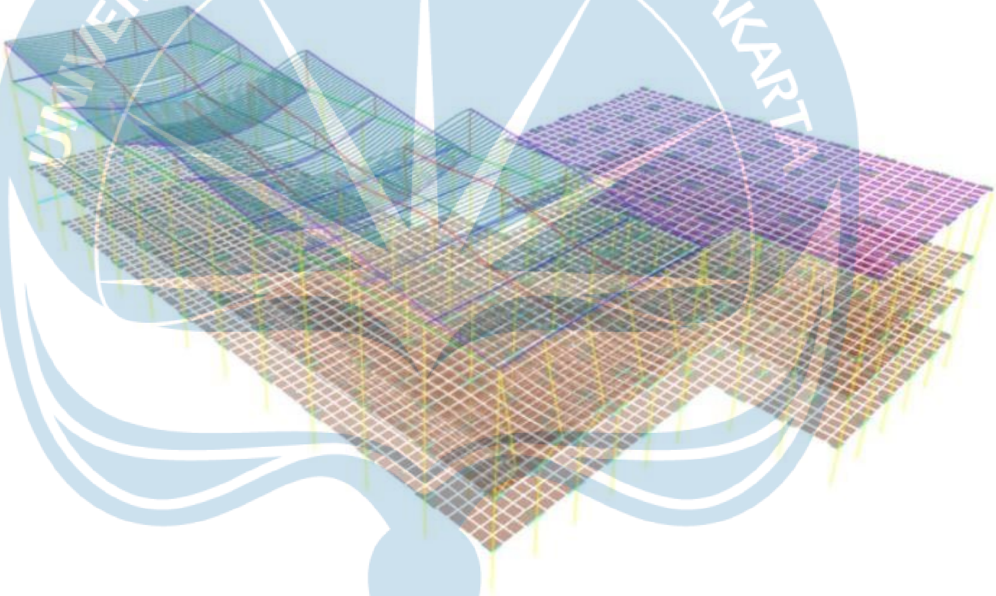
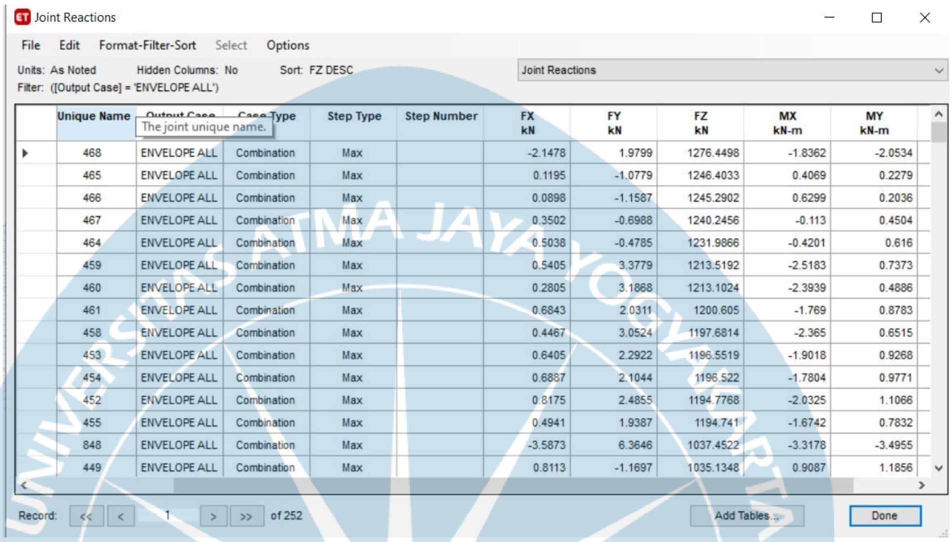


Figure 2.9 Running Results

2.4.8 Inner Result

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



Unique Name	Output Case	Case Type	Step Type	Step Number	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m
468	ENVELOPE ALL	Combination	Max		-2.1478	1.9799	1276.4498	-1.8362	-2.0534
465	ENVELOPE ALL	Combination	Max		0.1195	-1.0779	1246.4033	0.4069	0.2279
466	ENVELOPE ALL	Combination	Max		0.0898	-1.1587	1245.2902	0.6299	0.2036
467	ENVELOPE ALL	Combination	Max		0.3502	-0.6988	1240.2456	-0.113	0.4504
464	ENVELOPE ALL	Combination	Max		0.5038	-0.4785	1231.9866	-0.4201	0.616
459	ENVELOPE ALL	Combination	Max		0.5405	3.3779	1213.5192	-2.5183	0.7373
460	ENVELOPE ALL	Combination	Max		0.2805	3.1868	1213.1024	-2.3939	0.4886
461	ENVELOPE ALL	Combination	Max		0.6843	2.0311	1200.605	-1.769	0.8783
458	ENVELOPE ALL	Combination	Max		0.4467	3.0524	1197.6814	-2.365	0.6515
453	ENVELOPE ALL	Combination	Max		0.6405	2.2922	1196.5519	-1.9018	0.9268
454	ENVELOPE ALL	Combination	Max		0.6887	2.1044	1196.522	-1.7804	0.9771
452	ENVELOPE ALL	Combination	Max		0.8175	2.4855	1194.7768	-2.0325	1.1066
455	ENVELOPE ALL	Combination	Max		0.4941	1.9387	1194.741	-1.6742	0.7832
848	ENVELOPE ALL	Combination	Max		-3.5873	6.3646	1037.4522	-3.3178	-3.4955
449	ENVELOPE ALL	Combination	Max		0.8113	-1.1697	1035.1348	0.9087	1.1856

Figure 2.10 Inner Result

2.5 Roof Planning

The roof structure can be interpreted as the part of the building that holds or drains the load from the roof. The roof structure is divided into roof trusses and roof truss supports. The roof truss is the part that has the function of supporting the load of the roof covering in the form of a vertical and horizontal arrangement of beams. Supporting the roof truss itself is called the truss which serves to support the roof truss. These trusss are then connected to the structural columns to transfer the load to the ground. In the planning of the roof there is planning of truss elements, joints, and plans for curtains.

2.5.1 Purlin Plan

Roof planning begins with planning purlin. In planning purlin there are several thingsthat need to be considered :

a. For tiled roofs, the distance between the curtains in the horizontal direction can be taken at 1500 – 2000 mm, while for tin/asbestos roofs it is taken at 1000 – 1200mm.

b. The distance between the trusses determines the purlin distance, so the distancebetween the trusses should be the same as the distance between columns. However, if this is not possible, the truss distance is taken to be 2500 – 4000 mmfor tiled roofs, while for tin/asbestos roofs the truss distance is taken to be 6000 mm.

c. The number of sag-rods or tension rods that hold the load in the direction of the weak axes of the purlin is determined based on the distance between the trusses.The usual sag-rod distance is taken a maximum of 2000 mm.

d. Wind tie spans are cross-fitted between the trusses. Wind ties are allowed to be installed alternately, so there is no need to install each truss.

e. After considering the things above, the roof is planned according to the data below.

Table 2. 3 Roof Plan Data

3

Slope of roof (α)	10°
Distance between truss (L1)	2 m
Distance between purlin (a)	1 m
Roof covering	Metal roof
Quality of profile steel	BJ 41 (f_y = 250 MPa, f_u = 370 MPa)
Modulus of elasticity	200000000 N/m^2
Profile type	IWF 600 x 600 x 6 x 10
Cording	C 200 x 75 x 20

Purlin can be planned as follows:

a. Calculation of purlin loading

The roof load is 1.7238 kN/m, the curtain rodding self-load is 0.0637, and the ceiling weight is estimated at 0.4 kN/m. Calculation of curtain load can be seen in the following table:

Table 2.4 Purlin Load

Own weight	Estimated	0.0872 kN/m’
Roof weight	$\frac{\alpha}{\cos\alpha} \times roof\ weight$	0.812 kN/m’
Ceiling weight	$\alpha \times celling\ load$	0.4 kN/m’
Dead Load (DL)	Total purlin load	1.289 kN/m’
Live Load (L)	Worker Load	1 kN

b. Moment purlin plan

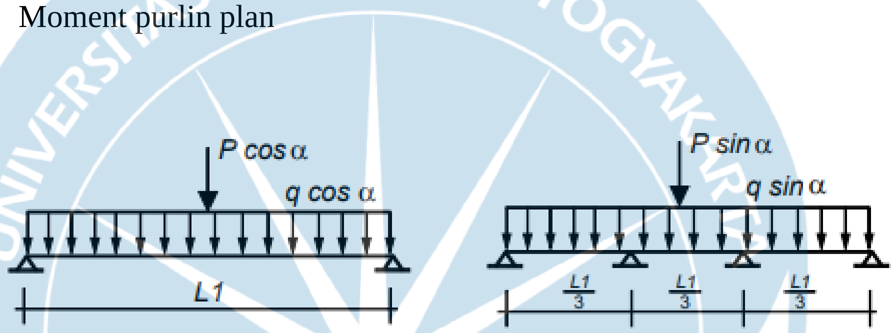


Figure 2.11 Purlin Load Direction Axis-2 And Axis-3

The purlin moment is searched in the 2nd and 3rd axis directions in the following way:

Table 2.5 Purlin Moment on Axis-2 And Axis-3 Direction

M_{3D}	$\frac{1}{8} q cos\alpha (\mathrm{L}_1)^2$	$2.5572\ kNm$
M_{3L}	$\frac{1}{4} P cos\alpha (\mathrm{L}_1)$	$0.492\ kNm$
M_{2D}	$\frac{1}{8} q sin\alpha \left(\frac{L_1}{3}\right)^2$	$0.1127\ kNm$
M_{2L}	$\frac{1}{4} P sin\alpha \left(\frac{L_1}{3}\right)$	$0.0868 kNm$

Then for find the ultimate moment (Mu) the following loading combinations bellow:

$$M_U = 1{,}4\ M_D$$

$$M_U = 1{,}2\ M_D + 1{,}6\ M_L$$

Tabel 2.6 Ultimate Moment for Axis-2 And Axis-3 Direction

M_{3U}	3.580 kNm
M_{3U}	3.857 kNm
M_{2U}	0.158 kNm
M_{2U}	0,274 kNm

From the two loading combinations, the largest M3U value was chosen, is 3.580 kNmand the largest M2U value is 0.274 kNm.

c. Check the tension on profile C

METRIC SIZE

DIMENSION	THICKNESS	SECTION AREA	WEIGHT UNIT	GEOMETRICAL MOMENT OF INERTIA		MODULUS OF SECTION		RADIUS OF GYRATION		CENTER OF GRAVITY	SHEAR CENTER	TORSION CONSTANT	WARPING CONSTANT
H x B x C	t	A	kg/m	Ix	Iy	Zx	Zy	rx	ry	Cy	Xo	J	Cw
mm	mm	cm²		cm⁴	cm⁴	cm³	cm³	cm	cm	cm	cm	cm⁴	cm⁶
C 100 x 50 x 20	2.0	4.54	3.56	71	17	14.3	5.4	3.97	1.93	1.87	4.48	605	444
	2.3	5.17	4.06	81	19	16.1	6.0	3.95	1.92	1.86	4.46	912	496
	2.5	5.59	4.39	87	20	17.3	6.5	3.94	1.90	1.86	4.45	1164	528
	2.8	6.20	4.87	95	22	19.1	7.1	3.92	1.89	1.86	4.42	1621	574
	3.0	6.61	5.19	101	23	20.2	7.4	3.91	1.88	1.86	4.41	1982	603
	3.2	7.01	5.50	106	24	21.3	7.8	3.90	1.87	1.86	4.40	2392	630
C 125 x 50 x 20	2.0	5.04	3.95	120	18	19.3	5.5	4.89	1.91	1.69	4.15	672	675
	2.3	5.75	4.51	136	21	21.8	6.2	4.87	1.89	1.69	4.12	1013	755
	2.5	6.21	4.88	147	22	23.5	6.6	4.86	1.88	1.69	4.11	1295	805
	2.8	6.90	5.42	162	24	25.9	7.2	4.84	1.86	1.69	4.08	1804	877
	3.0	7.36	5.78	172	25	27.5	7.6	4.83	1.85	1.69	4.07	2207	922
	3.2	7.81	6.13	181	27	29.0	8.0	4.82	1.84	1.68	4.05	2665	965
C 150 x 50 x 20	2.0	5.54	4.35	185	19	24.7	5.6	5.79	1.87	1.55	3.86	738	971
	2.3	6.32	4.96	210	22	28.0	6.3	5.77	1.86	1.55	3.84	1115	1088
	2.5	6.84	5.37	226	23	30.2	6.8	5.75	1.85	1.55	3.82	1425	1162
	2.8	7.60	5.97	250	26	33.3	7.4	5.73	1.83	1.54	3.80	1987	1267
	3.0	8.11	6.37	265	27	35.4	7.8	5.72	1.82	1.54	3.78	2432	1334
	3.2	8.61	6.76	280	28	37.4	8.2	5.71	1.81	1.54	3.77	2938	1398
C 150 x 65 x 20	2.0	6.14	4.82	218	36	29.1	8.3	5.96	2.43	2.12	5.19	818	1784
	2.3	7.01	5.50	248	41	33.0	9.4	5.94	2.42	2.12	5.16	1236	2006
	2.5	7.59	5.96	267	44	35.6	10.0	5.93	2.41	2.12	5.15	1581	2148
	2.8	8.44	6.63	295	48	39.4	11.0	5.91	2.39	2.12	5.13	2207	2352
	3.0	9.01	7.07	314	51	41.8	11.6	5.90	2.38	2.11	5.11	2702	2482
	3.2	9.57	7.51	332	54	44.2	12.2	5.89	2.37	2.11	5.09	3265	2608
C 200 x 75 x 20	2.0	7.54	5.92	467	56	46.7	10.6	7.87	2.73	2.20	5.49	1005	4571
	2.3	8.62	6.77	531	64	53.1	12.0	7.85	2.72	2.20	5.47	1520	5159
	2.5	9.34	7.33	573	68	57.3	12.9	7.84	2.71	2.20	5.45	1946	5537
	2.8	10.40	8.17	636	75	63.6	14.2	7.82	2.69	2.20	5.42	2719	6085
	3.0	11.11	8.72	676	80	67.6	15.0	7.80	2.68	2.19	5.41	3332	6437
	3.2	11.81	9.27	716	84	71.6	15.8	7.79	2.67	2.19	5.39	4030	6779

Figure 2.12 C-Profile

To check the tension on profile C (f_b), the following formula is used:

$$f_b = \frac{M'_{3U}}{\varphi W_3} + \frac{M'_{2U}}{\varphi W_2} \leq F_y$$

With a value of φ = 0.9 for bending and shear, W3 = 67.6 cm3 and W2 = 15.0 cm3, the fb value obtained is 83.698 MPa ≤ 240 MPa, so it according to terms. If not according to terms so choose another profile.

d. Check the purlin deflection

The purlin deflection can be calculated by the following formula:

Deflection 2-axis direction = $\delta 2 = \frac{5}{384}\frac{qcos\alpha(L_1)^4}{EI} + \frac{1}{48}\frac{Pcos\alpha(L_1)^3}{EI}$ = 1.6759 mm

Deflection 3-axis direction = $\delta 3 = \frac{5}{384}\frac{qsin\alpha(L_1)^4}{EI} + \frac{1}{48}\frac{Psin\alpha}{EI}\left(\frac{L_1}{3}\right)^3$ = 5.9728 mm

Deflection Purlin = $\delta = \sqrt{\delta_3^2 + \delta_2^2} \leq \frac{1}{240}L_1$ = 6.2035 mm

$$\frac{1}{240}\text{L} = 8.33 \text{ mm}$$

Because the value of $\delta = 6.2035 \leq 8.33$ mm , so the purlin is safe.

e. Sag-rod Calculation

The number of purlin under the nokis n = 10 rows. Sag-rod force for dead loadand live load can be calculated by the following formula:

$$F_{tD} = n.\left(\frac{L_1}{3}.qsin\alpha\right) = 3.006 \text{ kN}$$

$$F_{tL} = \frac{n}{2}.Psin\alpha = 1 \text{ kN}$$

From the two equations above, it is found that $F_{tD} = 3.006\ kN$ dan $F_{tL} = 1\ kN$.

Then, the value of the sag-rod force is put into the following load combinations::

$F_{tU} = 1{,}4.F_{tD}$ = 4.2085 kN

$F_{tU} = 1{,}2.F_{tD} + 1{,}6\ F_{tL}$ = 4.9965 kN

Then, the largest value ofF_{tU} is chosen. $F_{tU} = 4.9965$ kN

The required area of the sag-rod can be found using the following formula:

$$Asr = \frac{F_{tU}.10^3}{\varphi.\mathrm{F_y}} = 22.2065\ mm2$$

The required sag-rod area is 22.2065 mm2, so the selected sag-rod diameter is 10 mm.

2.5.2 Truss Load Plan

The plan for the load of the truss can be made after the purlin, sag-rod and so on are determined. Usually, the width of the trellis varies between 750 – 1250 mm. The width of the embankment (b) will be taken as 1000 mm or 1 m.

Loads P1, P2, and P3 are calculated according to the length of the purlin (distance between the trusses) and the purlin distance (supported of the width roof). Based on the applicable loading regulations, the self-weight of the trusss is estimated at 0.50 kN/m'. The per-m' purlin weight is 0.0637 kN/m and the roof weight is 0.953 kN/m obtained from the previous purlin calculations. The weight of the ceiling is also obtained from the calculation of the curtains at 0.297 kN/m.

Table 2. 7 Calculation of Projected Loads at Certain Points on the Roof

P_1 Load		
Truss own weight	$\frac{a}{2} x\ truss\ weight$	0.25 kN
Truss weight	$L_1\ x\ \ Weight\ of\ purlin\ per - m'$	0.34 kN
Roof weight	$\frac{\left(\frac{a}{2} + b\right)}{cos\alpha} x\ L_1\ x\ roof\ weight$	5.9391 kN
Ceiling weight	$\left(\frac{a}{2} + b\right)\ x\ L_1\ x\ ceiling\ weight$	2.88 kN
	P_1 Load =	9.41 kN
P_2 Load		
Truss own weight	$a\ x\ Truss\ weight$	0.5 kN
Truss weight	$L_1\ x\ Curtains\ weight\ per\ m$	0.34 kN
Roof weight	$\frac{a}{cos\alpha} x\ L_1\ x\ Roof\ weight$	6.59 kN

Ceiling weight	$a\ x\ L_1\ x\ Ceiling\ weight$	3.2 kN
	P₃ Load =	10.64 kN
P_3 Load		
Truss own weight	$a\ x\ Truss\ weight$	0.5 kN
Truss weight	2 x $L_1\ x\ Curtains\ weight\ per\ m$	0.69 kN
Roof weight	$\frac{a}{cos\alpha} x\ L_1\ x\ roof\ weight$	6.59 kN
Ceiling weight	$a\ x\ L_1\ x\ ceiling\ weight$	3.2 kN
	P_3 Load =	10.99kN

2.5.3 Wind Load

Wind load, Qw = 0.25 kN/m2

W1 Load = $\frac{(\frac{a}{2}+b)}{\cos\alpha}\ .Ch\ .L'.Qw$ = 0.8314

- X-way = Sin 10˚ × W1 = 0.4157
- Y-way = Cos 10˚ × W1 = 0.7200

W2 Load = $\frac{a}{\cos\alpha}\ .Ch\ .L'.Qw$ = 1.1733

- X-way = Sin 10˚ × W2 = 0.5867
- Y-way = Cos 10˚ × W2 = 2.1894

W3 Load = ½ $\frac{a}{\cos\alpha}\ .Ch\ .L'.Qw$ = 0.4619

- X-way = Sin 10˚ × W3= 0.2309
- Y-way = Cos 10˚ × W3 = 0.4000

W4 Load = ½ $\frac{a}{\cos\alpha}\ .Ch\ .L'.Qw$ = – 0.6928

- X-way = Sin 10˚ × W4 = – 0.6000
- Y-way = Cos 10˚ × W4 = –0.6093

W5 Load = $\frac{(\frac{a}{2}+b)}{\cos\alpha}\ .Ch\ .L'.Qw$ = – 1.2185

- X-way = Sin 10˚ × W1 = – 0.6093
- Y-way = Cos 10˚ × W1 = – 2.2738

W6 Load = $\frac{(\frac{a}{2}+b)}{\cos\alpha}\ .Ch\ .L'.Qw$ = – 1.2471

- X-way = Sin 10˚ × W1 = – 0.6235
- Y-way = Cos 10˚ × W1 = – 1.0800

2.5.4 Truss Element Plan

The planning of the truss elements uses WF profile steel 400 x 200 x 8 x 13. Things to consider in the planning of the truss elements are the planning of tension elements and the planning of compression elements. In the planning of the Muntilan Christian Church, all elements of the easel experience compression forces. The compressive rod force is a rod force which has a negative value. Compression element planning is carried out by checking profile slenderness, profile stability, moment capacity, shear capacity, and deflection control.

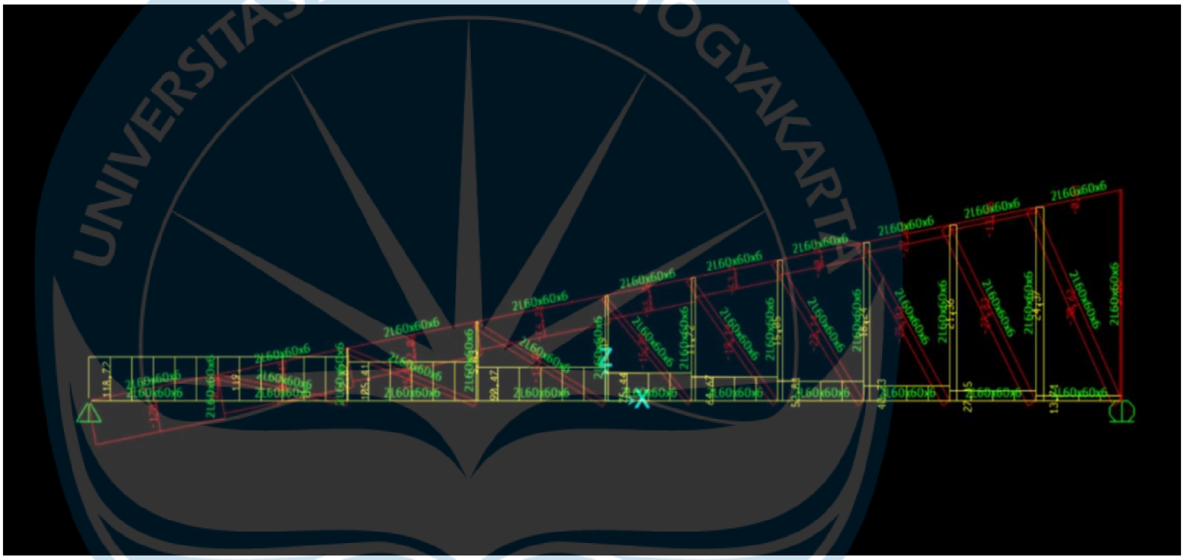


Figure 2. 13 Result from SAP2000

Table 2.8 Data profil WF 60 x 60 x 6

h	60	mm
b	60	mm
t	6	mm
$l_x = l_y$	228000	mm4
$i_x = i_y$	18.2	mm
$C_x = C_y$	17	mm
Tp	10	mm
j	2673	mm4
G	77200	Mpa
Ratio	0.3	SNI
E	200000	
Ag	1382	mm2
l_{xg}	456000	mm4
I_{yg}	896888	mm4
r_{xg}	18.2	mm
r_{yg}	25.47505235	mm
X0	0	mm
Y0	14.75	mm
r0	1196.497377	mm2
H	0.818167173	

(Source: www.grdsteel.com)

Table 2. 9 Data profil WF 65 x 65 x 6

h	65	mm
b	65	mm
t	6	mm
A	6.91	cm2
$l_{x=}$ l_y	126000	mm4
$i_{x=}$ i_y	14.9	mm
$C_{x=}$ C_y	14.4	mm
Tp	10	mm
j	2313	mm4
G	77200	Mpa
Ratio	0.3	SNI
E	200000	
Ag	1128	mm2
l_{xg}	252000	mm4
I_{yg}	550534.08	mm4
r_{xg}	14.9	mm
r_{yg}	22.09212818	mm
X0	0	mm
Y0	12.15	mm
r0	859.088883	mm2
H	0.82816388	

(Source: www.grdsteel.com)

2.5.5 Exterior Compression Member

a. Flexural Bending Check

$$\lambda = \frac{b}{t} = \frac{60}{6} = 10$$

$$\lambda_r = 0.45\sqrt{\frac{200000}{2}} = 12.7279$$

$\lambda = 10 < \lambda_r = 12.7279$ so the

b. Flexural Bending Check (Against X-X Axis)

$$\frac{\mathrm{KL}}{\mathrm{r_x}} = \frac{2000}{18.2} = 219.78$$

$$F_e = \frac{\pi^2 E}{18.2} = 40.865$$

$$4.71\sqrt{\frac{E}{240}} = 133.219$$

$$\frac{KL}{r_x} > 4.71\sqrt{\frac{E}{Fy}}, so\ Fcr\ using\ equation$$

$$F_{cr} = 0.877F_e = 35.8386$$

c. Check Against Torsional Bending

$$a = 2000$$

$$\frac{a}{r} = \frac{2000}{18.2} = 109.89 > 40, \text{so using} \left(\frac{KL}{r}\right)_m$$

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)^2 + \left(\frac{KL}{r}\right)^2} = 226.544$$

$$Because \left(\frac{KL}{r}\right)_m > 4.71\sqrt{\frac{E}{Fy}}, so\ Fcr\ using\ equation\ F_{cr} = 0.877F_e$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = 38.4613$$

$$F_{cry} = 0.877F_e = 33.7305$$

$$F_{crz} = \frac{\mathrm{GJ}}{A\ x\ r0} = 124.795$$

$$F_{crz} = \frac{F_{cry} + F_{crz}}{2H}\left[1 - \sqrt{1 - \frac{4F_{cry}.F_{crz}.H}{\left(F_{cry} + F_{crz})^2\right)}}\right] = 31.7592$$

d. Design Compressive Strength

F_{cr} = 35.8386Mpa

F_{cr} = 31.7592 Mpa Using Fcr which has a smaller value, namely Fcr = 31.7592 Mpa.

$\emptyset_c P_n = 0{,}9 \times F_{cr} \times A_g = 0 = 39502.1$ kN

$\emptyset_c P_n = 85{,}68$ kN > Gaya tekan maksimum = 7.33 kN (OK)

2.5.6 Interior Compression Member

a. Check Element Slenderness

$$\lambda = \frac{b}{t} = \frac{60}{6} = 10$$

$$\lambda_r = 0.45\sqrt{\frac{200000}{2}} = 12.7279$$

$\lambda = 10 < \lambda_r = 12.7279$

b. Flexural Bending Check (Against X-X Axis)

$$\frac{\mathrm{KL}}{\mathrm{r_x}} = \frac{2000}{18.2} = 187.589$$

$$F_e = \frac{\pi^2 E}{18.2} = 56.0937$$

$$4.71\sqrt{\frac{E}{240}} = 133.219$$

$$\frac{KL}{r_x} > 4.71\sqrt{\frac{E}{Fy}}, so\ Fcr\ using\ equation$$

$$F_{cr} = 0.877F_e = 49.1941$$

c. Check Against Torsional Bending

$a = 1397.54$

$\frac{a}{r} = \frac{1397.54}{14.9} = 93.7946 > 40$, so using $\left(\frac{KL}{r}\right)_m$

$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)^2 + \left(\frac{KL}{r}\right)^2} = 193.363$

$Because \left(\frac{KL}{r}\right)_m > 4.71\sqrt{\frac{E}{Fy}}, so\ Fcr\ using\ equation\ F_{cr} = 0.877F_e$

$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = 52.794$

$F_{cry} = 0.877F_e = 46.3004$ Mpa

$F_{crz} = \frac{\text{GJ}}{A\ x\ r0} = 184.266$ Mpa

$F_{cr} = \frac{F_{cry} + F_{crz}}{2H}\left[1 - \sqrt{1 - \frac{4F_{cry}.F_{crz}.H}{(F_{cry} + F_{crz})^2)}}\right] = 43.815$

d. Design Compressive Strength

F_{cr} = 49.1941 Mpa

F_{cr} = 43.815 Mpa Using Fcr which has a smaller value, namely Fcr = 43.815 Mpa.

$\emptyset_c P_n = 0{,}9 \times F_{cr} \times A_g = 54497.1$ kN

$\emptyset_c P_n = 85{,}68$ kN > Gaya tekan maksimum = 176.341 kN (OK)

2.5.7 Exterior Tension Member

1. **Calculation of Tensil Slenderness**

   $\lambda = \frac{L}{r} = \frac{2291.67}{18.2} = 125.916 < 300$ (OK)

2. **Tensile Check**

   $\emptyset Pn = FyAg = 0{,}9 \times 250 \times 1382 = 310950$ N

   $\emptyset Pn = 310950$ kN > Pu = 176.228 kN (OK)

2.5.8 Interior Tension Member

1. **Calculation of Tensile Slenderness**

   $\lambda = \frac{L}{r} = \frac{1375}{18.2} = 155.235 < 300$ (OK)

2. **Tensile Check**

   $\emptyset Pn = FyAg = 0{,}9 \times 250 \times 1128 = 253800$ N

   $\emptyset Pn = 253800$ kN > Pu = 20.35 kN (OK)

2.5.9 Connection Plan of Truss ElementsConnection Planning

a. Data

- A325-X bolts with bolt diameter used M-20
- Plate size used 6 x 250 mm
- ASTM A36 steel spliced
  buhul plate(fy: 240 Mpa ;
  fu: 410 Mpa)
- Strength of joglo roof pull rods:

  ➢ 176.228 kN (2L 60 x 60 x 6 Exterior profile)

  ➢ 20.305 kN (2L 50 x 50 x 6 Interior profile)

- BJ 41 (fy: 240 Mpa ; fu: 410 Mpa)

b. Tensile yield check on gross cross-section

Ag = 6 × 250 = 1500 mm2

ØPn = 0,9 × Fy × Ag

= 0,9 × 240 × 1500

= 324000 N

= 324 kN > 176.228 kN (2L 60x60x6 exterior profile)

= 324 kN > 20.305 kN (2L 50x50x6 interior profile)

c. Check of Tensile Collapse at Net Cross Section

An = (250-2 × (22 + 2)) × 6 = 1212 mm2

Max An = 0.85 × 1500 = 1275 mm2

Ae = An = 1212 mm2 ØPn

= 0.75 × Fu × Ae

= 0.75 × 410 × 1212

= 372690 N

= 372.69 kN > 250 kN (SAFE)

d. Bolt support strength

Rn = 2,4 × dt × Fu

= 2,4 × 20 × 6 × 410

= 118080 N

= 118.08 kN

ØRn = Ø × Rn (Ø : 0,75)

= 0,75 × 118.08

= 88.56 kN

e. Bolt Shear Strength

Rn = FnvAb

= 457 × (1/4 × π× 202) × 2

= 287141,5 N

ØRn = Ø × Rn (Ø : 0.75)

= 0,75 × 287141,5

= 215356 N

= 215.356 kN

Selected the smallest value of bolt fulcrum strength and bolt shear strengthØRn = 88.56 kN

f. Calculation of Number of Bolts

Number of Bolts = 250 × 88.56

= 2.82294

= **3 bolts**.

2.6 Stair Planing

Stairs are a very important structure for multi-storey buildings. The stairs are plannedfor the 1st and 2nd building with a size of 1980 x 4500 mm and a height of 3m. The stepshave a height (Optrede) of 180 mm and a width (Antrede) of 290 mm. The size of the stairs is designed with comfort in mind because if it is too high and too narrow it will cause users to feel uncomfortable. The stair plate has a thickness of 150 mm

To simplify the planning of the stairs, the staircase space plan is made as shown in Figure 2.18.

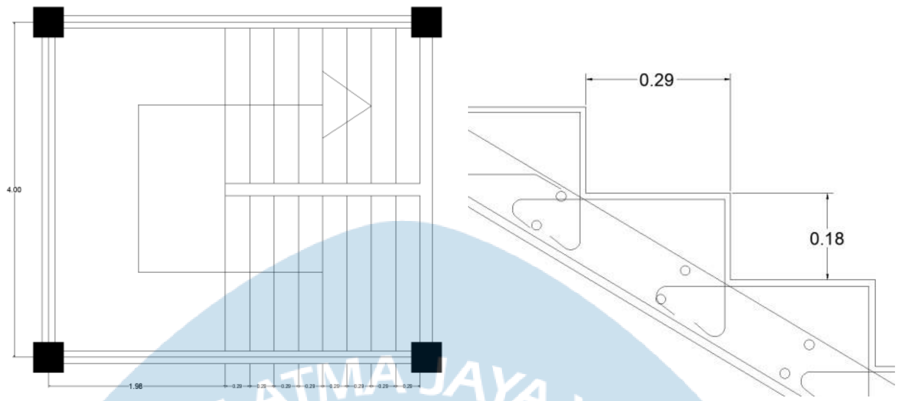


Figure 2. 14 Room Plan of Front Stairs and Stepss

The stair data required to plan the stairs can be seen in table 2.16.

Table 2.10 Stair Dimension

Floor width	2900 mm
Bordes width	1800 mm
Stair height (Optrede)	180 mm
Floor height (hlt)	3000 mm
Stair width (Antrede)	290 mm
Number of steps	16 pcs
Optrede/Antrede (O/A)	0,5833
Stair slope angle = arctan (O/A)	31.83 °
Thickness of stair slab (htg)	160 mm
Volume weight of concrete	24 kN/m^3
Volume weight of granite tiles	21 kN/m^3

2.6.1 Stair Load Plan

The load of the stairs can be known after planning the dimensions of the stairs which include, the slope of the stairs and the thickness of the stair plate The thickness ofstairs can be determined by using trial and error. Control is needed to ensure the safety with a concept of 600 $mm \leq 2 \times height\ of\ staircase\ x\ widht\ of\ starcase \leq 650\ mm$.

Table 2. 11 Stair Load Plan

Load qtg		
Self weight of stairs	$\frac{h_{tg}}{\cos \alpha} \times berat\ volume\ beton$	4.93 kN/m²
Weight of stairs	$\frac{1}{2} O \times berat\ volume\ beton$	2.4 kN/m²
Weight of ceramic tile and mortar	$0{,}05 \times berat\ volume\ ubin$	1.05 kN/m²
Railing weight (estimated)		1 kN/m²
	Load qtg	9.38 kN/m²
Load qbd		
Self weight of stairs	$h_{tg} \times berat\ volume\ beton$	4.32 kN/m²
Weight of ceramics and specs	$0{,}05 \times berat\ volume\ ubin$	1.05 kN/m²
Railing weight (estimated)		1 kN/m²
	Load q_{bd}	6.37 kN/m²

Live load (LL) is taken as 4.79 kN/m2 (building function as a house of worship).After the ladder load is determined, the plan force can be calculated using the help of theSAP2000 computer program. Images of SFD and BMD forces can be seen in the following figure.

The thickness of stairs can be determined by using trial and error. Control is neededto ensure the safety with a concept of $600\ mm \leq 2 \times height\ of\ staircase\ x\ widht\ of\ starcase \leq 650\ mm$. Then, 7 calculated the load that applied for the borders beam. Afterthat, input it into ETABS so that it can analyse the bending moment diagram (BMD).

a. *Shear Force Diagram* (SFD)

- Due to dead load

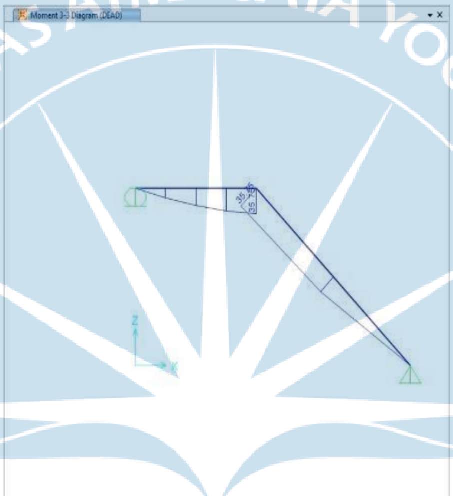


Figure 2.15 SFD Due to Dead Load

- Due to Live load

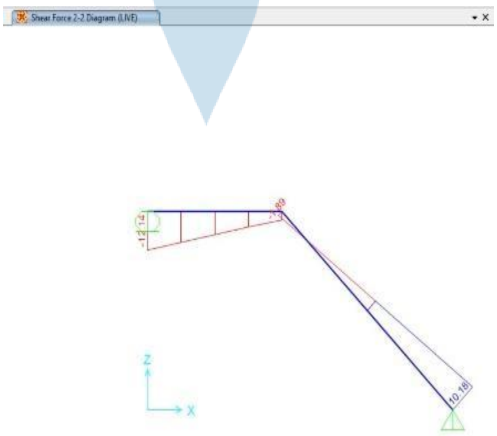


Figure 2.16 SFD Due to Live load

b. *Bending Momen Diagram* (BMD)

- Due to dead load

Figure 2.17 BMD Due to Dead Load

- Due to Live load

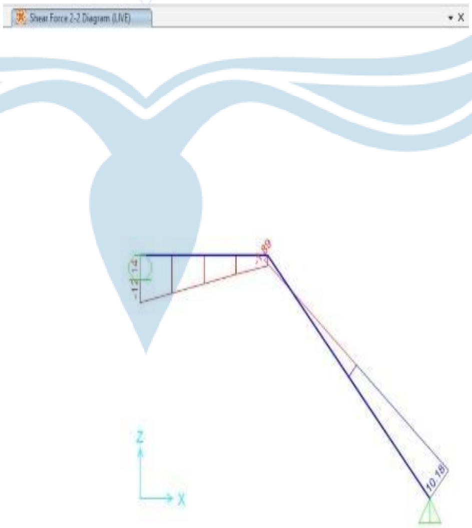


Figure 2.18 BMD Due to Live load

From the results of the SAP2000 analysis, the moments and shear on the stairs areobtained as follows:

Table 2.12 Front Stair Moment and Shear

	Live load	Dead load
Moment (kNm)	14.67	35.75
Shear (kN)	10.18	23.47

Table 2.13 Combination Moment and Shear

	Combination 1	Combination 2
Moment (kNm)	50.05	66.372
Shear (kN)	32.858	44.452
Choosing the largest:		
M_{ur}	66.372	
V_{ur}	44.452	

Table 2.14 Main Reinforcement Calculations

	Pedestal	Field
Moment	33.186 kNm	56.4 kNm
Rn	1.59	2.5
Ratio of reinforcemen needed	0.00402	0.00644
Area of reinforcement needed	201.06 mm^2	1159.29 mm^2
Reinforcement spacing	277.67 mm	114.49 mm
Main reinforcement	D16-300	D13-200
Shrinkage reinforcement	P10-200	

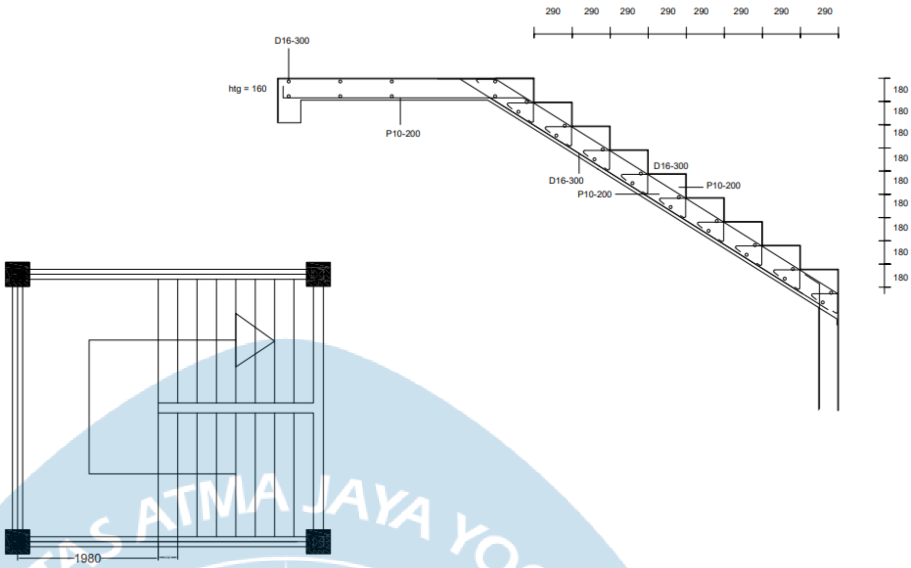


Figure 2. 19 Stair reinforcement

2.7 Slab Structural Planning

One part of the construction that needs to be calculated is slab planning. The calculation of the slab itself uses calculation assumptions involving the load acting onthe floor slab, the moment due to the factored load, and the plate reinforcement. Fromthe results of the calculation of the floor slab itself, the diameter of the reinforcementand the required reinforcement spacing will be obtained. The thickness of the floor slab is 15 cm for the 1st and 2nd building. The reinforcement used on the entire floor is reinforcement with a diameter of 10 and a spacing of 200 mm.

Slab is a flat and horizontal structure made of cast concrete. A concrete slab is a horizontal, flat building form. Building layout determines the different types of slabs.The more beams in a building, the more different types of slabs that can be made in accordance with SNI specifications. There are three types of slabs in this situation: types A, B, C, and D. Each type comes in a variety of lengths and widths. The Type A, B, and C slabs must be divided into one-way and two-way slabs. Ly Lx 2 for one-way slabs and Ly Lx 2 for two-way slabs is the formula to determine slab category.

To determine the moment that occurs in the structure, modeling in ETABS is required. The amount of reinforcement needed to withstand the moment is calculatedusing the slab's moment. 8 The more reinforcement is required, the higher the momentvalue. After moment analysis, the spacing between each reinforcement must be taken into account. The ability to resist the moment force decreases with increasing distancebetween reinforcements.

2.7.1 Preliminary Design

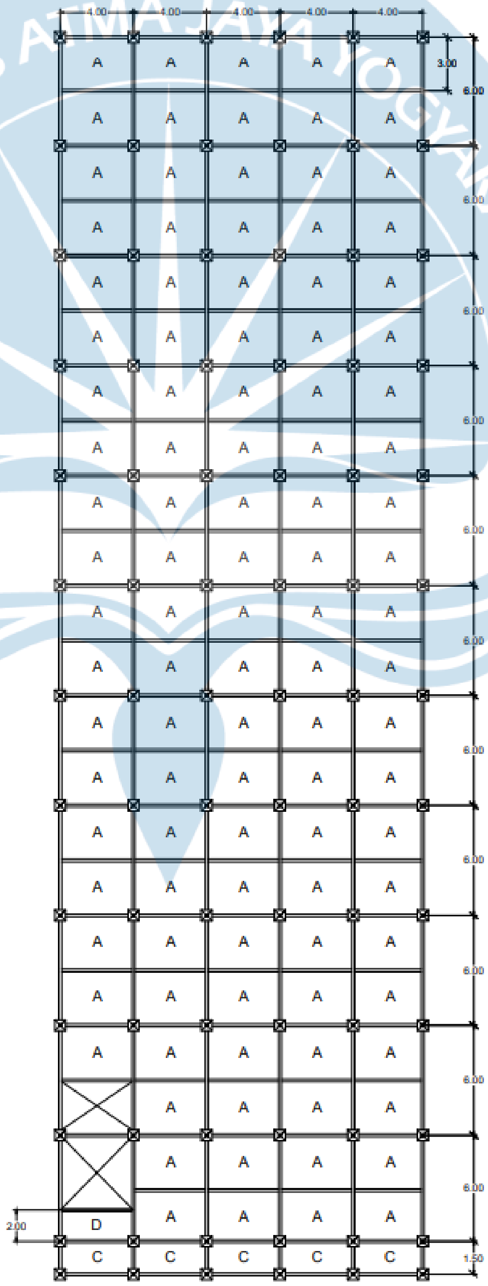


Figure 2. 20 Slab Plan

Table 2. 15 Loading of 1st and 2nd Building Slab

Slab Function	Types of Loading	Thickness (cm)	Vol	DL	DL Slub	LL	W_u = 1.2 D + 1.6L
			kN/ m^2	kN/m^2	kN/m^2	kN/m^2	kN/m^2
Floor	Own-load	120	24	3			
	Finishing Tile Load	50	21	1.05			
	Ceiling Load		0.2	0.2			
	ME installation			0.5			
			Total	4.75	2,13	4,79	13,364

Preliminary Design of the slab is the minimum provision allowed to design thefloor slab. The minimum plate thickness has been stated in SNI 2847: 2019 through a table:

Table 2.16 Minimum Thickness of Prestressed One-way Solid Slab

Support Condition	***h minimum***
Simple Support	l / 20
One ways	l / 24
Two ways	l / 28
Cantilever	l / 10

Through the table, if the yield stress used is more than 420 MPa, then the analysis of the minimum thickness (*h*) of the plate can be multiplied by 0.4 + f_y / 700 (SNI 2847: 2019). The use of plates at Muntilan Christian Church is calculated through this reference. To determine the floor slab in one-way or two-way Muntilan

Church planning, it needs to be seen through the formula: $\frac{Ly}{Lx} < 2$, for one-way slub and $\frac{Ly}{Lx} > 2$, for two-way slub

The floor slab of Muntilan Christian Church has 4 slab types, namely A, B, C and D. The calculation shows that Depok Church has a one-way floor slab as a whole. For that, the minimum h is used with the formula:

h min = $\frac{Lx \times 1000}{24}$. The result of the h calculation itself becomes the reference for determining the thickness of the plate used. For the all type Slub, *a* slab thickness of 150 mm is used, while for the first and second floors, a slub thickness of 150 mm is used as a safety requirement.

Table 2.17 Slub Types of 1st and 2nd Building

Floor	Ly/Lx	Ket	Lx (m)	Thickness (mm)	Used
A	1.33	two-way	3	83,3333	150
B	1	one-way	4	83,3333	150
C	2,6667	two-way	1.5	93,7500	150
D	2	one-way	2	93,7500	150

2.7.2 Calculation of Slab Type A

The reinforcement calculation of slab has been divided into 4 types of slab types as shown in the table above. This calculation is based on the loading as shown in Table 2. 29

The slab reinforcement plan is carried out after Wu is obtained. Wu was found tobe 13.364 kNm^2 . Wu on the slab can be found with the following combination:

$W_u = 1.2\ M_{DL} + 1.6\ M_{LL} = 13.364\ kN/m^2$

The slab reinforcement plan is calculated with the following known data:

Table 2. 18 General Data of Slab

Concrete blanket	20 mm
d	150 mm (pedestal)
Main reinforcement	d10 (pedestal)
Main reinforcement	d10 (field)
f_y	420 MPa
f'_c	30 MPa
h	150 mm

The following are the steps for calculating the reinforcement of floor slab:

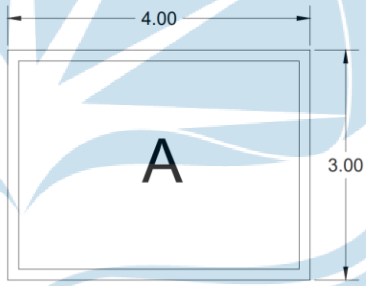


Figure 2.21 Slab Type A

Table 2. 19 Slab Type A Data

L_y	4000 mm
L_x	3000 mm
M_u	$0{,}001 \times W_u \times lx^2 \times k$
M_{lx}	3.824 kNm
M_{ly}	-3.824 kNm
M_{tx}	1.63476 kNm
M_{ty}	-1.63476kNm

The floor slab calculation step is divided into the calculation of x-direction field reinforcement, y-direction field reinforcement, x-direction pedestal reinforcement and y-direction pedestal reinforcement. The following is an example of calculation for x- direction field reinforcement:

1. Find d

   The effective height (d_x) can be found by the following formula:d_x = h slab - concrete blanket – d reinforcement/2

   The effective height is 113.5 mm

2. Finding the Coefficient of Cross-Sectional Capacity (Rn)

   The cross-sectional capacity coefficient is obtained through equation $Rn = \frac{M_u}{\emptyset b d^2}$ assuming that the tensile restraint, ø = 0.9. The result of the cross-sectional capacity coefficient (Rn) is 0.3298

3. Finding Reinforcement Ratio

   The minimum reinforcement ratio according to SNI is 0.0018. If the reinforcement ratio is smaller than 0.0018, then 0.0018 is used. If the reinforcementratio is greater than 0.0018, the calculated reinforcement ratio is used. In addition to checking the minimum reinforcement ratio, the maximum reinforcement ratio is also checked to see if it is overreinforced. The reinforcement ratio is obtained by equation :

$$\rho_{need} = \frac{0{,}85 f_c'}{f_y}\left(1 - \sqrt{1 - \frac{2Rn}{0{,}85 f_c'}}\right).$$

   The result of the reinforcement ratio obtained from the formula calculation is 0.0007917, which is smaller than that of ρ_{min} 0.0018, and certainly smaller than the maximum reinforcement ratio of 0.03299. After checking the minimum and maximum reinforcement ratios, the following are used $\rho_{minimum} = 0{,}0018$.

4. Finding the Number of Tensile Reinforcement

First must be known is the area of reinforcement required. The reinforcement area needs to be found with equation, then the dieter reinforcement area is calculated.

A_s D10 = $\left(\frac{1}{4}\right)\pi D^2$ The area of reinforcement needed based on the calculation is 227 mm^2. Then the area of reinforcement with a diameter of 10 is 663.666 mm^2. Then d10 reinforcement is used.

5. Finding the reinforcement spacing

The result of 359.0392 mm is obtained from the calculation of the spacing with equation, then the maximum spacing of 3 times the thickness of the plate is 45 mm. Therefore, a spacing of 200 mm is taken.

After obtaining a reinforcement spacing, then main reinforcement and **D13-200** reinforcement are used.

Calculation of Y-Direction Field Reinforcement is the same as X-Direction Support Reinforcement with different M_u values according to Table 2.20 Thus, the location of the difference in calculations is only in $R_{n, need.}$ The reinforcement of the X andY direction supports also has a difference in the value of M_u so that with the same calculation method as the X direction Field Reinforcement Calculation, therefore, the calculation of Y Direction Field Reinforcement, X Direction Support Reinforcement Calculation, and Y Direction Support Reinforcement Calculation at the *basement* floor are summarized in the appendix table.

Table 2. 20 X Direction Field Reinforcement

Type	dx	Way	Rn	ρ	ρ min	ρ used	As need	s use	As Installed	Use Reinforcement				Check
	mm	Mlx					mm		mm					
A	113.5	3,66408	0.32982	0.00079	0,002	0,0020	227	300	442.4409654	D	13	-	300	OK
B	113.5	3,66408	0.23747	0.00057	0,002	0,0020	227	300	442.4409654	D	13	-	300	OK
C	113.5	8,24418	0.05844	0.00014	0,002	0,0020	227	300	442.4409654	D	13	-	300	OK
D	113.5	8,11332	0.05751	0.00014	0,002	0,0020	227	300	442.4409654	D	13	-	300	OK

Table 2. 21 Y Direction Field Reinforcement

Type	dx	Way	Rn	ρ	ρ min	ρ used	As need	s use	As Installed	Use Reinforcement				Check
	mm	Mly					mm		mm					
A	100.5	0,75608	0.17984	0.00043	0,002	0,002	201	300	442.4409654	D	10	-	300	OK
B	100.5	0,75608	0.30288	0.00073	0,002	0,002	201	300	442.4409654	D	10	-	300	OK
C	100.5	1,70118	0.10937	0.00026	0,002	0,002	201	300	442.4409654	D	10	-	300	OK
D	100.5	4,5801	0.29447	0.00071	0,002	0,002	201	300	442.4409654	D	10	-	300	OK

Table 2. 22 X Direction Pedestal Reinforcement

Type	dx	Way	Rn	ρ	ρ min	ρ used	As need	s use	As Installed	Use Reinforcement				Check
	mm	Mlx					mm		mm					
A	125	3,66408	0.32128	0.00077	0,0020	0,002	230	300	442.4409654	D	10	-	300	OK
B	125	2,21008	0.23132	0.00055	0,0020	0,002	230	300	442.4409654	D	10	-	300	OK
C	125	4,97268	0.05693	0.00014	0,0020	0,0021	230	300	442.4409654	D	10	-	300	OK
D	125	4,5801	0.05602	0.00013	0,0020	0,0020	230	300	442.4409654	D	10	-	300	OK

Table 2.23 Y Direction Pedestal Reinforcement

Type	dx	Way	Rn	ρ	ρ min	ρ used	As need	s use	As Installed	Use Reinforcement				Check
	mm	Mlx					mm		mm					
A	125	2,21008	0.16475	0.00039	0,0020	0,0020	210	300	442.4409654	D	10	-	300	OK
B	125	2,21008	0.27748	0.00067	0,0020	0,0020	210	300	442.4409654	D	10	-	300	OK
C	125	4,97268	0.10020	0.00024	0,0020	0,0020	210	300	442.4409654	D	10	-	300	OK
D	125	4,5801	0.26977	0.00065	0,0020	0,0020	210	300	442.4409654	D	10	-	300	OK

2.8 Beam Planning

In this instance, the beam structure consists of a primary beam and a practical beam.The primary beam connects each column and withstands the majority of forces that are applied to the building. The secondary beam is the part of the building that helps supportthe slab with an excessively long length and resist forces that occur in the building. Since the reinforcement beam in this instance is being used, the reinforcement needs to be calculated to arrive at the ideal value that will give it enough strength to withstand the force.

According to ETABS calculations, the beam that needs to be examined has the largest moment on each floor. The biggest moment was chosen because, if the beam canwithstand it, other, smaller moments that may occur in the building will also be able to withstand it. The area of steel is used to calculate the diameter of the reinforcement usedafter calculating the total moment that occurs in the beam. The maximum spacing neededto achieve effective strength can be calculated based on the total amount of steel used. Itis also necessary to model the beam in AutoCAD to ensure that other people can see it clearly.

2.8.1 Preliminary Design

The initial stage in beam planning is to conduct preliminary design or determine the initial dimensions of the beam. According to SNI 2847:2019, the minimum beam height is as follows.

$$h_{min} = \left(\frac{l}{18,5}\right)$$

Based on SNI 2847:2019, the net span for structural components, l_n should not be less than four times its effective height.

$$l_n \geq 4\ d$$

Meanwhile, the component width, b_w should not be less than 0.3h and 250 mm.

For a 40 x 55 main beam, it has span (l) = 6000 mm. The 40 x 50 main beam has span (l) = 4000 mm.

Table 2. 24 Preliminary Design of Beams

Beams	h min	h plan	ln	4d	bw	0,3h	Check
BI 40 x 55	500	550	10600	2350	400	165	Ok
BI 40 x 50	324,324	500	5500	1750	400	150	Ok

2.8.2 Main Beam 40 x 55

Table 2.25 Specification of 40 x 65 Main Beam

Beam Data	
Beam width (b)	400 mm
Beam height (h)	550 mm
Total span (l)	6000 mm
Concrete blanket (Cc)	40 mm
Concrete quality (f'c)	25 MPa
Reinforcement Data	
Diameter of flexural reinforcement (D)	25 mm
Diameter of stirrup reinforcement (d)	12 mm
Flexural reinforcement quality (f_y)	420 MPa
Shear reinforcement quality (f_{yt})	280 MPa
Modulus of Elasticity (E)	200000 MPa
β_1	0,85
d = h - 2Cc - 2 d. stirrup - 2 d.reinforcemnt	587.5 mm

Based on the structural analysis with the ETABS program, the forces in the beamare obtained as in the following table.

Table 2. 26 Inner force Main beam 40 x 55

Force	Reaction	
	- (kNm)	+ (kNm)
M_u^- (kNm)	-299.08	183.230
V_u^+ (kNm)	183.2297	145.4425

1. Reinforcement Calculation

Table 2.27 Calculation of Reinforcement

Concrete stress distribution form factor, For : fc' ≤ 30 MPa,	b_1	0.85
Reinforcement ratio at balanced condition	$\rho_{maks} = 0{,}75 \frac{0{,}85 f_c' \beta_1}{f_y} \left(\frac{600}{600+f_y}\right)$	0.030
Maximum moment resistance factor	$R_{max} =$ 0.75 × rb × fy × [1 – ½ × 0.75 × rb × fy / (0.85 × fc')]	7.770
Flexural strength reduction factor	ϕ	0.80
Distance of reinforcement to the outside of the concrete	*ds* = ts + Ø + D/2	66 mm
Number of reinforcement bars in one row	*ns* = (b - 2 × ds) / (25 + D)	5.380
Distance of Horizontal center to center between reinforcement bars	x = (b - ns × D - 2 × ds) / (ns - 1)	30.71 mm

2. Calculation of Support Reinforcement

Table 2.28 Calculation of support Shear Reinforcement

Nominal positive moment of the plan	$M_n = M_u^+ / \phi$	373.850 kNm
Estimated distance of center of reinforcement to side of concrete	ds	66 mm
Effective height of the beam	d = h - ds	486 mm
Moment resistance factor	$R_n = M_n \times 10^6 / (b \times d^2)$	3.982
$R_n > R_{max}$		OK
Ratio of required reinforcement	$\rho = 0.85 \times fc' / fy \times [1 - Ö \times [1 – 2 \times Rn / (0.85 \times fc')]$	0.0104
Minimum ratio of reinforcement	$\rho_{min} = \frac{\sqrt{f_c'}}{4f_y}.$	0.0033
Miximum ratio of reinforcement	$\rho_{maks} = 0{,}75 \frac{0{,}85 f_c' \beta_1}{f_y} \left(\frac{600}{600+f_y} \right)$	0.0228
Ratio of reinforcement used	ρ	0.0104
Area of reinforcement required	$A_s = \rho \times b \times d$	2009 mm^2
Required Number of reinforcement,	$n = A_s / (p / 4 \times D^2)$	4.095 ≈ 5
Used reinforcement	5Ø25	
Area of reinforcement used	$A_s = n \times \pi \times b3 \times D^2$	2454 mm^2
Checking/Beam Analysis		
Equivalent concrete compressive stress block height	a = As × fy / (0.85 × fc' × b)	101.062 mm
Nominal moment,	$M_n = A_s \times f_y \times (d - a / 2) 10^{-6}$	447.350 kNm
Moment resistance of the beam	ϕMn	357.880 kNm
Check Requirement:	$\phi Mn \geq Mu+$	**Safe (OK)**
Check the width of Beam		
Thickness of concrete	2 × 40	80 mm

blanket:		
2 foot stirrups	2 × 13	26 mm
Number of reinforcement bars used	5 × 22	125 mm
Net distance between reinforcements	4 × 25	100 mm
TOTAL		331 mm
Check Requirement:	Total < b beam	**OK**

3. Calculation of field Reinforcement

Table 2.29 Calculation of field Reinforcement

Nominal positive moment of the plan	$M_n = M_u^+ / \phi$	229.037 kNm
Estimated distance of center of reinforcement to side of concrete	ds =	66 mm
Effective height of the beam	d = h - ds =	486 mm
Moment resistance factor	$R_n = M_n \times 10^6 / (b \times d^2) =$	2.4393
$R_n > R_{max}$		OK
Ratio of required reinforcement	$\rho = 0.85 \times fc' / fy \times [1 - Ö \times [1 – 2 \times Rn / (0.85 \times fc')]$	0.0061
Minimum ratio of reinforcement	$\rho_{min} = \frac{\sqrt{f_c'}}{4 f_y}.$	0.0033
Miximum ratio of reinforcement	$\rho_{maks} = 0{,}75 \frac{0{,}85 f_c' \beta_1}{f_y} \left(\frac{600}{600+f_y} \right)$	0.0228
Ratio of reinforcement used	ρ	0.0033
Area of reinforcement required	$A_s = \rho \times b \times d$	632 mm^2
Required Number of reinforcement	$n = A_s / (\pi / 4 \times D^2)$	1.287 ≈ 3
Used reinforcement	3Ø22	

Area of reinforcement used	$A_s = n \times \pi / 3 \times D^2$	2945 mm^2
Checking/Beam Analysis		
Equivalent concrete compressive stress block height	a = As × fy / (0.85 × fSc' × b)	121.275 mm
Nominal moment	$M_n = A_s \times f_y \times (d - a / 2) 10^{-6}$	524.319 kNm
Moment resistance of the beam,	ϕMn	419.455 kNm
Check Requirement:	$\phi Mn \geq Mu+$	Safe (OK)
Check the width of Beam		
Thickness of concrete blanket:	2 × 40	80 mm
2 foot stirrups:	2 × 13	26 mm
Number of reinforcement bars used	6 × 25	150 mm
Net distance between reinforcements	5 × 25	125 mm
TOTAL		381 mm
Check Requirement:	Total < b beam	**OK**

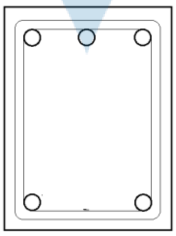
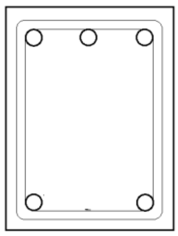
4. Calculation Shear Reinforcement

Table 2.30 Calculation Shear Reinforcement

Ultimate plan shear force,	V_u	146.365 kNm
Shear strength reduction factor,	ϕ	0.75 kN
Yield stress of shear reinforcement	f_y	280
Shear strength of concrete	$V_c = (\sqrt{f_c'}) / 6 \times b \times d \times 10^{-3}$	176.367 kN
Shear resistance of concrete	$\phi \times V_c =$	132.688 kN
		→ Need Shear Reinforcment

Shear resistance of stirrups	$f \times V_s = V_u - f \times V_c =$	13.681
Shear strength of stirrups	V_s	18.241 kN
Cross-sectioned stirrups are used	Ø	13
Area of shear reinforcement per meter of beam length	$A_{v\,=}\ Vs \times S/fy \times d$	0.134 mm^2
	$A_{v/s\ need}$	0.134 mm^2/mm
Maximum spacing of stirrup	$s_{max} = d / 2$	242 mm
Taken stirrup spacing	s	150 mm
Area used for shear reinforcement	$A_v = 2 \times 1/4 \times p \times Ø^2$	265 mm^2
	$A_{v/s\ use}$	1.769 mm^2/mm
A stirrup is used	13 Ø 150 mm	
Check Requirement	$A_{v/s\ need} > A_{v/s\ use}$	**OK**

Table 2.31 Recap of Reinforcement of Main Beam 40 x 55

Beam 40x55		
Size	Pedestal	Field
		
Main Reinforcement	5D25	6D25
Stirrup	2P13-150	2P13-150

2.8.3 Main Beam 40 x 50

Table 2. 32 Specification of 40 x 50 Main Beam

Beam Data	
Beam width (b)	400 mm
Beam height (h)	500 mm
Total span (l)	4000 mm
Concrete blanket (Cc)	40 mm
Concrete quality (f'c)	30 MPa
Reinforcement Data	
Diameter of flexural reinforcement (D)	25 mm
Diameter of stirrup reinforcement (d)	13 mm
Flexural reinforcement quality (f_y)	420 MPa
Shear reinforcement quality (f_{yt})	280 MPa
Modulus of Elasticity (E)	200000 MPa
β_1	0,85
d = h - 2Cc - 2 d. stirrup - 2 d.tul	587.5 mm

Based on the structural analysis with the ETABS program, the forces in the beam are obtained as in the following table.

Table 2. 33 Inner force Main beam 40 x 55

Force	Reaction	
	- (kNm)	+ (kNm)
M_u (kNm)	-299.08	183.2297
V_u (kNm)	-108.069	103.2157

1. Reinforcement Calculation

Table 2. 34 Calculation of Reinforcement

Concrete stress distribution form factor, For : fc' ≤ 30 MPa,	b_1 =	0.85
Reinforcement ratio at balanced condition	$\rho_{maks} = 0{,}75\,\frac{0{,}85 f_c' \beta_1}{f_y}\left(\frac{600}{600+f_y}\right)$	0.030
Maximum moment resistance factor	R_{max} = 0.75 × rb × fy × [1 – ½ × 0.75 × rb × fy / (0.85 × fc')]	7.770
Flexural strength reduction factor	ϕ	0.80
Distance of reinforcement to the outside of the concrete	*ds* = ts + Ø + D/2	66 mm
Number of reinforcement bars in one row	*ns* = (b - 2 × ds) / (25 + D)	5.380
Distance of Horizontal center to center between reinforcement bars	x = (b - ns × D - 2 × ds) / (ns - 1)	30.71 mm

2. Calculation of Support Reinforcement

Table 2.35 Calculation of support Shear Reinforcement

Nominal positive moment of the plan	$M_n = M_u^+ / \phi$	373.850 kNm
Estimated distance of center of reinforcement to side of concrete	ds	66 mm
Effective height of the beam	d = h - ds	436 mm
Moment resistance factor	$R_n = M_n \times 10^6 / (b \times d^2)$	4.951
$R_n > R_{max}$		OK
Ratio of required	ρ = 0.85 × fc' / fy × [1 - Ö × [1 –	0.0132

reinforcement	2 × Rn / (0.85 × fc')]	
Minimum ratio of reinforcement	$\rho_{min} = \frac{\sqrt{f_c'}}{4 f_y}.$	0.0033
Miximum ratio of reinforcement	$\rho_{maks} = 0{,}75 \frac{0{,}85 f_c' \beta_1}{f_y}\left(\frac{600}{600+f_y}\right)$	0.0228
Ratio of reinforcement used	ρ	0.0132
Area of reinforcement required	$A_s = \rho \times b \times d$	2299 mm^2
Required Number of reinforcement,	$n = A_s / (p / 4 \times D^2)$	4.670 ≈ 5
Used reinforcement	5Ø25	
Area of reinforcement used	$A_s = n \times \pi \times b3 \times D^2$	2454 mm^2
Checking/Beam Analysis		
Equivalent concrete compressive stress block height	a = As × fy / (0.85 × fc' × b)	101.062 mm
Nominal moment,	$M_n = A_s \times f_y \times (d - a / 2) 10^{-6}$	396.809 kNm
Moment resistance of the beam	ϕMn	316.647 kNm
Check Requirement:	ϕMn ≥ Mu+	**Safe (OK)**
Check the width of Beam		
Thickness of concrete blanket:	2 × 40	80 mm
2 foot stirrups	2 × 12	26 mm
Number of reinforcement bars used	5 × 25	125 mm
Net distance between reinforcements	4× 25	100 mm
TOTAL		331 mm
Check Requirement:	Total < b beam	**OK**

3. Calculation of field Reinforcement

Table 2.36 Calculation of field Reinforcement

Nominal positive moment of the plan	$M_n = M_u^+ / \phi$	229.037 kNm
Estimated distance of center of reinforcement to side of concrete	ds =	66 mm
Effective height of the beam	d = h - ds =	435 mm
Moment resistance factor	$R_n = M_n \times 10^6 / (b \times d^2) =$	3.0330
$R_n > R_{max}$		OK
Ratio of required reinforcement	$\rho = 0.85 \times fc' / fy \times [1 - Ö \times [1 – 2 \times Rn / (0.85 \times fc')]$	0.0077
Minimum ratio of reinforcement	$\rho_{min} = \frac{\sqrt{f_c'}}{4 f_y}.$	0.0033
Miximum ratio of reinforcement	$\rho_{maks} = 0{,}75 \frac{0{,}85 f_c' \beta_1}{f_y} \left(\frac{600}{600 + f_y} \right)$	0.0228
Ratio of reinforcement used	ρ	0.0033
Area of reinforcement required	$A_s = \rho \times b \times d$	567 mm²
Required Number of reinforcement	$n = A_s / (\pi / 4 \times D^2)$	1.154 ≈ 6
Used reinforcement	6Ø25	
Area of reinforcement used	$A_s = n \times \pi / 3 \times D^2$	2945 mm²
Checking/Beam Analysis		
Equivalent concrete compressive stress block height	a = As × fy / (0.85 × fSc' × b)	121.275 mm
Nominal moment	$M_n = A_s \times f_y \times (d - a / 2) 10^{-6}$	462.469 kNm
Moment resistance of the beam,	ϕMn	369.975 kNm
Check Requirement:	$\phi Mn \geq Mu+$	**Safe (OK)**
Check the width of Beam		
Thickness of concrete	2 × 40	80 mm

blanket:		
2 foot stirrups:	2 × 12	26 mm
Number of reinforcement bars used	6 × 25	150 mm
Net distance between reinforcements	5 × 25	125 mm
TOTAL		381mm
Check Requirement:	Total < b beam	**OK**

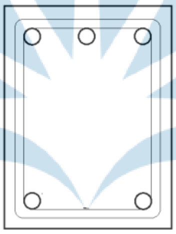
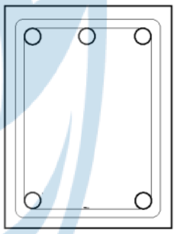
4. Calculation Shear Reinforcement

Table 2.37 Calculation Shear Reinforcement

Ultimate plan shear force,	V_u	108.062 kNm
Shear strength reduction factor,	ϕ	0.75 kN
Yield stress of shear reinforcement	f_y	280
Shear strength of concrete	$V_c = (\sqrt{f_c'}) / 6 \times b \times d \times 10^{-3}$	156.657 kN
Shear resistance of concrete	$f \times V_c =$	118.993 kN
		→ Need Shear Reinforcment
Shear resistance of stirrups	$f \times V_s = V_u - f \times V_c =$	-
Shear strength of stirrups	V_s	108.068 kN
Cross-sectioned stirrups are used	Ø	13
Area of shear reinforcement per meter of beam length	$A_{v=}$ Vs × S/fy × d	0.888 mm^2
	$A_{v/s\ need}$	0.888 mm^2/mm

Maximum spacing of stirrup	$s_{max} = d / 2$	217 mm
Taken stirrup spacing	s	150 mm
Area used for shear reinforcement	$A_v = 2 \times 1/4 \times p \times Ø^2$	265 mm^2
	$A_{v/s\ use}$	1.769 mm^2/mm
A stirrup is used	13 Ø 150 mm	
Check Requirement	$A_{v/s\ need} > A_{v/s\ use}$	**OK**

Table 2. 38 Recap of Reinforcement of Main Beam 40 x 50

Beam 40x50		
	Pedestal	Field
Image		
Main Reinforcement	5D25	6D25
Stirrup	2P13-150	2P13-150

2.8.4 Secondary Beam 250 x 350

Table 2.39 Specification of 25 x 35 Secondary Beam

Beam Data	
Beam width (b)	250 mm
Beam height (h)	350 mm
Total span (l)	4000 mm
Concrete blanket (Cc)	40 mm
Concrete quality (f'c)	25 MPa
Reinforcement Data	
Diameter of flexural reinforcement (D)	25 mm
Diameter of stirrup reinforcement (d)	12 mm

Flexural reinforcement quality (f_y)	420 MPa
Shear reinforcement quality (f_{yt})	280 MPa
Modulus of Elasticity (E)	200000 MPa
β_1	0,85
d = h - 2Cc - 2 d. stirrup - 2 d.tul	587.5 mm

Based on the structural analysis with the ETABS program, the forces in the beam are obtained as in the following table.

Table 2.40 Inner force structural beam 40 x 55

Force	Reaction	
	- (kNm)	+ (kNm)
M_u^- (kNm)	-23.2667	14.3491
V_u^+ (kNm)	-18.591	18.1829

1. Reinforcement Calculation

Table 2.41 Calculation of Reinforcement

Concrete stress distribution form factor, For : fc' ≤ 30 MPa,	b_1 =	0.85
Reinforcement ratio at balanced condition	$\rho_{maks} = 0,75\frac{0,85f_c'\beta_1}{f_y}\left(\frac{600}{600+f_y}\right)$	0.030
Maximum moment resistance factor	R_{max} = 0.75 × rb × fy × [1 – ½ × 0.75 × rb × fy / (0.85 × fc')]	7.770
Flexural strength reduction factor	ϕ	0.80
Distance of reinforcement to the outside of the concrete	*ds* = ts + Ø + D/2	64 mm

Number of reinforcement bars in one row	*ns* = (b - 2 × ds) / (25 + D)	2.596
Distance of Horizontal center to center between reinforcement bars	x = (b - ns × D - 2 × ds) / (ns - 1)	40.67 mm

2. Calculation of Support Reinforcement

Table 2.42 Calculation of support Shear Reinforcement

Nominal positive moment of the plan	$M_n = M_u^+ / \phi$	29.658 kNm
Estimated distance of center of reinforcement to side of concrete	ds	66 mm
Effective height of the beam	d = h - ds	286 mm
Moment resistance factor	$R_n = M_n \times 10^6 / (b \times d^2)$	1.450
$R_n > R_{max}$		**OK**
Ratio of required reinforcement	ρ = 0.85 × fc' / fy × [1 - Ö × [1 – 2 × Rn / (0.85 × fc')]	0.0036
Minimum ratio of reinforcement	$\rho_{min} = \frac{\sqrt{f_c'}}{4 f_y}$.	0.0033
Miximum ratio of reinforcement	$\rho_{maks} = 0{,}75 \frac{0{,}85 f_c' \beta_1}{f_y} \left(\frac{600}{600+f_y} \right)$	0.0228
Ratio of reinforcement used	ρ	0.0036
Area of reinforcement required	$A_s = \rho \times b \times d$	254 mm^2
Required Number of reinforcement,	$n = A_s / (p / 4 \times D^2)$	0.669 ≈ 3
Used reinforcement	3Ø22	
Area of reinforcement used	$A_s = n \times \pi \times b3 \times D^2$	1140 mm^2
Checking/Beam Analysis		
Equivalent concrete compressive stress block height	a = As × fy / (0.85 × fc' × b)	75.132 mm
Nominal moment,	$M_n = A_s \times f_y \times (d - a / 2) 10^{-6}$	118.992 kNm

Moment resistance of the beam	ϕMn	95.193 kNm
Check Requirement:	$\phi Mn \geq Mu+$	**Safe (OK)**
Check the width of Beam		
Thickness of concrete blanket:	2×40	80 mm
2 foot stirrups	2×12	26 mm
Number of reinforcement bars used	3×22	66 mm
Net distance between reinforcements	2×25	50 mm
TOTAL		222 mm
Check Requirement:	Total < b beam	**OK**

3. Calculation of field Reinforcement

Table 2.43 Calculation of field Reinforcement

Nominal positive moment of the plan	$M_n = M_u^+ / \phi$	17.936 kNm
Estimated distance of center of reinforcement to side of concrete	ds =	64 mm
Effective height of the beam	d = h - ds =	286 mm
Moment resistance factor	$R_n = M_n \times 10^6 / (b \times d^2) =$	0.8771
$R_n > R_{max}$		**OK**
Ratio of required reinforcement	$\rho = 0.85 \times fc' / fy \times [1 - Ö \times [1 - 2 \times Rn / (0.85 \times fc')]$	0.0021
Minimum ratio of reinforcement	$\rho_{min} = \frac{\sqrt{f_c'}}{4 f_y}.$	0.0033
Miximum ratio of reinforcement	$\rho_{maks} = 0,75 \frac{0,85 f_c' \beta_1}{f_y} \left(\frac{600}{600 + f_y} \right)$	0.0228
Ratio of reinforcement used	ρ	0.0033
Area of reinforcement	$A_s = \rho \times b \times d$	233 mm^2

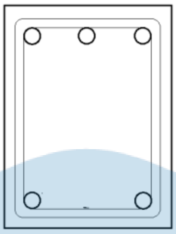
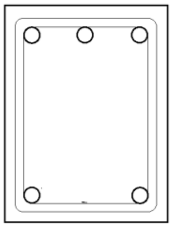
required		
Required Number of reinforcement	$n = A_s / (\pi / 4 \times D^2)$	$0.613 \approx 3$
Used reinforcement	3Ø22	
Area of reinforcement used	$A_s = n \times \pi / 3 \times D^2$	1140 mm^2
Checking/Beam Analysis		
Equivalent concrete compressive stress block height	a = As × fy / (0.85 × fSc' × b)	75.132 mm
Nominal moment	$M_n = A_s \times f_y \times (d - a / 2) 10^{-6}$	118.992 kNm
Moment resistance of the beam,	ϕMn	95.193 kNm
Check Requirement:	ϕMn ≥ Mu+	**Safe (OK)**
Check the width of Beam		
Thickness of concrete blanket:	2 × 40	80 mm
2 foot stirrups:	2 × 12	26 mm
Number of reinforcement bars used	3 × 22	66 mm
Net distance between reinforcements	2 × 25	50 mm
TOTAL		222 mm
Check Requirement:	Total < b beam	**OK**

4. Calculation Shear Reinforcement

Table 2.44 Calculation Shear Reinforcement

Ultimate plan shear force,	V_u	18.183 kNm
Shear strength reduction factor,	ϕ	0.75 kN
Yield stress of shear reinforcement	f_y	280
Shear strength of concrete	$V_c = (\sqrt{f_c'}) / 6 \times b \times d \times 10^{-3}$	65.270 kN
Shear resistance of concrete	$f \times V_c =$	48.953 kN
	→ Need Shear Reinforcment	
Shear resistance of stirrups	$f \times V_s = V_u - f \times V_c =$	-
Shear strength of stirrups	V_s	18.183 kN
Cross-sectioned stirrups are used	Ø	13
Area of shear reinforcement per meter of beam length	$A_{v\,=}$ Vs × S/fy × d	0.227 mm²
	$A_{v/s\ need}$	0.227 mm²/mm
Maximum spacing of stirrup	$s_{max} = d / 2$	143 mm
Taken stirrup spacing	s	150 mm
Area used for shear reinforcement	$A_v = 2 \times 1/4 \times p \times Ø^2$	265 mm²
	$A_{v/s\ use}$	1.769 mm²/mm
A stirrup is used	13 Ø 150 mm	
Check Requirement	$A_{v/s\ need} > A_{v/s\ use}$	**OK**

Table 2.45 Recap of Reinforcement of Secondary Beam 25 x 35

B30x35		
	Pedestal	Field
Image		
Main Reinforcement	3D25	3D25
Stirrup	2P13-150	2P13-150

2.9 Column Planning 600 X 600

In column planning, which begins with determining the preliminary design using a predetermined main beam size, followed by determining the size of the column to be used. Followed by an examination of the column portal type in the x and y directions. Checking the slenderness of columns on each floor starting with the calculation of top-down bridles, effective length factors in the x and y directions. Then calculate the longitudinal reinforcement of the column with structural analysis data based on ETABS followed by the calculation of flexural reinforcement using SPColumn which results in the number of reinforcement bars and reinforcement diameter. Furthermore, the calculation of column transverse reinforcement is based on SNI 2847-2019 SRPMK column.

2.9.1 Preliminary Design

In determining the preliminary design using the beam size to determine the appropriate column size.

Table 2.46 Preliminary Design Data

Column Height (l)	3000 mm
Column Net Height (lu)	3000 mm
L beam 1	4000 mm
L beam 2	6000 mm

2.9.2 Column 60 x 60

Table 2.47 Specification of 60 x 60 Main Column

Column Data	
Column width (b)	600 mm
Column height (h)	600 mm
Total span (l)	3000 mm
Concrete blanket (Cc)	40 mm
Concrete quality (f'c)	30 MPa
Reinforcement Data	
Diameter of flexural reinforcement (D)	25 mm
Diameter of stirrup reinforcement (d)	13 mm
Flexural reinforcement quality (f_y)	420 MPa
Shear reinforcement quality (f_{yt})	280 MPa
Modulus of Elasticity (E)	200000 MPa

Based on the structural analysis with the ETABS program, the forces in the column are obtained as in the following table.

Table 2.48 Inner force Main column 60 x 60

Force	Reaction	
	- (kNm)	+ (kNm)
M_2 (kNm)	-507.051	14.3491
M_3 (kNm)	-776.632	782.046
V_2 (kNm)	-195.8095	195.7748
V_3 (kNm)	-163.4204	156.9172
P (kNm)	-1395.3604	298.7161

1. Column Reinforcement Calculation

Table 2.49 Calculation of Reinforcement

The distance of the center of the bending reinforcement to the concrete side	$d_s = t_s + Æ + D/2$	66 mm
Effective High Column	$d = b - ds$	535 mm
The magnitude of the minimum eccentricity (e0)	$e_0 = Mu / Pu$	2681 mm
	$e_{min} = 15 + 0{,}03h$	33 mm
	$Agr = b \times h$	360000 ≈ 0.360
	$Pu = 0{,}1 \times f'c \times Agr$	1080.0
Determine the reduction factor	Pu < 0,1 × f'c × Agr	
	298.72 > 1080.0	
	Because Pu > 0,1 × f'c × Agr, so using f = 0.65	
Determine Reinforcement	Pu / f × Agr × 0,85 × f'c	0.005
	(Pu / f × Agr × 0,85 × f'c) × (e0 / h)	0.022
	d' / h	0.109
Four side reinforcement column is obtained	r = b × r	0.0100
	$A_{st} = r \times Agr$	3600
Area of Reinforcement used	$n = A_s / (p / 4 \times D^2)$	7.331

Used reinforcement	**8D25**	
	$A_s = n \times (1/4 \times p \times D^{2)}$	3925 mm^2
The distance between the column deform reinforcement	x = h - 2×ds	469
Maximal nominal axial force	ϕPn = 0.8×ϕ× (0,85×fc× (Ag-As)+(fy×As))	5578.775
Check Requirement:	ϕPn ≥ Pu	**Safe(OK)**
Check the width of Beam		
Thickness of concrete blanket:	2 × 40	80 mm
2 foot stirrups	2 × 13	26 mm
Number of reinforcement bars used	5 × 25	125 mm
Net distance between reinforcements	4 × 25	100 mm
TOTAL	331 mm	
Check Requirement:	Total < b beam	**OK**

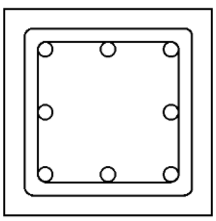
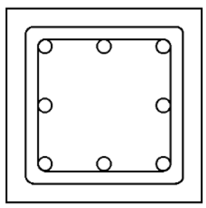
2. Calculation of Shear Reinforcement

Table 2.50 Calculation Shear Reinforcement

Ultimate plan shear force,	V_u	195.775 kNm
Shear strength reduction factor,	ϕ	0.75 kN
Yield stress of shear reinforcement	f_y	280
Shear strength of concrete	$V_c = (\sqrt{f_c'}) / 6 \times b \times d \times 10^{-3}$	292.758 kN
Shear resistance of concrete	f ×V_c =	219.568 kN

		→Need Shear Reinforcment
Shear resistance of stirrups	$f \times V_s = V_u - f \times V_c =$	-
Shear strength of stirrups	V_s	18.183 kN
Cross-sectioned stirrups are used	Ø	13
Area of shear reinforcement per meter of beam length	$A_v = V_s \times S/fy \times d$	714.286 mm^2
	$A_{v/s\ need}$	0.714 mm^2/mm
Maximum spacing of stirrup	$s_{max} = d / 2$	267 mm
Taken stirrup spacing	s	150 mm
Area used for shear reinforcement	$A_v = 2 \times 1/4 \times p \times Ø^2$	265 mm^2
	$A_{v/s\ use}$	1.769 mm^2/mm
A stirrup is used	13 Ø 150 mm	
Check Requirement	$A_{v/s\ need} > A_{v/s\ use}$	**OK**

Table 2. 51 Recap of Reinforcement of Structural Column 60 x 60

C60x60		
	Pedestal	Field
Image		
Side Reinforcement	8D25	8D25
Sengkang	2P13-150	2P13-150

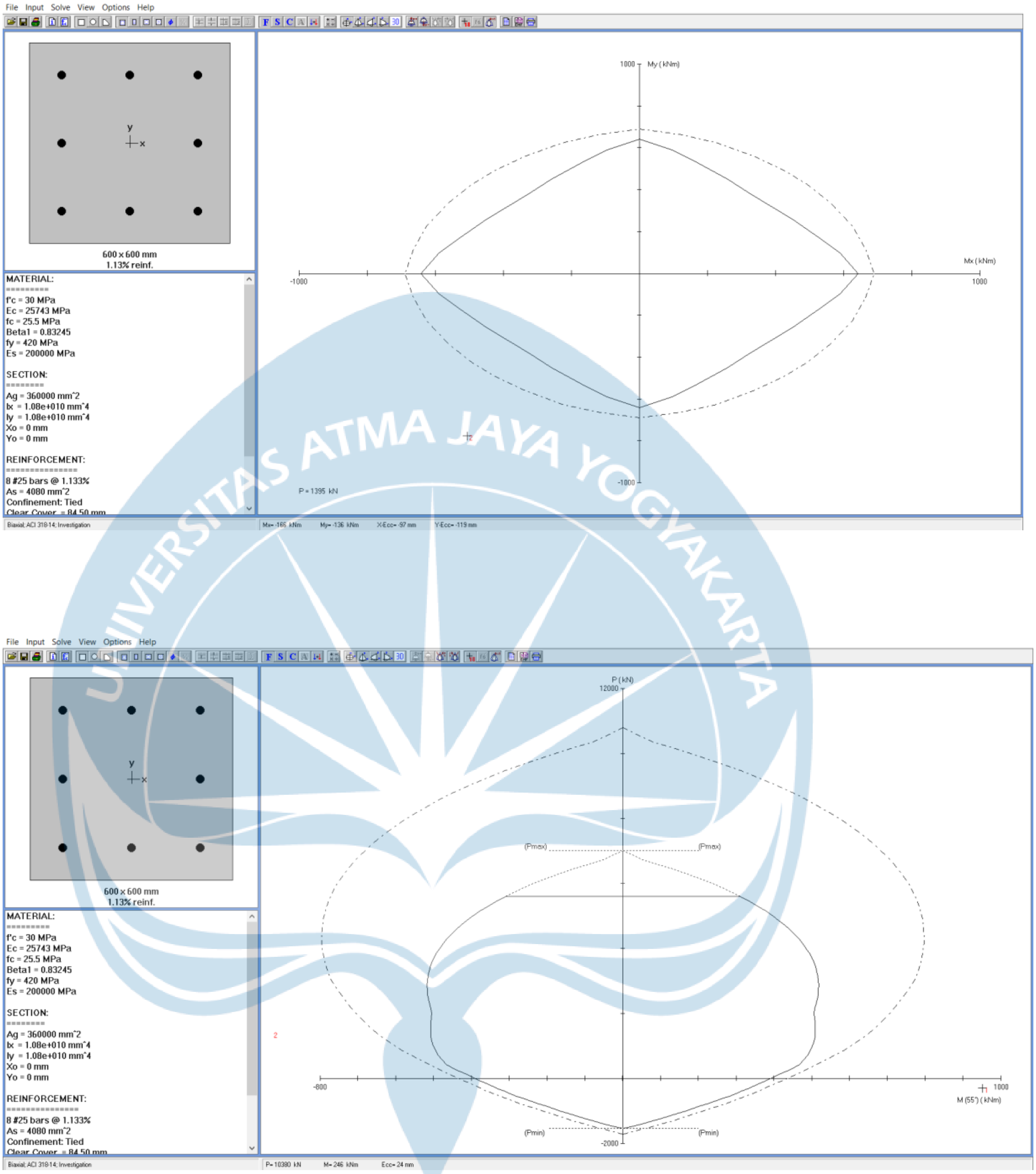


Figure 2.22 Result SPColumn of Structural Column 60 x 60

2.10 Conclusion

In the design of the roof, the purlin used is a C200 x 75 x 20 profile, the sag-rod used has a diameter of 10 mm, the trusses use the IWF profile 600 x 600 x 6 x 10 and 650 x 650 x 6 x 10. The roof profile is safe against deflection and stress.

The Stair for 1st and 2nd building with a size of 1980 x 4500 mm and a height of 3m. The steps have a height (Optrede) of 180 mm and a width (Antrede) of 290, with thickness 150 mm. The stair consists of Pedestal reinforcement D16-300, and Field reinforcement D13-200.

The 1st and 2nd Building have 4 types which are divided into type A, type B, type C, and type D which differ in the size of the slab itself. Floor slab reinforcement includes field and support reinforcement that has complied with the safety requirements for floor slabs. Field reinforcement for the entire floor uses reinforcement with size D13 with a spacing of 300, as well as reinforcement for all floors using reinforcement with the same size, using reinforcement D10 with a spacing of 300.

The beam consists of a main beam 40 x 55 and main beam 40x50, which has main pedestal reinforcement 5D25 with 2P13-150 as stirrups, and main field reinforcement 6D25 with 2P13-150 as stirrups as support reinforcement. For Secondary beam measuring 25 x 35 which has main pedestal reinforcement 3D25 with 2P13-150 as stirrups, and main field reinforcement 3D25 with 2P13-150 as stirrups as support reinforcement.

Columns on 1st and 2nd Building with a size of 60 x 60 cm with a bending diameter of 25 mm with a shear reinforcement diameter of 8 mm and a concrete cover of 40 mm with a concrete quality of 25 Mpa and Fy 420. Use Main reinforcement 8D25, with stirrups 2P13-150.