between floors that is caused by the design earthquake, for each of the structure's orthogonal axes. The divergence between floors must be less than the allowable deviation of floors, with the purpose being to limit the occurrence of exaggerated melting of steel and fracturing of concrete, preventing damage to the non-structural elements, and preventing drifting that could disturb the occupants of the building.

1.6.4 Ultimate Limit Performance

The safety of the structure to support the ultimate design load is determined by the ultimate limit performance. To assess the ultimate limit performance, the ultimate load combination is applied to determine the maximum internal forces that take place in the structural elements. These internal forces would be used in the design process of the building's structural elements including the columns, beams, slabs, foundations, etc.

CHAPTER II
YOUTH CENTER SUPERSTRUCTURE DESIGN

2.1 General Description of Structure

Superstructure is every structural component in the building that is built above the ground. In the Bengkulu Youth Center project, the superstructure consists of the roof structure, columns, beams, slabs, stair components, and the joints that connect the beams and columns. The superstructure is mainly composed of steel-reinforced concrete, with the roof components, such as the purlins and trusses, being made of steel.

The layout of the building as seen from the top view is irregular. The building consists of two main parts which can be designated the north and south sections of the youth center. The north section includes three stories comprising the communal rooms, activity rooms, working spaces, as well as the staff office space. The south section is two stories tall and houses the auditorium space and its surrounding features. These two sections each have a hip roof that is identical to each other, and the sections are connected by an eight-meter wide and two-story high corridor.

Viewing the building from a side view, the two sections can be seen clearly. It shows that the building is relatively wide especially compared to its height. It also clearly shows the vertical irregularities of the building, that are mainly caused by the difference in floor area between the stories of the building. As a result, these irregularities must be considered during the structural analysis of the building to ensure safety and compliance with standards.

The selection of the site class will serve as the basis for the superstructure's design. Next, a preliminary design is created using modeling software like MIDAS Gen to determine the internal forces resulting from various predicted loading inputs. The output of internal forces is then examined using a variety of standards and specifications to evaluate the durability of the structural design. Following are the rules that were employed in this design:

– 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.

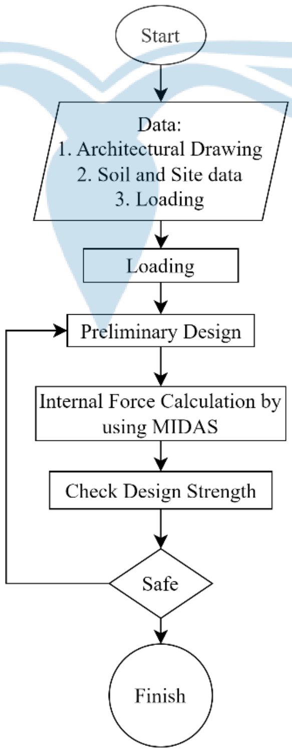
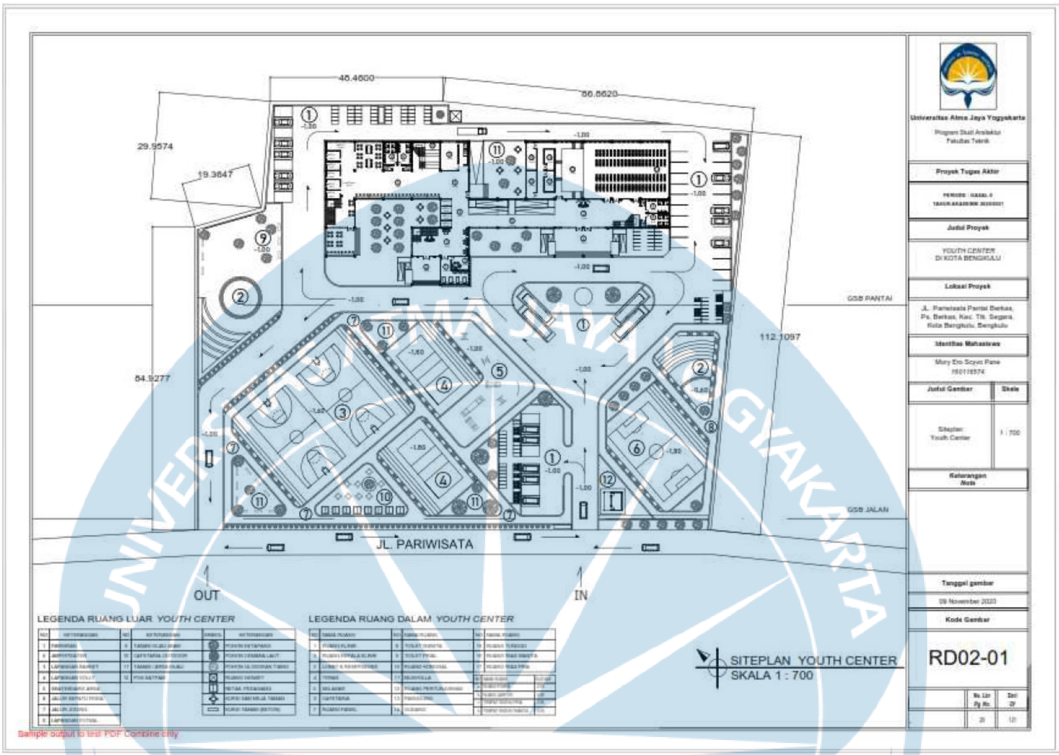
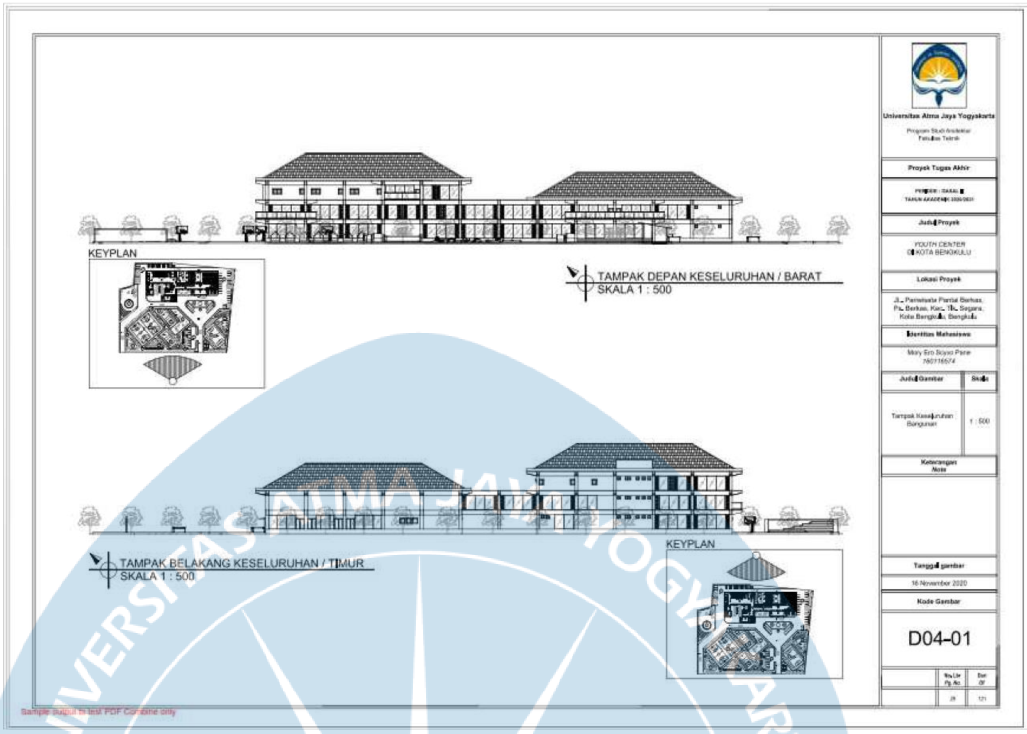


Figure 2. 1 Flowchart of Planning Super Structure

2.2 Project Description



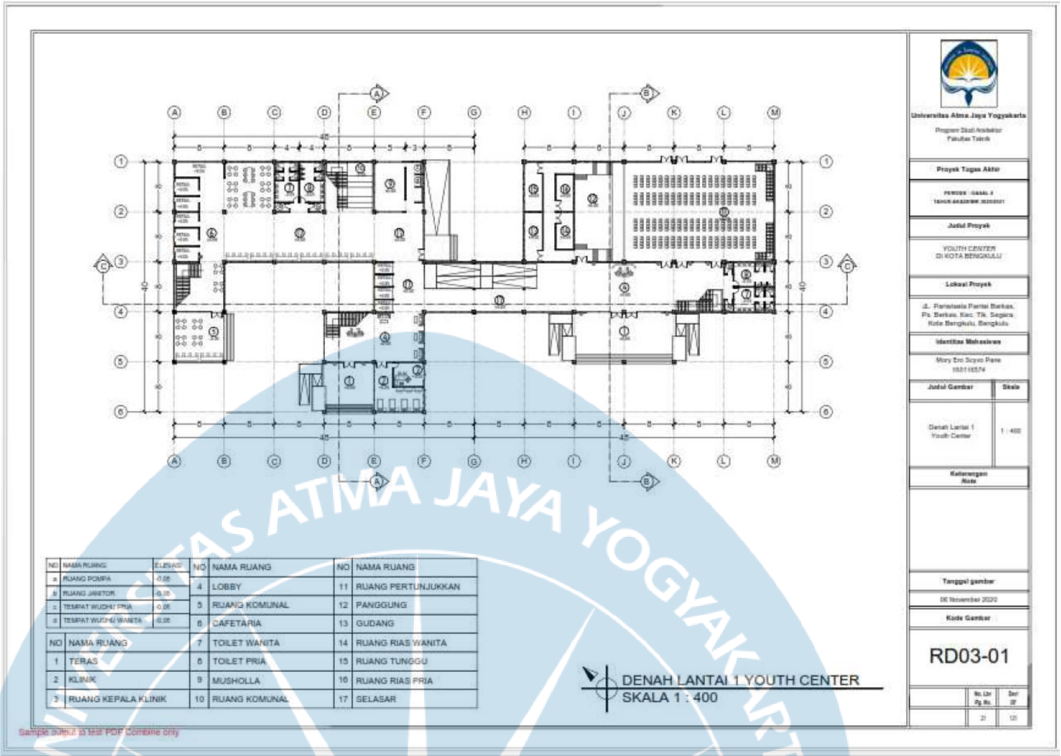
Located in the heart of Bengkulu City, the newly envisioned Youth Center stands tall as a beacon of community engagement and empowerment. Spanning three stories, this vibrant hub is set to become a dynamic space where the energy of youth converges with opportunities for growth and development. The Building will be 40m wide and 90m long.



Perched prominently in the heart of Bengkulu City, the envisioned Youth Center emerges as a transformative architectural marvel, standing at three stories tall. A unique feature distinguishes this center—the third floor is exclusively designed for educational resources and career development services, creating a dedicated space for the intellectual and professional growth of the city's youth. Cleverly, this floor occupies only half of the building, maintaining a harmonious balance with the remaining two stories.

The third floor serves as a beacon of educational excellence, boasting well-appointed classrooms, counseling spaces, and collaborative zones, all meticulously crafted to inspire and facilitate the exploration of untapped potential. Meanwhile, the other half of the structure, comprising two stories, seamlessly complements this focus, offering versatile multipurpose rooms, recreational spaces, and areas for community engagement.

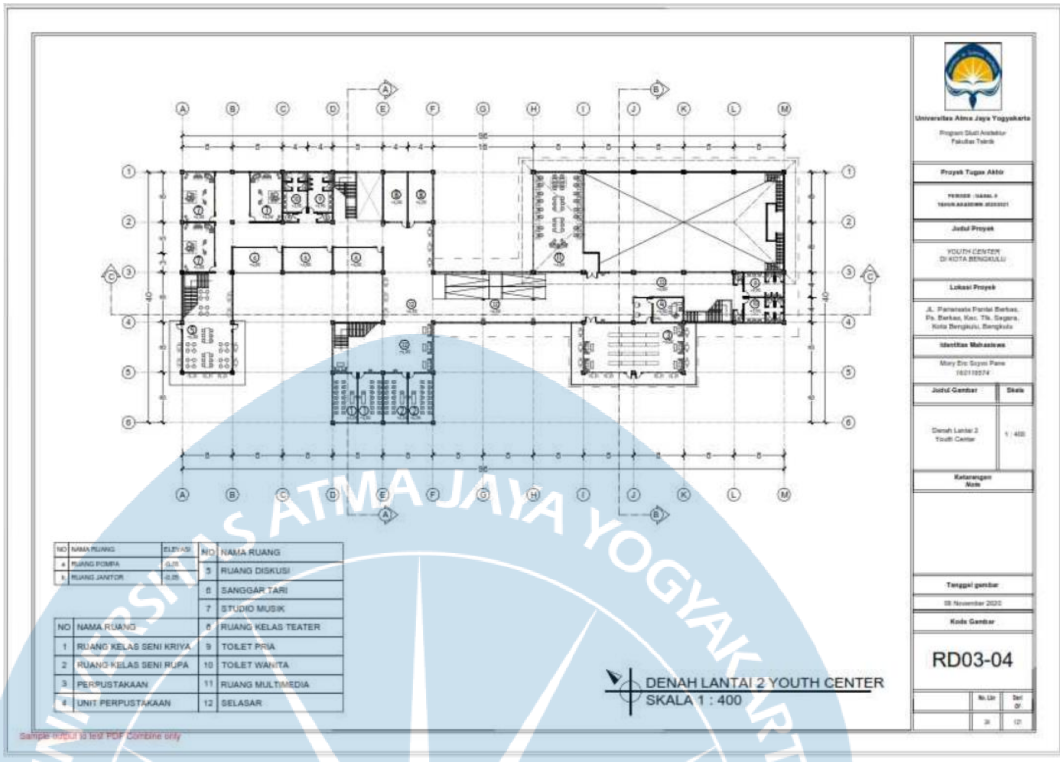
2.2.1 1st Story



The inaugural story of the proposed Youth Center in Bengkulu City is meticulously planned to embody an innovative and functional design. At the core of this floor lies the main atrium, a central space radiating with the energy of community engagement and interaction.

The atrium serves as a vibrant nucleus, connecting various areas within the building and providing a focal point for gatherings and events. Complementing the atrium, the ground floor is thoughtfully divided to include two strategically positioned bathrooms, ensuring convenience and accessibility for visitors. These facilities are designed to meet the highest standards of comfort and cleanliness, enhancing the overall experience for everyone utilizing the Youth Center..

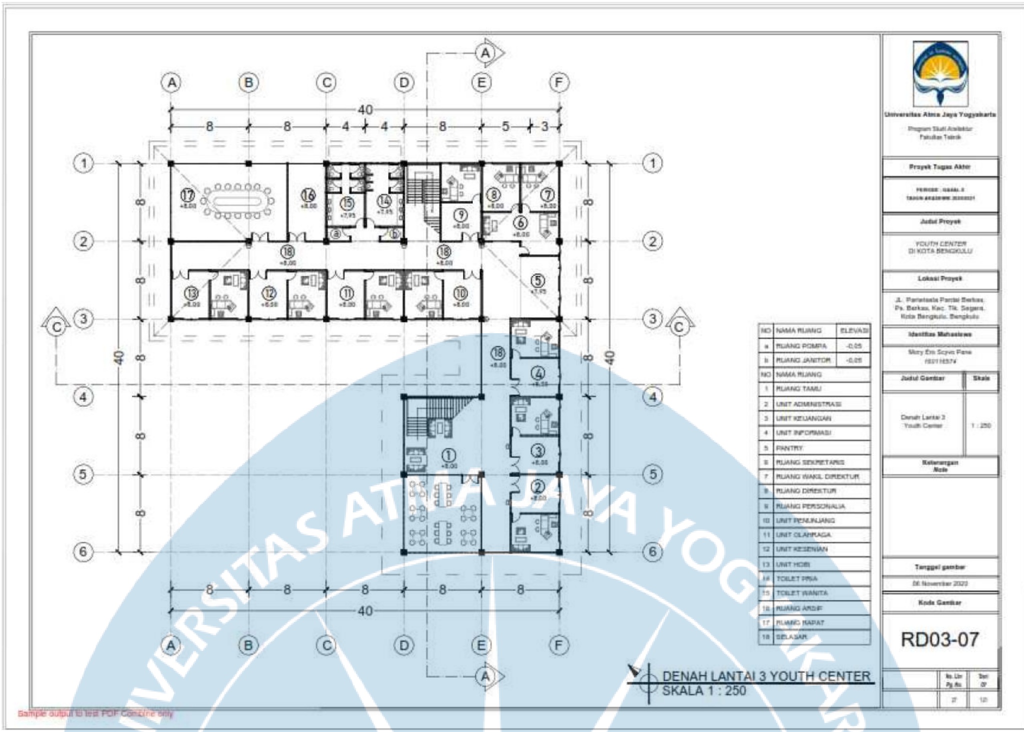
2.2.2 2nd Story



The architectural vision for the Youth Center's 2nd story building is centered around an innovative and purposeful floor plan. As one ascends, the second floor unfolds into a dynamic space dedicated to fostering the holistic development of the youth community. This carefully designed level is strategically curated to cater to a myriad of needs, primarily focusing on recreational activities, skill-building initiatives, and the nurturing of creative expression.

Upon entering the second floor, visitors are greeted by an inviting layout that seamlessly combines open spaces with designated zones for various recreational pursuits. The area is thoughtfully equipped with modern amenities that not only elevate the aesthetic appeal but also serve as facilitators for social interaction. Comfortable lounges, collaborative workspaces, and vibrant common areas create an atmosphere conducive to building connections and fostering a sense of community among the youth

2.2.3 3rd Story



The Elevating the landscape of Bengkulu City, the envisioned Youth Center emerges as a distinctive three-story structure, seamlessly blending modern design with a commitment to community development. Ingeniously, the third floor is devoted exclusively to educational resources and career development services, making it a focal point for the intellectual and professional growth of the city's youth. The meticulously planned floor layout of the third story, occupying only half of the building, ensures an optimal environment for diverse learning experiences and skill acquisition.

This educational haven offers a variety of thoughtfully designed spaces, including well-equipped classrooms, counseling rooms, and collaborative study areas, all aimed at nurturing young minds and encouraging the exploration of their full potential. With a strategic location at its core, the Youth Center becomes a dynamic hub for inspiration, education, and collaboration.

The remainder of the building, comprising two stories, seamlessly complements the educational focus of the third floor. These levels house multipurpose rooms for community events, recreational spaces for fostering social connections, and areas designed to accommodate various youth-centric activities.

This holistic approach ensures that the Youth Center serves as a comprehensive resource for the local community.

2.3 Soil Interpretation Data and Site Class Determination

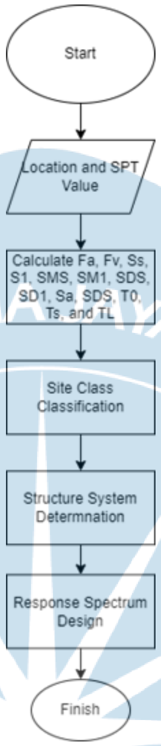


Figure 2. 2 Flowchart of Soil Interpretation Data

The Standard Penetration Test bore log, which was provided to reflect the site's soil data, is the foundation for the soil interpretation. SNI 1726:2019 is the reference standard used to interpret the data. Figure 2. 2 depicts the method for interpreting soil data.

In SNI 1726:2019 Table 3, the risk category for buildings may be shown. The four risk categories are I, II, III, and IV. This building, which is an apartment, is under risk category II. The building risk category is used to estimate the earthquake priority factor. According to SNI 1726:2019 table 4, the earthquake priority factor (Ie) is displayed. Because the residential home's structure falls under building risk category III in this work, the earthquake priority factor (Ie) is 1.25. Soil characteristics at the project location have an impact on the ground acceleration parameters (Ss and S1).

The acceleration of the MCER earthquake at ground level is determined by the values of Ss and S1, where Ss and S1 are the acceleration's spectral response parameters for short intervals and 1.0 second periods, respectively. Based on the structure's location, the values for SS and S1 are 1.5 and 0.600, respectively, and can be found on Spectra Design Indonesia's official website, rsa.ciptakarga.pu.go.id. All of these factors are displayed in Table 2. 1.

Table 2. 1 Risk Parameter of Building Site

Risk Category	III
Ie	1.25
Ss	1.5
S1	0.600

Table 2. 2 Class Site Classification

Depth (m)	**Material**	**SPT N Value**	**Thickness (m)**	**Thickness/ N**
2	Coarse Sand	34	2	0.05882
4	Coarse Sand	49	2	0.04082
6	Coarse Sand	51	2	0.03922
8	Coarse Sand	52	2	0.03846
10	Coarse Sand	52	2	0.03846
12	Coarse Sand	54	2	0.03704
14	Coarse Sand	55	2	0.03636
16	Coarse Sand	55	2	0.03636
18	Coarse Sand	57	2	0.03509
20	Dense Sand	59	2	0.03390
22	Dense Sand	60	2	0.03333
24	Dense Sand	60	2	0.03333
26	Dense Sand	60	2	0.03333
28	Dense Sand	60	2	0.03333
30	Dense Sand	60	2	0.03333
32	Dense Sand	60	2	0.03333
34	Dense Sand	60	2	0.03333
36	Dense Sand	60	2	0.03333
38	Dense Sand	60	2	0.03333
40	Dense Sand	60	2	0.03333
Σ			40	0.72786
$\bar{N}$ch= ΣT/Σ(T/N)	54.95545069	Site Class SC		

2.4 Structure System Determination

According to SNI 1726:2019 tables 8 and 9, the intended structure will be classified as belonging to the design category seismic (KDS) based on the relationship between SDS, SD1, and KDS. That table indicates that the seismic design for SDS and SD1 is class C.

Table 2. 3 Parameters of Structure

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

2.5 Structural Factored Load

Structural loading is the weight or force applied to a structural structure. It is divided into five categories: dead load, additional dead load, live load, wind load, and earthquake load. The construction needs to be strong enough and able to withstand this weight. Therefore, a load analysis is essential to figuring out the maximum load that a structure can support. Written in compliance with SNI 1727:2013 is the load.

2.5.1 Combination of Factored Load

The ultimate load combinations are established using the Auto-Generation feature from MIDAS GEN Software according to the regulations outlined in ACI 318-19, which are the following:

cLCB1: 1.4(D)

cLCB2: 1.2(D) + 1.6(L)

cLCB3: 1.2(D) + 1.0(1.0(1.00)RX+0.3(1.00)RY) + 1.0(L)

cLCB4: 1.2(D) + 1.0(1.0(1.00)RX-0.3(1.00)RY) + 1.0(L)

cLCB5: 1.2(D) + 1.0(1.0(1.00)RY+0.3(1.00)RX) + 1.0(L)

cLCB6: 1.2(D) + 1.0(1.0(1.00)RY-0.3(1.00)RX) + 1.0(L)

cLCB7: 1.2(D) - 1.0(1.0(1.00)RX+0.3(1.00)RY) + 1.0(L)

cLCB8: 1.2(D) - 1.0(1.0(1.00)RX-0.3(1.00)RY) + 1.0(L)

cLCB9: 1.2(D) - 1.0(1.0(1.00)RY+0.3(1.00)RX) + 1.0(L)

cLCB10: 1.2(D) - 1.0(1.0(1.00)RY-0.3(1.00)RX) + 1.0(L)

cLCB11: 0.9D

cLCB12: 0.9(D) + 1.0(1.0(1.00)RX+0.3(1.00)RY)

cLCB13: 0.9(D) + 1.0(1.0(1.00)RX-0.3(1.00)RY)

cLCB14: 0.9(D) + 1.0(1.0(1.00)RY+0.3(1.00)RX)

cLCB15: 0.9(D) + 1.0(1.0(1.00)RY-0.3(1.00)RX)

cLCB16: 0.9(D) - 1.0(1.0(1.00)RX+0.3(1.00)RY)

cLCB17: 0.9(D) - 1.0(1.0(1.00)RX-0.3(1.00)RY)

cLCB18: 0.9(D) - 1.0(1.0(1.00)RY+0.3(1.00)RX)

cLCB19: 0.9(D) - 1.0(1.0(1.00)RY-0.3(1.00)RX)

The service load combinations are established using the Auto-Generation feature from MIDAS GEN Software according to the regulations outlined in ACI 318-19, which are the following:

cLCB20 SERV :(D)

cLCB21 SERV :(D) + L

cLCB22 SERV :(D) + 0.7(1.0(1.00)RX+0.3(1.00)RY)

cLCB23 SERV :(D) + 0.7(1.0(1.00)RX-0.3(1.00)RY)

cLCB24 SERV :(D) + 0.7(1.0(1.00)RY+0.3(1.00)RX)

cLCB25 SERV :(D) + 0.7(1.0(1.00)RY-0.3(1.00)RX)

cLCB26 SERV :(D) - 0.7(1.0(1.00)RX+0.3(1.00)RY)

cLCB27 SERV :(D) - 0.7(1.0(1.00)RX-0.3(1.00)RY)

cLCB28 SERV :(D) - 0.7(1.0(1.00)RY+0.3(1.00)RX)

cLCB29 SERV :(D) - 0.7(1.0(1.00)RY-0.3(1.00)RX)

cLCB30 SERV :(D) + 0.75L + 0.75(0.7(1.0(1.00)RX+0.3(1.00)RY))

cLCB31 SERV :(D) + 0.75L + 0.75(0.7(1.0(1.00)RX-0.3(1.00)RY))

cLCB32 SERV :(D) + 0.75L + 0.75(0.7(1.0(1.00)RY+0.3(1.00)RX))

cLCB33 SERV :(D) + 0.75L + 0.75(0.7(1.0(1.00)RY-0.3(1.00)RX))

cLCB34 SERV :(D) + 0.75L - 0.75(0.7(1.0(1.00)RX+0.3(1.00)RY))

cLCB35 SERV :(D) + 0.75L - 0.75(0.7(1.0(1.00)RX-0.3(1.00)RY))

cLCB36 SERV :(D) + 0.75L - 0.75(0.7(1.0(1.00)RY+0.3(1.00)RX))

cLCB37 SERV :(D) + 0.75L - 0.75(0.7(1.0(1.00)RY-0.3(1.00)RX))

cLCB38 SERV :0.6D

cLCB39 SERV :0.6(D) + 0.7(1.0(1.00)RX+0.3(1.00)RY)

cLCB40 SERV :0.6(D) + 0.7(1.0(1.00)RX-0.3(1.00)RY) cLCB41 SERV :0.6(D) + 0.7(1.0(1.00)RY+0.3(1.00)RX)

cLCB42 SERV :0.6(D) + 0.7(1.0(1.00)RY-0.3(1.00)RX)

cLCB43 SERV :0.6(D) - 0.7(1.0(1.00)RX+0.3(1.00)RY)

cLCB44 SERV :0.6(D) - 0.7(1.0(1.00)RX-0.3(1.00)RY)

cLCB45 SERV :0.6(D) - 0.7(1.0(1.00)RY+0.3(1.00)RX)

cLCB46 SERV :0.6(D) - 0.7(1.0(1.00)RY-0.3(1.00)RX)

2.5.2 Dead Load (Self Weight)

The "self-weight" or "dead load" of a building is the total weight of all the structural elements that comprise its primary structure, including floor slabs, beams, columns, and so on. The size of the object multiplied by its unit weight is the main method used to determine dead load. When using Midas software to model a structure, the self-weight of the structure is automatically calculated by the program based on the density of the material and the dimensions of the structural components that are entered into the program.

2.5.3 Additional Dead Load

Additional deadload refers to the extra weight or mass that is added to a structure or building beyond its basic or initial design requirements. Dead load refers to the permanent weight of the structure itself, including its foundation, walls, floors, roofs, and any fixed equipment or fixtures. However, during the lifespan of a structure, there may be circumstances where additional weight or loads need to be considered for design and safety purposes.

2.5.4 Live Load

Live load refers to the temporary or moving loads that are applied to a structure or building during its normal use. Unlike dead load, which is the permanent weight of the structure itself, live loads are dynamic and can vary in magnitude and location. Live loads are typically caused by people, furniture, equipment, vehicles, or any other movable objects that occupy or use the structure.

2.5.5 Wind Load

Wind load refers to the force exerted by the wind on a structure or building. Wind is a dynamic and variable force that can exert pressure on the surfaces and components of a structure, causing stress and potentially affecting its stability and structural integrity.

When wind blows against a building, it creates a pressure difference between the windward (facing the wind) and leeward (opposite the wind) sides of the structure. This pressure difference results in a force known as wind load. Wind load can act horizontally, vertically, or laterally on the structure, and it varies depending on factors such as wind speed, direction, turbulence, and the shape and height of the building. The wind load estimation is approximated using the procedure of the code that is presented in SNI 1727:2013 chapter 26.

Table 2. 4 Parameters of Wind Load

Parameter	Value	Unit	SNI 1727:2020
Width of Building, B	16.0	m	
Length of Building, L	96.0	m	
Wall Height	12.0	m	
Soil to Roof Height	15.1	m	
Effective Height, h	13.6	m	
L/B	6.00		
h/L	0.14		
Roof Angle, θ	21.0	o	
Roof Type	Pitched		
Base Wind Velocity, V	32.00	m/s	Chapter 26.5.1
Coefficient Factor of Wind Direction, Kd	0.95		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 26.11
Coefficient of Internal Pressure, (GCpi)	0.18		Table 26.13-1
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	0.78		Table 27.3.1
Velocity pressure, q_z	467.9577	N/m2	Equation 27.3-1

SNI 1727:2013 figure 27.4-1 calculates the coefficient of external pressure Cp*. For the surface of the wall and roof, the coefficient is split. A plus (+) or minus (-) symbol indicates that the wind is approaching the surface, respectively.

Table 2. 5 Coefficient Pressure for Wind Load

Coefficient of External Pressure, Cp*	
Wall Surface	Cp*
Wall side of windward	0.80
Wall side of leeward	-0.20
Edge wall	-0.70
Roof Surface	Cp*
Windward side	-0.30
Leeward side	-0.60

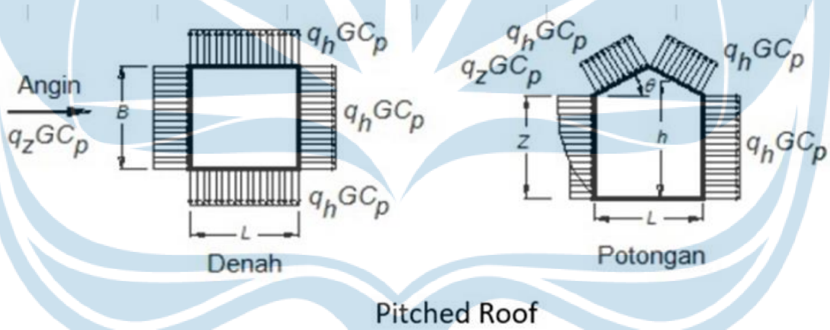


Figure 2. 3 Pitched Roof Wind Load (Source SNI 1727:2013 figure 27.4-1)

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			
Roof Wind, Windward	-203	N/mm2	Chapter 27.4.2 & Equation 27.4.1
Roof Wind, Leeward	-322	N/mm2	Chapter 27.4.2 & Equation 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 , (GC_{Pi})

2.5.6 Earthquake Loading

An earthquake produces a dynamic load that vibrates laterally. As a dynamic load, it can move in both forward and backward directions. Based on the irregularity of the building, SNI 1726:2019 table 16 permits an earthquake analysis. It can be examined using three different methods: time history analysis, response spectrum analysis, and static equivalent. Static equivalent analysis simply takes into account earthquakes as static loads, whereas response spectrum analysis and time history analysis treat earthquakes as dynamic loads. The response spectrum and static equivalent that will be used as the seismic load in the MIDAS simulation are employed in this project's analysis.

A. Response Spectrum Analysis

To determine the spectrum response of the MCER earthquake acceleration imprinted on the soil surface, an amplification factor in the periods of 0.2 seconds (Fa) and 1 seconds (Fv) is required. The amplification factor is computed using the site class and ground acceleration characteristics. Site class, spectral response characteristics of the earthquake acceleration, and a factor of amplification in 0.2 second period (Fa) (value = 1.2). The MCER mapper is active for 0.2 seconds (Ss). The site class and spectrum response earthquake acceleration yield the value of the amplification factor at 1 second period (Fv), which comes out to be 0.56. The site coefficients (Fa and Fv) can be computed by linear interpolation and are derived from SNI 1726:2019 tables 6 and 7, respectively.

Moreover, SNI 1726:2019 eqs. 7 and 8 can be utilized to compute the response spectrum parameters acceleration in short period (SMS) and one second period (SM1) based on the values of the Fa and Fv.

$SMS = Fa \times Ss = 1.5\ g$ (SNI 1726:2019 eq. 7) ....Eq. 2.2

$SM1 = Fv \times S1 = 1.02\ g$ (SNI 1726:2019 eq. 8) ....Eq. 2.3

The design spectral acceleration parameters for the short period 0.2 second (SDS) and period 1 second (SD1) must then be set to 1 based on the SMS and SM1 values in order to create the response spectrum curve. The values of SDS and SD1 are computed using Equations 9 and 10 of SNI 1726:2019.

$SDS = 23 \times SMS = 0.8533\ g$ (SNI 1726:2019 eq. 9) ....Eq. 2.4

$SD1 = 23 \times SM1 = 0.6382\ g$ (SNI 1726:2019 eq. 10) ....Eq. 2.5

The spectra response curve is constructed using the response spectra parameters that were previously calculated, and is based on the response spectrum acceleration and (Sa) and period (T). SNI 1726:2019's equations 11, 12, and 13 are utilized to calculate the response spectrum acceleration. The period is calculated using SNI 1726:2019 figure 3. For "long transition period," or TL, the value is found in SNI 1726:2019 figure 20.

$Sa = SDS\ (0.4 + 0.6\ (\frac{T}{T0}))$ (SNI 1726:2019 eq. 11)

$Sa = \frac{SD1}{T}$ (SNI 1726:2019 eq. 12) .... eq. 2.6

$Sa = \frac{SD2 \times TL}{T2}$ (SNI 1726:2019 eq. 13) .... eq. 2.7

$T0 = 0.2 \times \frac{SD1}{SDS}$ (SNI 1726:2019 figure 3) .... eq. 2.8

$Ts = \frac{SD1}{SDS}$ (SNI 1726:2019 figure 3) .... eq. 2.9

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

Table 2. 7 displays the response spectrum design calculation. Figure 2. 4 displays the response spectrum curve and response spectrum design. The seismic dynamic load calculation from Table 2. 7 will be entered into the MIDAS Model.

Table 2. 7 Design of Response Spectrum

T (s)	Note	Sa (g)	Note
0	0	0.4	SDS*(0.4+0.6*T/T0)
0.1360	T0	1.0000	SDS
0.68	Ts	1.0000	SDS
0.78	Ts + 0.1	0.8718	SD1/T
0.88	Ts + 0.2	0.7727	SD1/T
0.98	Ts + 0.3	0.6939	SD1/T
1.08	Ts + 0.4	0.6296	SD1/T
1.18	Ts + 0.5	0.5763	SD1/T
1.28	Ts + 0.6	0.5313	SD1/T
1.38	Ts + 0.7	0.4928	SD1/T
1.48	Ts + 0.8	0.4595	SD1/T
1.58	Ts + 0.9	0.4304	SD1/T
1.68	Ts + 1	0.4048	SD1/T
1.78	Ts + 1.1	0.3820	SD1/T
1.88	Ts + 1.2	0.3617	SD1/T
1.98	Ts + 1.3	0.3434	SD1/T
2.08	Ts + 1.4	0.3269	SD1/T
2.18	Ts + 1.5	0.3119	SD1/T

2.28	Ts + 1.6	0.2982	SD1/T
2.38	Ts + 1.7	0.2857	SD1/T
2.48	Ts + 1.8	0.2742	SD1/T
2.58	Ts + 1.9	0.2636	SD1/T
2.68	Ts + 2	0.2537	SD1/T
2.78	Ts + 2.1	0.2446	SD1/T
2.88	Ts + 2.2	0.2361	SD1/T
2.98	Ts + 2.3	0.2282	SD1/T
3.08	Ts + 2.4	0.2208	SD1/T
3.18	Ts + 2.5	0.2138	SD1/T
3.28	Ts + 2.6	0.2073	SD1/T
3.38	Ts + 2.7	0.2012	SD1/T
3.48	Ts + 2.8	0.1954	SD1/T
3.58	Ts + 2.9	0.1899	SD1/T
3.68	Ts + 3	0.1848	SD1/T
3.78	Ts + 3.1	0.1799	SD1/T
3.88	Ts + 3.2	0.1753	SD1/T
3.98	Ts + 3.3	0.1709	SD1/T
4.08	Ts + 3.4	0.1667	SD1/T
4.18	Ts + 3.5	0.1627	SD1/T
4.28	Ts + 3.6	0.1589	SD1/T
4.38	Ts + 3.7	0.1553	SD1/T
4.48	Ts + 3.8	0.1518	SD1/T
4.58	Ts + 3.9	0.1485	SD1/T
4.68	Ts + 4	0.1453	SD1/T

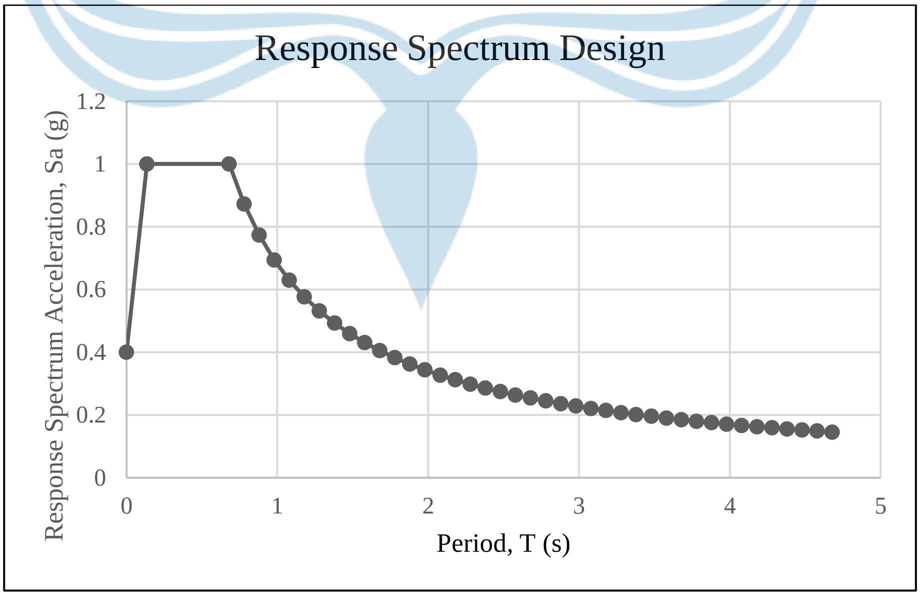


Figure 2. 4 Response Spectrum Graph

B. Static Equivalent Analysis

The two divisions of static equivalent load are X and Y directions. According to this load, an earthquake has a distinct impact on a building's height than other types of loads. Figure 2.5 illustrates how force and displacement rise with height. The parameters for the earthquake static load are displayed in the X and Y directions, respectively, in Tables 2. 8 and 2. 9. The MIDAS model will use this parameter as a static equivalent load input.

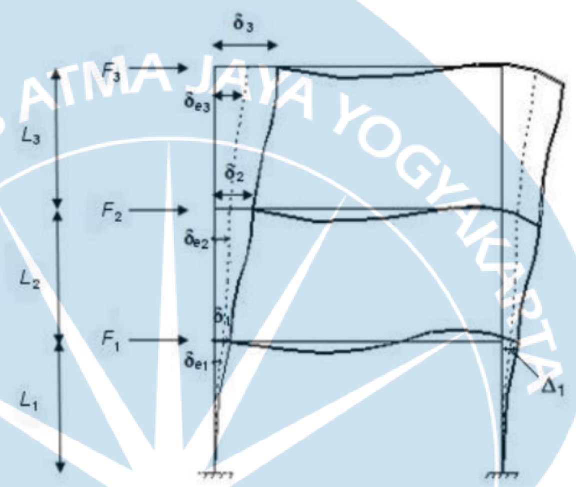


Figure 2. 5 Static Equivalent Load (Source: SNI 1726:2019 Figure 10)

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	1.5
1 Sec Spectral Accel, $S1$	0.6
Long-Period Transition Period, T_L	6
Site Class	C
Fa	1.2
Fv	1.4
SDS	1.2
SD1	0.56
Response Modification, R	8
System Overstrength, Ω	3
Deflection Amplification, Cd	5.5
Occupancy Importance, I	1.25

Table 2. 9 Static Equivalent Parameter in Y Direction

Description	**Value**
Direction and Eccentricity	Y Dir, Y Dir ± Eccentricty
0.2 Sec Spectral Accel, S_s	1.5
1 Sec Spectral Accel, $S1$	0.6
Long-Period Transition Period, T_L	6
Site Class	C
Fa	1.2
Fv	1.4
SDS	1.2
SD1	0.56
Response Modification, R	8
System Overstrength, Ω	3
Deflection Amplification, Cd	5.5
Occupancy Importance, I	1.25

2.6 Structural Modelling

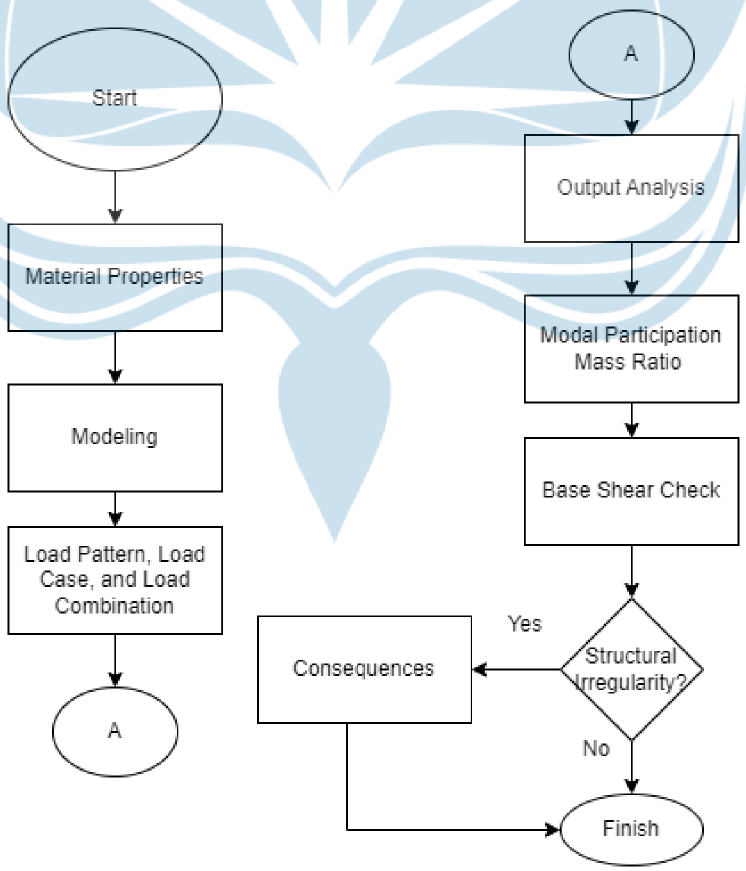


Figure 2. 6 Structural Modelling Flowchart

The procedure of structural modeling is shown in Figure 2. 6. The internal force and displacement of the structure brought on by numerous loads acting simultaneously are estimated using MIDAS software. Structural modeling is helpful in determining the internal forces that exist inside structural components and the behavior of the structure under operating load. Based on the results of the structural modeling, the dimensions section of the required structural elements is created.

a. Material Properties

The materials that are used in the structure is defined in MIDAS Gen as follows:

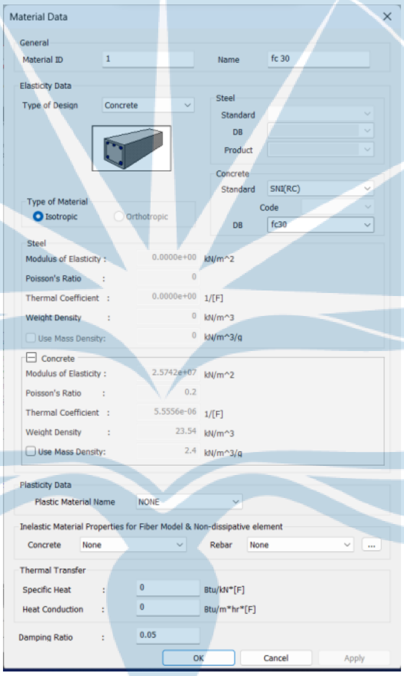


Figure 2. 7 Material Properties for Concrete

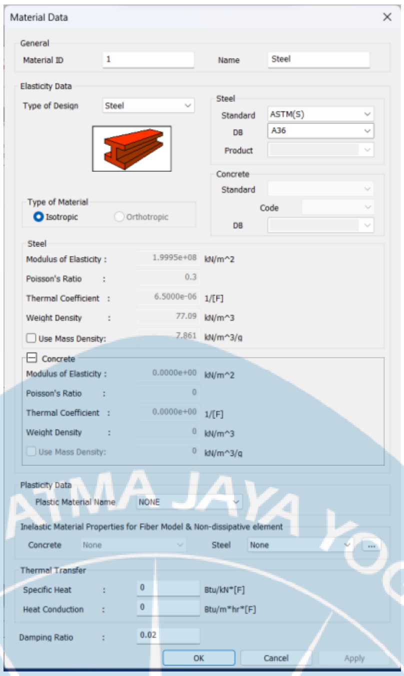
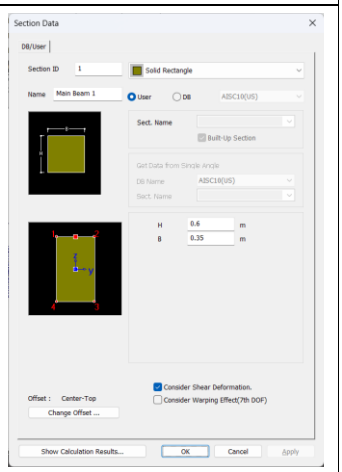


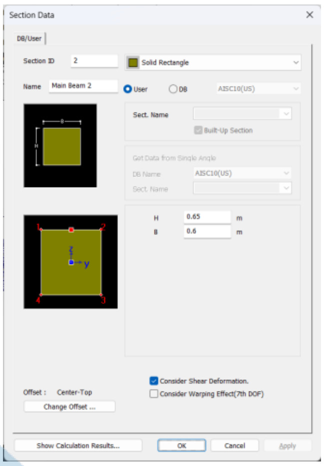
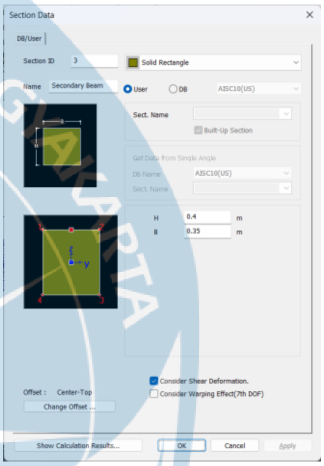
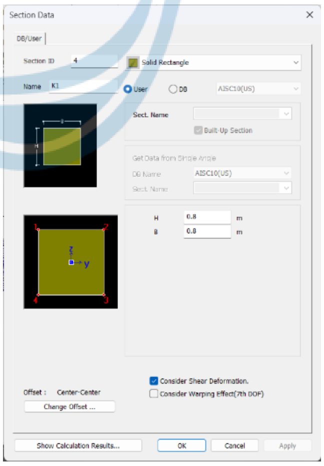
Figure 2. 8 Material Properties for Steel

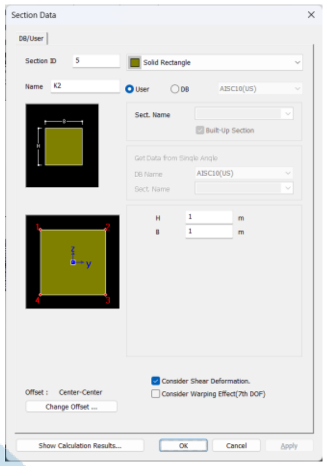
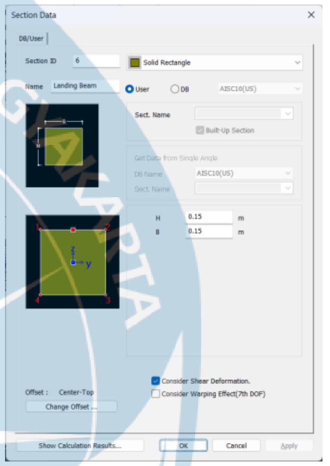
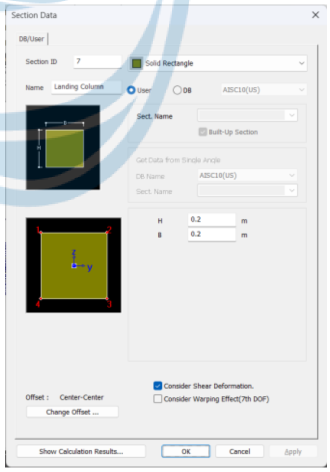
b. Beam, Column, and Slab Section Properties

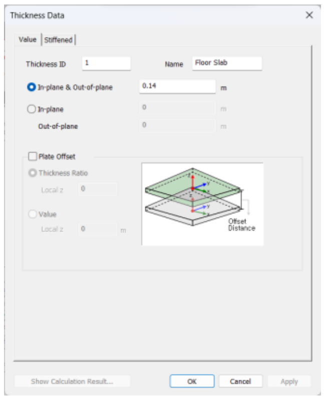
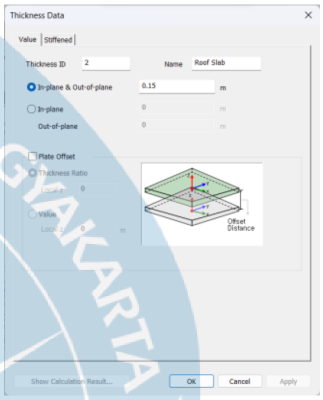
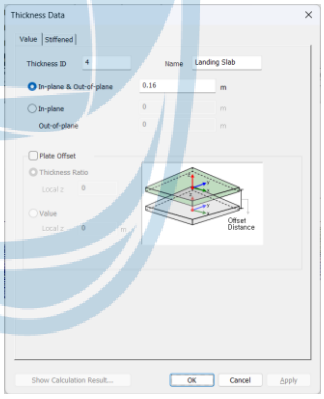
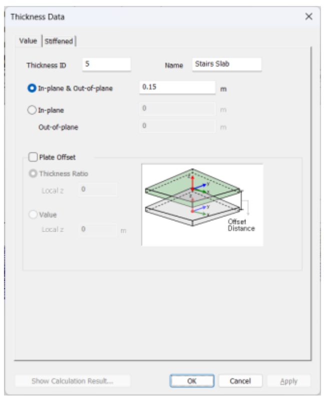
The following are the characteristics of the slab thicknesses used in the structure, as well as the qualities of the beam and column sections. It's also important to note that all of the beam sections have their offset set to "center-top" in order to replicate a genuine concrete structure.

Table 2. 10 Section Properties

No.	Name and Description	Properties in MIDAS Gen
1.	Main Beam 1 35x60 cm main beam This is the primary beam used throughout the entire building except for the parts surrounding the auditorium room.	

2.	Main Beam 2 60x65 cm main beam This is the main beam used around the auditorium area.	
3.	Secondary Beam 35x40 cm secondary beam This secondary beam is used throughout the entire structure.	
4.	Column 1 80x80 cm column This column is used throughout the entire structure except for the area surrounding the auditorium.	rein 

5.	Column 2 100x100 cm column This column is used around the auditorium area.	
6.	Landing Beam 15x15 cm beam This is the beam that supports the landing slabs for all the stairs structures in the building.	
7.	Landing Column 20x20cm column These columns support the landing slabs for all the stairs structures in the building.	

8.	Floor Slab 14cm thick slab This is used as the floor slab throughout the structure.	
9.	Roof Slab 15cm thick slab This is used as the roof slab for the parts of the roof without the hip roof.	
10.	Landing Slab 16cm thick slab This is used as the landing slabs for the stairs in the structure.	
11.	Stairs Slab 15 cm thick slab This is used as the stairs slab in the structure.	

Structural modeling can take into consideration the impacts of steel damage during an earthquake by reducing the moment of inertia of the cross section of the structural elements. The slab's moment of inertia is equal to 25% of the moment's starting inertia. In beam structural components, the moment of inertia is reduced to 35 percent of its initial value. When in the column, the moment of inertia decreases to 70% of its starting value. The left, right, front, and plan views of the building model are displayed in Figures 2. 7 through 2. 8 and Figures 2. 9 through Figure 2. 10, respectively.

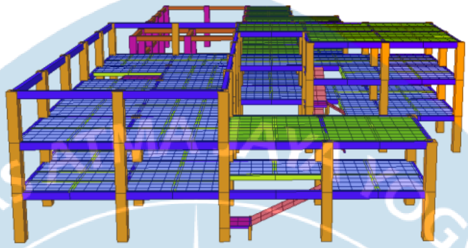


Figure 2. 9 Building Model Left View

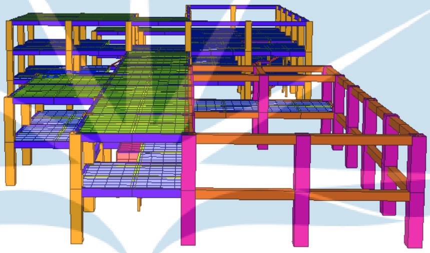


Figure 2. 10 Building Model Right View

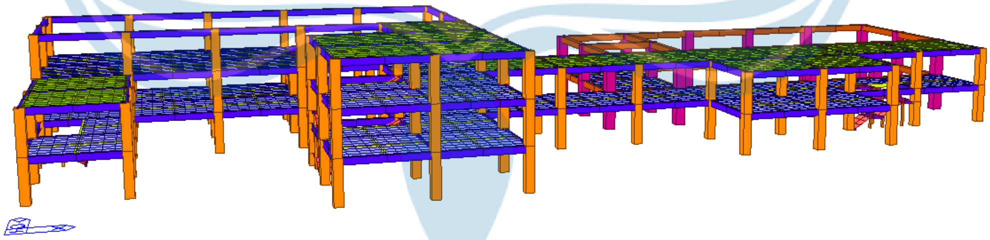


Figure 2. 11 Building Model Front View

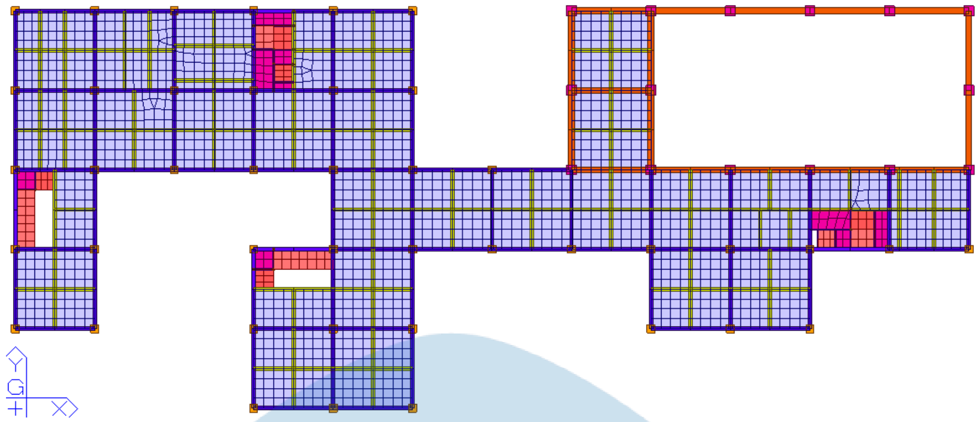


Figure 2. 12 Second Story Plan View

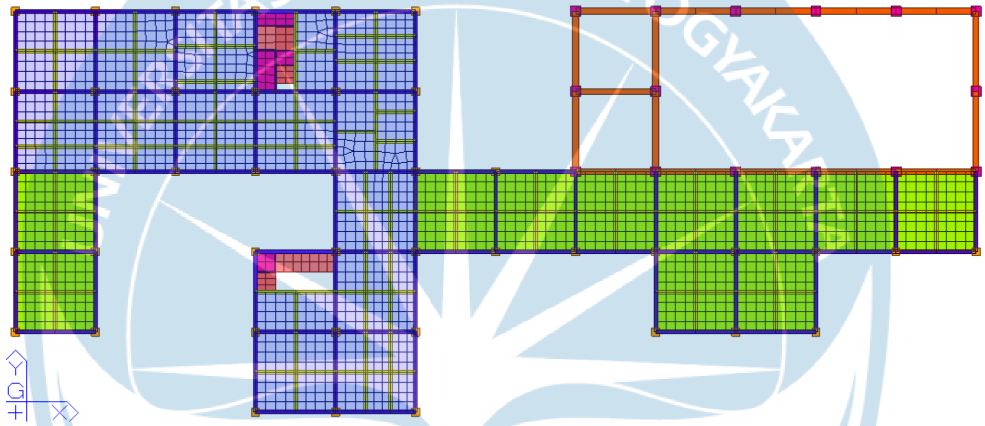


Figure 2. 13 Third Story Plan View

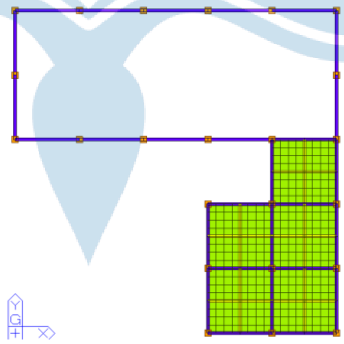


Figure 2. 14 Fourth Story Plan View

2.7 Evaluating Structural Systems Output and Structural Irregularities

2.7.1 Modal Mass Ratio Check

The capital involvement mass ratio derived from structural modeling results is presented in Table 2. 11. The number of modes (modes) required to ascertain the natural vibrational variation for the structure must be sufficient to achieve a combined mass participation up to 100% of the actual mass of each x and y direction of the response taken into consideration by the model, per Article 7.9.1 of SNI 1726:2019. The results of structural modeling show that for both directions involving 60 modes of vibration, the total mass participation mass ratio in both the X and Y directions is very close to The table presented in section 11 summarizes the capital involvement mass ratio determined through structural modeling. As stipulated in Article 7.9.1 of SNI 1726:2019, a sufficient number of vibration modes are necessary to capture the natural vibrational behavior of the structure. This ensures that the combined mass participation ratio in both the X and Y directions, encompassing the model's response, reaches at least 100% of the actual mass. The structural modeling results indicate that for both directions, utilizing 60 vibration modes yields a total mass participation ratio close to 100%.

Table 2. 11 Mass Participating Ratio

Mode No	TRAN-X		TRAN-Y	
	MASS(%)	SUM(%)	MASS(%)	SUM(%)
1	6.7472	6.7472	50.3311	50.3311
2	0.7506	7.4979	0.0641	50.3952
3	1.4701	8.9679	0.2712	50.6664
4	57.1281	66.0961	11.0322	61.6986
5	0.292	66.3881	0.1451	61.8438
6	0.0207	66.4088	0.0038	61.8476
7	0.0203	66.429	0.003	61.8506
8	0.0057	66.4347	0.0453	61.8958
9	0.0623	66.4971	0.0102	61.906
10	0.0332	66.5303	0.0026	61.9086
11	0.0002	66.5305	0.0012	61.9099
12	0.0024	66.5329	0.0034	61.9133
13	0.0002	66.5331	0.001	61.9143
14	0.0405	66.5736	0.0051	61.9194
15	0.0784	66.6519	0.0045	61.9239
16	0.0172	66.6692	0.0011	61.9249
17	0.0808	66.7499	0.0113	61.9362
18	0.0005	66.7504	0.0046	61.9408
19	0.169	66.9194	0.0181	61.9589
20	0	66.9195	0.0005	61.9594
21	0.0019	66.9214	0.0002	61.9596

22	0.0039	66.9252	0	61.9596
23	0.0034	66.9287	0.0001	61.9597
24	0.0219	66.9506	0.0119	61.9716
25	0.0034	66.954	0.0009	61.9725
26	0	66.954	0	61.9726
27	0.0144	66.9684	0.0037	61.9763
28	0.0039	66.9722	0.0091	61.9854
29	0.0097	66.982	0.0115	61.9969
30	0.0012	66.9832	0.0061	62.003
31	0.0643	67.0475	0.001	62.0041
32	0.302	67.3495	0.0069	62.011
33	0.0125	67.3621	0.0365	62.0475
34	0.0668	67.4289	0.0213	62.0687
35	0.0027	67.4315	0.0001	62.0688
36	0.0117	67.4433	0.1172	62.186
37	0.0007	67.4439	0.0023	62.1883
38	0.0047	67.4487	0.0046	62.1929
39	0.021	67.4697	0.0028	62.1957
40	0.0362	67.5059	0.0942	62.2899
41	0.2378	67.7437	0.4844	62.7743
42	0.0232	67.7668	0.0087	62.783
43	0.0159	67.7827	0.166	62.949
44	5.0972	72.88	8.7707	71.7197
45	1.2069	74.0868	2.5107	74.2305
46	0.0005	74.0873	0.0002	74.2307
47	0	74.0873	0.0032	74.2339
48	0.0247	74.112	0.0158	74.2497
49	0.0061	74.1181	0.0043	74.2539
50	0.3199	74.438	0.0109	74.2648
51	0.0088	74.4468	0.0043	74.2691
52	0.0009	74.4477	0.0003	74.2695
53	0.0845	74.5321	0.0006	74.2701
54	0.0128	74.545	0	74.2701
55	0.0059	74.5509	0.0238	74.2939
56	0.0014	74.5523	0.0042	74.2981
57	0.0034	74.5557	0.0038	74.302
58	0.0194	74.5752	0.0492	74.3512
59	0.0023	74.5775	0.0036	74.3548
60	0.0001	74.5776	0.0057	74.3605
61	0.0193	74.5968	0.0018	74.3623
62	0.0002	74.597	0.0005	74.3628
63	0.0107	74.6077	0.0036	74.3665
64	0.8529	75.4606	1.1502	75.5167
65	2.0233	77.4838	2.6671	78.1838
66	0.0005	77.4843	0.02	78.2038

67	0	77.4843	0.0099	78.2137
68	0.2154	77.6997	0.5433	78.757
69	0	77.6998	0.0005	78.7575
70	0.0041	77.7038	0.0073	78.7649
71	0.0421	77.7459	0.3401	79.105
72	0	77.7459	0	79.105
73	3.1154	80.8613	5.1019	84.207
74	0.0185	80.8798	0.0351	84.242
75	0.0014	80.8811	0.0009	84.2429
76	0.0007	80.8819	0.0028	84.2457
77	0.7806	81.6625	0.2055	84.4512
78	1.2593	82.9218	0.2629	84.7142
79	0.0716	82.9934	0.0142	84.7284
80	0.042	83.0354	0.0175	84.7459
81	0.2115	83.2469	0.0484	84.7943
82	0.0267	83.2736	0.0054	84.7996
83	0.1059	83.3795	0.0189	84.8186
84	0.0084	83.3879	0.0006	84.8192
85	0.0004	83.3884	0	84.8192
86	0.0007	83.3891	0.0001	84.8193
87	0.0008	83.3899	0.0003	84.8196
88	0.1052	83.495	0.0014	84.8211
89	0.0014	83.4964	0	84.8211
90	0.0029	83.4993	0.0007	84.8218
91	0.0253	83.5246	0.0003	84.822
92	0.0804	83.6051	0.0085	84.8305
93	0.0675	83.6726	0.0137	84.8442
94	0.0084	83.681	0.0003	84.8445
95	0.0464	83.7274	0.0088	84.8532
96	0	83.7274	0.0001	84.8533
97	0.0002	83.7276	0.0014	84.8547
98	0	83.7276	0.0011	84.8557
99	0.0012	83.7288	0.0033	84.8591
100	0.0294	83.7582	0.0023	84.8614

2.7.2 Load Combination Base Shear Check

It is essential to compare the base shear acquired through seismic load analysis using the equivalent lateral force (ELF) with the computed base shear derived from the response spectrum (RS) seismic load analysis. The base shear in the response spectrum (RS) should encompass a minimum of 85% of the equivalent lateral force (ELF) base shear. If this criterion is not satisfied, it is necessary to adjust the force scale factor on the response spectrum (RS). To aid in determining whether a recalculation of the force-scale factor for the response spectrum (RS) is required, the base shear calculation and verification results are presented in the following section. The subsequent value represents the base shear obtained from MIDAS.

Table 2. 12 Base Reaction

SUMMATION OF REACTION FORCES PRINTOUT			
Load	FX (kN)	FY (kN)	FZ (kN)
EXP	-2927.33	0	0
EXN	-2927.33	0	0
EYP	0	-2927.33	0
EYN	0	-2927.33	0
RX(RS)	5114.099	1817.869	154.6291
RY(RS)	1817.869	4608.243	90.89043
RX(ES)	0	0	0
RY(ES)	0	0	0

Scaling is deemed unnecessary as per the MIDAS output, which indicates that the response spectrum variation exceeds 85%. Consequently, the response spectrum will be employed to proceed with the modeling analysis.

2.7.3 Structural Irregularity

During the design phase, Article 7.3.2 of SNI 1726:2019 should be referenced to ascertain whether the structure is irregular or regular. There are two distinct categories of structural imperfections, namely vertical irregularities and horizontal irregularities. For more comprehensive information and a detailed exploration of vertical and horizontal abnormalities, Tables 13 and 14 in SNI 1726:2019 offer additional types and explanations.

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

a. Torsional Irregularity

Torsional irregularity is identified when the maximum story drift (calculated torque plus unexpected torque) at one end of a structure, transverse to the axis, surpasses 1.2 times the average story drift at both ends of the structure. For the 1b torsional irregularity, the threshold is set at 1.4 times, with all other conditions remaining the same. It's important to note that the stipulations for torsional irregularity outlined in the reference articles are applicable specifically to structures with rigid or semi-rigid diaphragms. Table 2 provides a breakdown of the greatest story drifts in relation to average drifts.

Table 2. 13 Torsional Irregularity Check

Story	X Direction		Y Direction	
	$\Delta_{max}/\Delta_{avg}$	Check	$\Delta_{max}/\Delta_{avg}$	Check
3	1.289	H.1a	1.222	H.1a
2	1.219	H.1a	1.593	H.1b
1	1.424	H.1b	1.743	H.1b

b. Interior Angle Irregularity

The inner angle irregularity, identified as the second horizontal irregularity, occurs when the projection dimension in both the x and y directions exceeds 15% of the main portion of the structure. The ratios between the projection dimensions (P) and the total structural dimensions (L) are detailed in Table 2. In this structure, interior angle irregularity is present, as the ratios in both directions exceed 0.15.

Table 2. 14 Interior Angle Irregularity calculation

Lx	96	m
Px	16	m
Ly	40	m
Py	16	m
Lx/Px	0.1667	
Ly/Py	0.4000	
Check	H.2	

c. Stiffness Irregularity

In any grade, a soft-grade stiffness irregularity is evident when the lateral stiffness is less than 70% of the lateral stiffness of the story above or less than 80% of the average stiffness of the three stories above. If the lateral stiffness of a level falls below 60% of the story level above it or less than 70% of the average stiffness of the three levels above it, it is classified as having a significant soft-level stiffness irregularity. The deviations in stiffness noted in this structure are outlined in Table 2.14.

d. Mass Irregularity

Mass irregularity is defined as all levels being effective at more than 150% of the closest effective rate. If the floor beneath a roof is lighter, then there's no need to inspect it. According to MIDAS Gen, Table 2.15 illustrates the weight irregularity identified in the building.

Table 2. 15 Mass Irregularity Check

Story	Level (m)	Story Weight (kN)	Story Weight Ratio	Remark
Roof	12	4225.759	-0.755	Irregular
3F	8	17257.39	-0.053	Regular
2F	4	18223.21	0	Regular
1F	0	2453.329	0	Regular

2.8 Consequence of Irregularity

2.8.1 Consequence of Horizontal Irregularity

This structure exhibits horizontal irregularity as its torsional irregularity. 1.a. Therefore, in accordance with article 11.3.4 for torsional irregularity 1.a or 1.b, the structure must account for accidental eccentricity, which is the assumed displacement of each center of mass from its actual location by a distance equal to 5% of the diaphragm dimension from structure parallel to the direction of mass shift. The eccentricity scale of 0.05 can be entered into the MIDAS Gen for the load scenarios of earthquake spectrum X and earthquake lateral force X.

In accordance to article 7.7.3, to account for the irregularities of 1.a and 1.b, the structural model must be adjusted. In this situation, the structure must be studied using a three-dimensional model. Therefore, the modelling settings in MIDAS Gen was changed to 3-D model to adjust the analysis method.

When torsional irregularities of type 1a or 1b occur as defined in structures designed for seismic design category C, D, E, or F, the effect must be calculated by multiplying Mta at each level by the torque magnification factor (Ax and Ay) in

1726:019 chapter 7.8.4.31. Table 2. 16 and Table 2. 17 show the results of the computation of the torque magnification factor in the X and Y directions, respectively.

$$A_x = \left(\frac{\delta max}{1.2\delta_{avg}}\right)^2$$

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. 16 Torque Magnification Factor in X Direction

Dir. X	$\Delta_{max}/\Delta_{avg}$	A_x	**%**	**Ecc. (m)**
Story4	0.000	0.000	0.0%	0.000
Story3	1.289	1.154	5.8%	2.308
Story2	1.219	1.031	5.2%	2.062
Story1	1.424	1.408	7.0%	2.816

Table 2. 17 Torque Magnification Factor in Y Direction

Dir. Y	$\Delta_{max}/\Delta_{avg}$	A_x	**%**	**Ecc. (m)**
Story4	0.000	0.000	0.0%	0.000
Story3	1.222	1.038	5.2%	4.982
Story2	1.593	1.762	8.8%	8.460
Story1	1.743	2.109	10.5%	10.123

2.9 Consequence of Vertical Irregularity

The structure experiences vertical irregularities 1a, 1b, and 2; however, no consequences are required because of the seismic design category C.

2.10Roof Structure Design

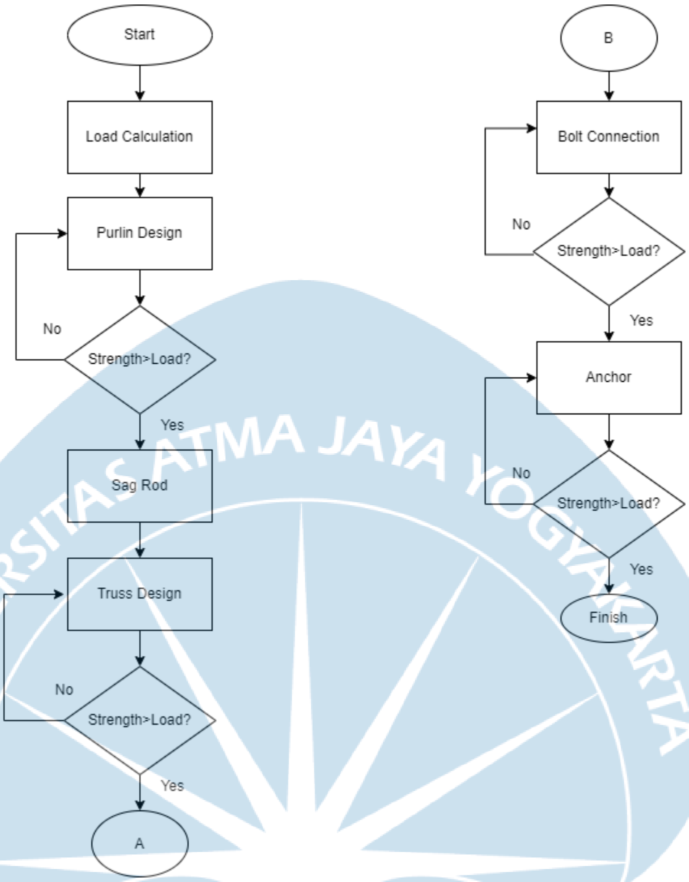


Figure 2. 15 Roof Structure Design Flowchart

Roof design comprises 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 MIDAS Software, and last to check the strength and displacement of structure. Figure 2. 15 illustrates the roof structure design process.

2.10.1 Purlin

The purlin is constructed from C125x50x20x2 steel. Table 2.18 provides the dimensions of the steel, and Table 2.19 presents its properties. The tributary area, where the load acts on the purlin, is calculated in Table 2.20. Figure 2.16 illustrates the plan view of the roof, where the purlin bears the load, subsequently transferred by the roof cover.

Table 2. 18 Purlin Dimension

Description	Value	Unit
Purlin Type	C125x50x20x2	
H	125	mm
B	50	mm
C	20	mm
t	2	mm
L	4	m

Table 2. 19 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	2000	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. 20 Purlin Tributary Area

Tributary Area			
Description	**Value**	**Unit**	**Note**
Tributary Length (Lt)	1.25	m	
Top Purlin	5	m2	$Lt \times L$
Middle Purlin	10	m2	$2 \times Lt \times L$
Bottom Purlin	5	m2	$Lt \times L$
α	21	°	Truss Angle

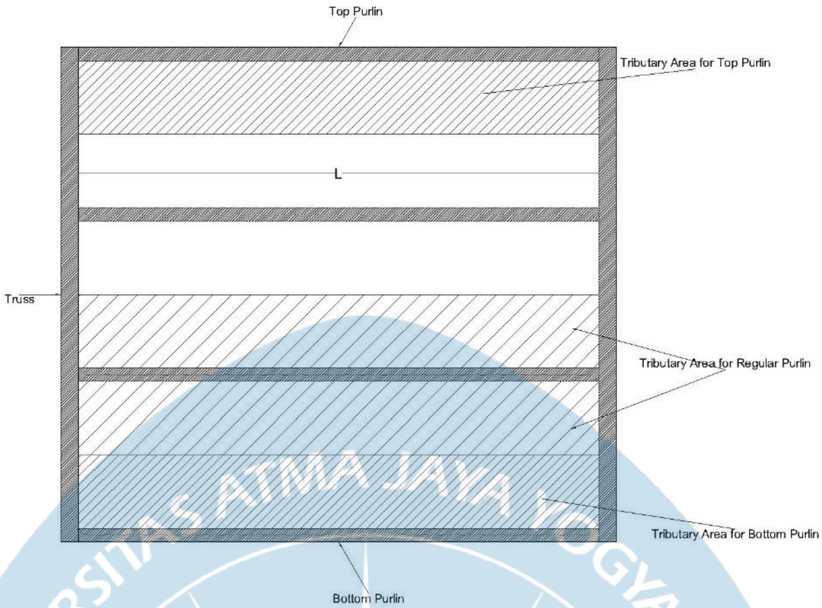


Figure 2. 16 Purlin Tributary Area

a. **Loading**

The loading of purlin for top and bottom can be seen in Table 2. 21 and the loading for middle purlin can be seen in Table 2. 22.

Table 2. 21 Top and Bottom Purlin Load

Description	Value	Unit	Note
Roof Weight	13	kg/m2	Metal
Ceiling Weight	18	kg/m2	
Roof (ADL)	0.65	kN	Roof Weight×Tributary
Ceiling (ADL)	0.9	kN	Ceiling Weight×Tributary
Purlin (DL)	0.158	kN	Purlin Weight×Length
Total Dead Load	1.708	kN	Roof+Ceiling+Purlin
Live Load (LL)	1	kN	SNI
Windward Wind Load (Wt)	-0.203	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)	-1.016	kN	(Wt)×Tributary
Leeward Wind Load (Wh)	-1.6116	kN	(Wh)×Tributary
Take Wind Load (WL)	-1.6116	kN	Max from Wt and Wh

Table 2. 22 Middle Purlin Load

Description	Value	Unit	Note
Roof Weight	13	kg/m2	Metal
Ceiling Weight	18	kg/m2	

Roof (ADL)	1.3	kN	Roof Weight×Tributary
Ceiling (ADL)	1.8	kN	Ceiling Weight×Tributary
Purlin (DL)	0.158	kN	Purlin Weight×Length
Total Dead Load	3.258	kN	Roof+Ceiling+Purlin
Live Load (LL)	1	kN	SNI
Windward Wind Load (Wt)	-0.203	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)	-2.032	kN	(Wt)×Tributary
Leeward Wind Load (Wh)	-3.2231	kN	(Wh)×Tributary
Take Wind Load (WL)	-3.2231	kN	Max from Wt and Wh

Loads on the purlin can be projected in both the X and Y directions. Figure 2.17, titled "Load Projection of D, ADL, and LL," illustrates the load projection of dead load, additional dead load, and live load. The projection of wind load is presented in Figure 2.18.

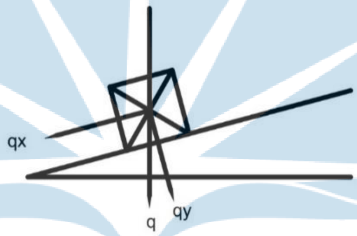


Figure 2. 17 Load Projection of D, ADL, and LL

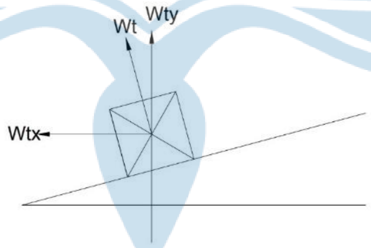


Figure 2. 18 Load Projection of WL

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 for the top and bottom purlin are provided in Table 2.23. The corresponding values for the middle purlin can be found in Table 2.24.

Table 2. 23 Maximum Loading and Moment of Top and Bottom Purlin

Top and Bottom Purlin	**X**	**Comb.**	**Y**	**Comb.**
Load (kN)	1.0191	2	2.8438	2
Moment (kNm)	0.4568	2	1.2748	2

Table 2. 24 Maximum Loading and Moment of Middle Purlin

Middle Purlin	**X**	**Comb.**	**Y**	**Comb.**
Load (kN)	1.6346	1	4.5610	1
Moment (kNm)	0.8173	1	2.2805	1

b. Profile Check

Due to the steel material used in the purlin, the profile must undergo an assessment to determine if it is compact, non-compact, or slender. The calculation is outlined in Table 2.25. The flange is established as non-compact when λ_p ≤ λ ≤ λ_r, and the web is deemed compact when λ ≤ λ_p. Consequently, if one component is compact and the other is non-compact, the overall profile is classified as non-compact.

Table 2. 25 Profile Check for Compact, Non-Compact, and Slender

	λ	**λp**	**λr**	**Note**
Flange	25	10.97	28.86751346	Non-compact
Web	62.5	108.54	164.5448267	Compact

Where the formula of λ, λ_p, and λ_r is presented in Table 2. 26.

Table 2. 26 Formula forλ, λ_p, and λ_r

Element	$\boldsymbol{\lambda}$	$\boldsymbol{\lambda_p}$	$\boldsymbol{\lambda_r}$
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 undergo an evaluation for lateral torsional buckling, which can be determined from the values of length Lb, Lp, and Lr (refer to Table 2.27 and Table 2.28). The profile demonstrates inelastic lateral torsional buckling under a Y-direction load and exhibits no lateral torsional buckling under an X-direction load. As a result, the instantaneous strength varies based on this condition.

Table 2. 27 Purlin Lb, Lp, and Lr for Y Direction

Lb	2000	mm
Lp	970.4103325	mm
Lr	132287.5391	mm

Table 2. 28 Purlin Lb, Lp, and Lr for X Direction

Lb	2000	mm
Lp	2484.453678	mm
Lr	745917.3565	mm

Where:

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

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

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

c. Moment Strength

The moment strength of the purlin, treated as a steel beam, is contingent on the presence or absence of lateral torsional buckling, which can be inelastic, elastic, or exhibit no torsional buckling. The moment calculations are outlined in Table 2.29 and Table 2.30 for the Y direction and X direction, respectively. These tables demonstrate that the factored moment strength (φMn) exceeds the maximum moment (Mu) acting on the purlin.

Table 2. 29 Moment Design Calculation for Y Direction Load

Description	Mn	Unit	Note
Inelastic LTB	7113979.033	Nmm	$L_p < L_b \leq L_r$
The flange is non-compact	3728060.753	Nmm	

Take φMn	3.355254678	kNm	0.9×smallest Mn
Mu for Top and Bottom Purlin	1.274781814	kNm	φMn>Mu Safe
Mu for Middle Purlin	2.280481622	kNm	φMn>Mu Safe

Where:

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

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

Table 2. 30 Moment Design Calculation for X Direction Load

Description	Mn	Unit	Note
No LTB	1815550.035	Nmm	$L_b \leq L_p$
The flange is non-compact	1116653.316	Nmm	
Take φMn	1.004987984	kNm	0.9×smallest Mn
Mu for Top and Bottom Purlin	0.456847462	kNm	φMn>Mu Safe
Mu for Middle Purlin	0.817293946	kNm	φMn>Mu Safe

Where:

Mn for No LTB = $F_y.Z_x$

d. Displacement Check

Purlins, behaving similarly to steel beams, may experience displacement due to loading. According to Segui (2015), a maximum deflection of 1 inch or 25.4 mm is considered acceptable for a steel beam. The deflection calculations are presented in Table 2.31 for the Y direction and Table 2.32 for the X direction, respectively.

Table 2. 31 Deflection at Y Direction

Description	Value	Unit	Note
δmax of Top and Bottom Purin	9.874213134	mm	Safe
δmax of Middle Purlin	15.83667793	mm	Safe
δallowed	25.5	mm	1 inch

Table 2. 32 Deflection at X Direction

Description	Value	Unit	Note
δmax of Top and Bottom Purlin	11.77663384	mm	Safe
δmax of Middle Purlin	18.88857119	mm	Safe
δallowed	25.5	mm	1 inch

Where:

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

2.10.2 Sag Rod

To provide alignment and lateral support for the purlin, a sag rod is utilized. The sag rod helps prevent the purlin from drooping under lateral load and moment. Additionally, on the top of the truss, ties are attached to the purlin, serving the same purpose. The calculations for the sag rod and tie rod are presented in Table 2.33. It is noted that the sag rod and tie rod will employ the smallest diameter permitted according to SNI 1729:2015, which is 10 mm.

Table 2. 33 Sag Rod and Tie Rod Calculation

Sag Rod Calculation			
T	1.019131775	kN	
Ab required	4.896729249	mm2	T/(0.75×0.75×Fu)
Diam. Required	2.496939991	mm	
Use Thread Rod diameter	10	mm	SNI 1729:2015 J3.4
Ab	78.53981634	mm2	
Tie Rod at Ridge			
P	1.019146314	kN	
Ab required	4.896799106	mm2	P/(0.75×0.75×Fu)
Diam. Required	2.496957802	mm	
Use Thread Rod diameter	10	mm	SNI 1729:2015 J3.4
Ab	78.53981634	mm2	

2.10.3 Truss

The truss utilizes a steel profile WF 300x150x6.5x9, and Table 2.34 presents the characteristics of the steel truss. The load on the truss is a compilation of the load from the purlin and the self-weight of the truss itself, as indicated in Table 2.35. In MIDAS Gen, the truss model is depicted in Figure 2.19, while Figures 2.19, 2.20, and 2.21 illustrate the loads acting on the truss for super dead load, live load, and wind load, respectively.

Table 2. 34 Truss Profile

WF 300 x 150 x 6.5 x 9			
Steel Type	A36		
Fy	250	Mpa	

Fu	400	Mpa	
Ag	4678	mm2	
b	150	mm	
h	300	mm	
t1	6.5	mm	
t2	9	mm	
E	200000	Mpa	
rx	12.4	cm	
ry	3.29	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	

Table 2. 35 Load Recap on Truss

Load Recap on Truss		
SDLmid	3.26	kN
SDLcorner	1.71	kN
L	1.00	kN
Wmid	-3.22	kN
Wcorner	-1.61	kN

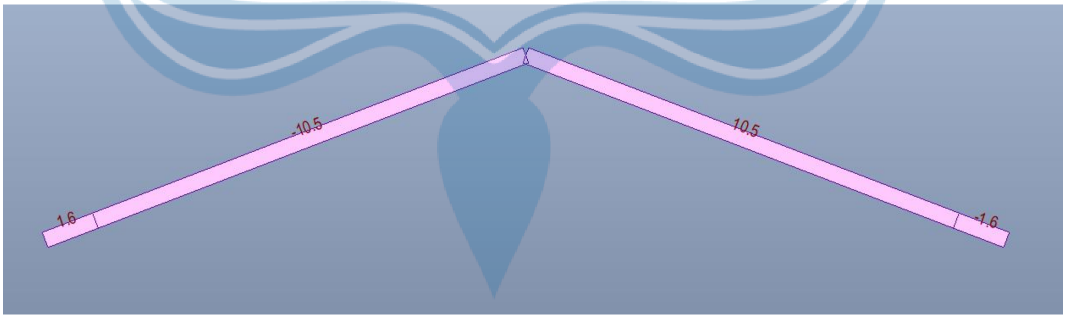


Figure 2. 19 Super Imposed Dead Load on Truss

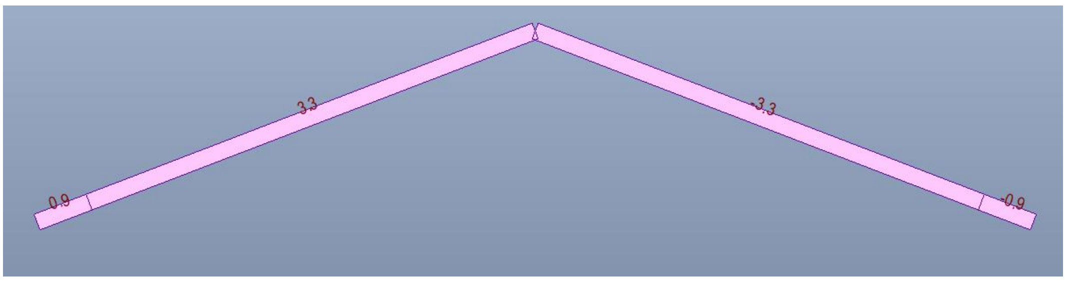


Figure 2. 20 Live Load on Truss

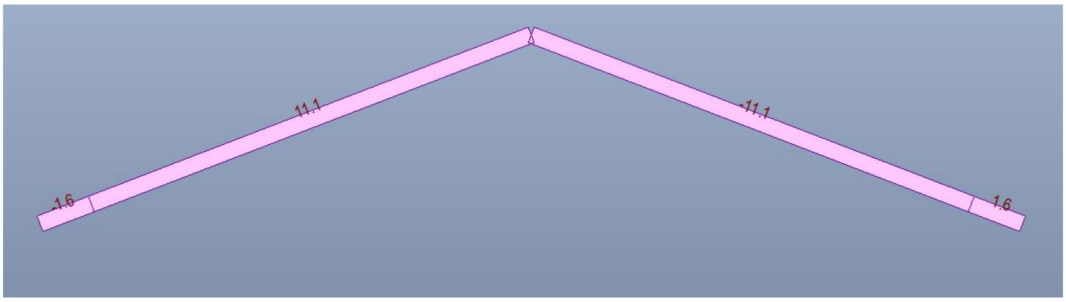


Figure 2. 21 Wind Load on Truss

The building features a hip roof design, composed of three trusses with the same profile but differing in shape and installation. Figure 2.22, Figure 2.23, and Figure 2.24 depict the types of trusses employed in this building.

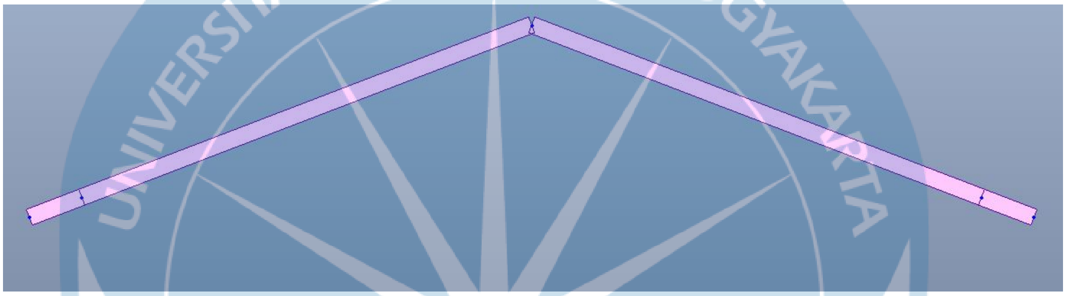


Figure 2. 22 First Truss

Figure 2. 23 Second Truss

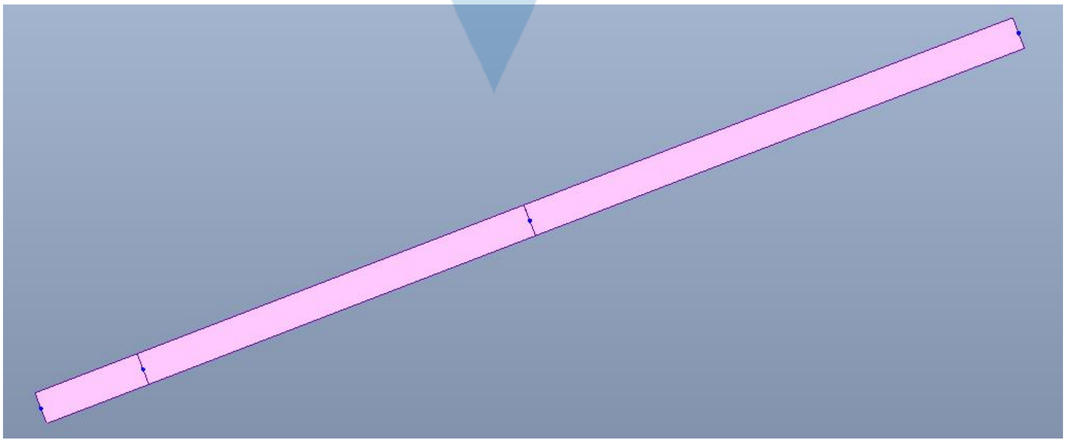


Figure 2. 24 Third Truss

a. MIDAS Model Output

The verification of the required truss strength is conducted by examining the axial force and displacement outputs from MIDAS. Table 2.36 presents the axial force output, and Table 2.37 displays the displacement of each joint for further analysis.

Table 2. 36 Truss Axial Force Output

Member	Axial Force (kN)	Combination
1	0.858	1.4D
2	-44.746	1.4D
3	-44.746	1.4D
4	0.858	1.4D
Maximum	44.746	

Table 2. 37 Truss Displacement

Joints	Displacement (mm)	Combination
J1	0	
J2	0	
J3	-0.001035	1.4D
J4	-0.000029	1.4D
J5	-0.000029	1.4D
Maximum	0.001035	

If the maximum displacement is smaller than 1 inch or 25.5 mm, it indicates that the truss is within the acceptable limits and can be considered safe based on the criteria set for displacement.

b. Tension Member Check

In the analysis of a truss, compression and tension members are evaluated separately. Tension members are assessed based on factors such as effective length factor (K), net area (An), reduction factor for shear lag (Ue), effective area (Ae), and safety factor (φ). Table 2.38 confirms that the tension member with the highest tension load has been verified and is deemed safe from yielding and rupture.

Table 2. 38 Tension Member Check

Check for Tension Member			
Pu max	44.746	kN	

L	8000	mm2	
K	1		
Slenderness	243.1610942		KL/r
	KL/r<300 OK		
An	4678	mm2	Ag-n×dh×t×2 (2 for double angle)
Ue	1		SNI 1729:2015 Table D3.1
Ae	4678	mm2	An×Ue
Yield Strenth φPn	1052.55	kN	0.9×Fy×Ag
	Safe		
Rupture Strenth φPn	1403.4	kN	0.75×Fu×Ae
	Safe		

c. Compression Member Check

Each member within the compression group undergoes examination considering factors such as effective length factor (K), steel modulus of elasticity (E), steel yield strength (Fy), critical elastic buckling stress (Fe), critical compressive stress (Fcr), and safety factor (φ). As per Table 2.39, it is confirmed that every member is safe against compression loads.

Table 2. 39 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	0.858	1000	30.4	133.2189	2136.5917	238.0515	1002.2442	Safe
2	44.746	8569	260.46	133.2189	29.09787349	25.51884	107.4394	Safe
3	44.746	8569	260.46	133.2189	29.09787349	25.51884	107.4394	Safe
4	0.858	1000	30.4	133.2189	2136.5917	238.0515	1002.2442	Safe

Where:

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

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

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. 40).

Table 2. 40 Slenderness Check

λ	23.07692308	
λr	12.72792206	SNI 1279:2015 Table B4.1.a
Λ>λr Slender		

Where:

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

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

2.10.4 Connection

For the truss connections, bolt connections will be employed. The axial force in the maximum member at joint J1 (refer to Figure 2.25 and Table 2.41 Joint with Maximum Axial Force) is utilized for the verification. The qualities of bolts are listed in Table 2.42 for reference.

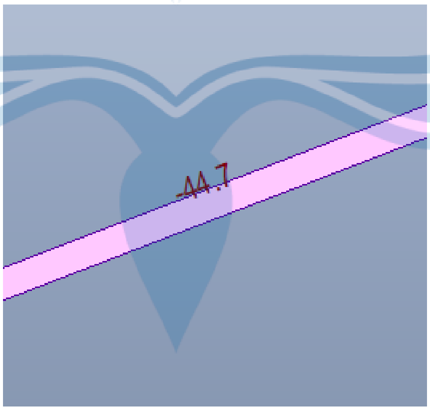


Figure 2. 25 Joint with Maximum Axial Force

Table 2. 41 Joint with Maximum Axial Force

Joint:	J1	
Member	Axial Force	
1	-44.746	kN
16	44.746	kN

Table 2. 42 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)	8		
db	20.00	mm	
dh	22	mm	
End-Plate Type	BJ37		
End-Plate Thickness	10	mm	
Fy of End-Plate	240	Mpa	
Fu of End-Plate	370	Mpa	
End-Plate Depth, d	310	mm	
End-Plate Width, b	160	mm	
End-Plate amount, m	1		
Fy of Member	250	Mpa	
Fu of Member	400	Mpa	
Fu of Bolt	370	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. Bolt Connection Strength

The strength of the bolt connection needs to be checked for shear strength, slip-critical strength, bearing strength, yielding and rupture, and block shear strength. The initial design involves a total of 8 bolts. The checks for the strength of these 8 bolts in resisting shear, slip-critical forces, bearing forces, and nominal moment are presented in Table 2.43, Table 2.44, Table 2.45, and Table 2.46, respectively. The calculations demonstrate that the 8 bolts are sufficient to withstand the axial forces in the members. Therefore, the decision is made to use 8 bolts for the connection.

Table 2. 43 Shear Strength

Shear Strength for 1 bolt			
Ab	380.13	mm2	
n	8		
Ruv	2.4375	kN	
Vd	106.057	kN	ϕf*Fnv*Ab
Check Vd>Ruv	Safe		

Table 2. 44 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	246.792	kN	φRn×n>Pu Safe

Table 2. 45 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	

Table 2. 46 Nominal Moment of Bolt

ΣT	410400		
neutral axis, a	10.6875	mm	ΣT/(fy*b)
d1	74.31		
d2	144.31	mm	
d3	214.31	mm	
d4			
fMn	2193075	Nmm	fy*a^2*b/2
Td1	7624206	Nmm	
Td2	14806206	Nmm	
Td3	21988206	Nmm	
Td4	0	Nmm	
Mn	41950524	Nmm	
	41.95052	kNm	
φMn	37.75547	kNm	
Check φMn>Mu	Safe		

b. Weld Connection Strength

Certainly! The bolt connection strength needs to be assessed for shear strength, slip-critical strength, bearing strength, yielding and rupture, and block shear strength. Initially designed with a total of 8 bolts, the evaluations for these bolts in resisting shear, slip-critical forces, bearing forces, and nominal moment are outlined in Table 2.43, Table 2.44, Table 2.45, and Table 2.46, respectively. The calculations confirm that the 8 bolts possess sufficient strength to endure the axial forces in the members. As a result, the decision is made to utilize 8 bolts for the connection.

Table 2. 47 Weld Connection Calculation

Description	**Value**	**Unit**	**Note**
F_{EXX}	70	ksi	
	482.6330105	Mpa	
b	300	mm	
d	132	mm	
2b	600	mm	
2d	264	mm	
A	864	mm	A = 2b + 2d
Sx	45408	mm2	Sx = bd + d^2/3
ft	737.7554616	N/mm	ft = Mu/Sx
fv	22.56944444	N/mm	fv = Vu/A
fr	738.1006035	N/mm	fr = ftotal = sqrt(fv^2+ft^2)
	0.841771875	mm	
Fnw	289.5798063	N/mm2	
ϕRn	243.7601365	N/mm	ϕRn = 0.75*0707*D/16*Fnw
D	3.027979119		
w	5	mm	
Use w = 5 mm for weld connection			

2.10.5 Anchor

An anchor serves as a connection between steel and concrete, linking the steel truss to the ring balk. This connection is vital for transferring the load from the truss to the concrete beam through the anchor. Therefore, a thorough check is required to assess various potential failures in the anchor, with a focus on compressive strength and shear strength. The anchor exhibits a shear strength of 140.2 kN, a compression strength of 1740.375 kN, spalling shear strength of 142.94 kN, pry-out strength of 201.7 kN, and breakout strength of 35.47 kN. The corresponding calculations are outlined in Table 2.50, Table 2.51, Table 2.53, Table 2.54, and Table 2.55, respectively.

Table 2. 48 Anchor Properties

Type	**Steel Headed Stud**			
Dmin	25	mm	>2.5*t(member)	SNI 1729:2015 Section I8.1
Actual D	25.5	mm		
Lmin	100	mm	>4*D	SNI 1729:2015 Section I8.2
Actual L	204	mm	h/d>8 =>h=8*D	Section I8.3 when subjected to shear and tension

Table 2. 49 Reaction and Maximum Load of Truss

Load			
Ru	36.85	kN	Reaction
Pu	53.25	kN	Member Load

Table 2. 50 Anchor Shear Strength

Description	**Value**	**Unit**	**Note**
Shear Strength			
Asa	490.8738521	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)	215689.8821	N	
Rg×Rp×Asa×Fu	219911.4858	N	
Qnv	215.6898821	kN	SNI 1729:2015 Eq I8.1
φ	0.65		
φQnv	140.1984234	kN	

Table 2. 51 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		

Description	Value	Unit	Note
φPc	1740.375	kN	φPc>Ru Safe

Table 2. 52 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	200	mm	
hef	204	mm	SNI 2847:2019 Fig R2.2

Table 2. 53 Anchor Spalling Shear Strength in Concrete

Description	Value	Unit	Note
Concrete spalling			
Asa	490.8738521		$0.25\pi D^2$
Fu	448	Mpa	
Vsa	219911.4858	N	
φ	0.65		SNI 2847:2019 Sec 17.3.3
φVsa	142.9424657	kN	

Table 2. 54 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	0.798039216		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	333906	mm2	Fig R17.4.2.1b
Anco	395352	mm2	Fig R17.4.2.1a
kc	10		
λ	1	normal	Sec 17.2.6 & 19.2.4
fc'	30	Mpa	
Nb	159590.0749	N	Eq. 17.4.2.2
Ncpg	134456.0682	N	Ncpg=Ncbg Sec 17.5.3.1b
Vcpg	268912.1365	N	
ϕ	0.75		Sec 17.3.3 Cond. A

Description	Value	Unit	Note
ϕVcpg	201.6841024	kN	

Table 2. 55 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	75000	mm2	Case 3 Fig R17.5.2.1.b
Avco	45000	mm2	
L	204	mm	anchor length
D	25.5	mm	anchor diameter
fc'	30	Mpa	
ca1	100	mm	
λ	1	normal	Sec 17.2.6 & 19.2.4
Vb1	25153.58929		Eq. 17.5.2.2a
Vb2	20265.73463		Eq. 17.5.2.2b
Take Vb	20265.73463		
Vcb	47.28671413	kN	Eq 17.5.2.1.b
ϕ	0.75		Sec 17.3.3 Cond. A
ϕVcb	35.4650356	kN	

a. Number of Anchor

Upon comparing the highest load with the smallest anchor strength, the total number of anchors required is determined. It is evident that the maximum load is 53.25 kN, and the minimum shear strength necessary to prevent the anchor from breaking free is 35.47 kN. Consequently, two anchors are needed for one side. Given that the truss has a WF profile, a total of four anchors are required, accounting for both sides.

Table 2. 56 Anchor Number

Recap Shear Strength		
φVsa	142.9424657	kN
ϕVcpg	201.6841024	kN
ϕVcb	35.4650356	kN
Smallest ϕV	35.4650356	kN

Recap Shear Strength		
n anchor	1.501478826	
n use	2	1 side
	4	2 sides

b. Conclusion

In this construction, the employed purlin is made of steel, specifically C125x150x150x2, with a length of 4 meters. To prevent sagging between purlin lengths, sag rods with a diameter of 10 mm are utilized. The truss is constructed using steel WF 300x150x6.5x9. Each member on the truss is connected using 8 bolts. Additionally, the connection between the steel truss and the concrete ring balk involves the use of 4 anchors.

2.11 Beam Design

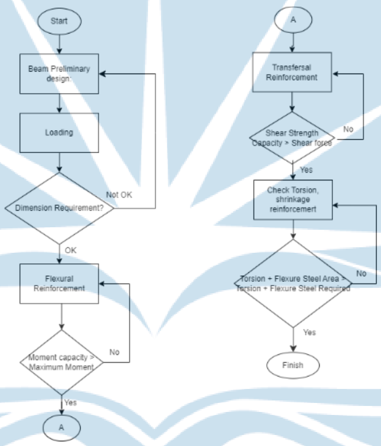


Figure 2. 26 Beam Design Flowchart

The dimensions and qualities of the reinforced concrete utilized for constructing the beam in this project are detailed in Table 2.57. Beams designed for buildings with earthquake-resistant special moment structural systems (SRPMK) must adhere to the specifications outlined in SNI 2847:2019 Article 18.6, as indicated in Table R18.2. The calculation employs a standard concrete beam with $\lambda = 1$, resulting in a beam type 2 with a length of 8 meters. The beam design process is visually represented in Figure 2.26, titled "Beam Design Flowchart.

Table 2. 57 Beam Properties and Dimensions

Parameter	**SNI 2847:2019**	**Equation**	**Unit**	**Value**
Beam Length, L		Input	mm	8000
Beam Width, b		Input	mm	600
Beam Height, h		Input	mm	650
Support Length	18.6.4.1	2 * h	mm	1300
Longitudinal Reinforcement Diameter, d_b		Input	mm	25
Additional Reinforcement Diameter, d_{bt}		Input	mm	13
Stirrups Diameter, d_s		Input	mm	13
Concrete Cover, c_c		Input	mm	40
Effective Beam Height, d		$h - c_c - d_s - d_b/2$	mm	584.5
Concrete Compressive Strength, f_c'		Input	MPa	30
Longitudinal Reinf. Yield Strength, f_y		Input	MPa	420
Stirrups Yield Strength, f_{yv}		Input	MPa	420
β_1	Table 22.2.2.4.3	0.65 <= 0.85 - 0.05 * (fc' - 28) / 7 <= 0.85		0.8357
Column Length, c_1		Input (Beam width perpendicular side)	mm	1000
Column Width, c_2		Input (Beam width parallel side)	mm	1000
L_n		$L - c_1$	mm	7000
λ		Assume not using lightweight concrete		1

2.11.1 Beam Flexural Design

In the design of a beam, it is essential to consider internal forces such as moments, shear, and torsion. The structural analysis program MidasGen is utilized to determine these internal forces. Table 2.58 provides the output of internal forces for Beam type 1.

Table 2. 58 Beam Internal Force

Parameter	Unit	Value
Ultimate Moment $M_{u,support}$ (-)	kN-m	-461.05
Ultimate Moment $M_{u,support}$ (+)	kN-m	332.28
Ultimate Moment $M_{u,field}$ (-)	kN-m	-183.63
Ultimate Moment $M_{u,field}$ (+)	kN-m	88.56
Ultimate Axial Force, Pu	kN	238.32

Beams designed for Special Resisting Moment Design Frame must comply with specific axial load and size standards, as outlined in SNI 2847:2019 Article 18. However, it's noted that the axial force requirement is no longer considered in the 2019 standard and can be disregarded. Table 2.59 presents the results of the verification process for these requirements.

Table 2. 59 Dimension and Geometry Checking for Beam

Parameter	SNI 2847:2019	Equation	Value	Note
Axial Force Requirement	Not advised. See R18.6.1 and 18.6.4.7	Pu <= $0.1A_gf_c$'?	Pu=238.32 kN < 0.1Agfc=1170 kN	OK
Effective Height Requirement	18.6.2.1	L_n >= 4d?	Ln = 7000 mm > 4d = 2338 mm	OK
Width Requirement 1	18.6.2.1	b >= min(0.3h, 250 mm)?	b=600 mm > 0.3h=195 mm, 250 mm	OK
Width Requirement 2	18.6.2.1	b <= c_2 + 2 * min (c_2, 0.75 c_1)?	b=600 mm < 2500 mm	OK

Flexural design is the primary focus of the beam design process, considering the moment capacity contributed by the steel reinforcing bar. The design accounts for locations at support for both positive and negative moments, as well as at the span for both. The results of the computation for negative support, positive support, and negative span are presented in Tables 2.60, 2.61, 2.62, and 2.63, respectively. The nominal moment capacity for each reinforcement point is then compared to the ultimate moment in the preceding table.

Table 2. 60 Negative Support Flexural Design

Parameter	SNI 2847:2019	Equation	Unit	Value
Number of Negative Support Reinforcement, n		Input		5
d_b			mm	25
Net Distance Between Reinforcements		$(b - 2\ c_c - 2\ d_s - n * d_b) / (n - 1)$	mm	92.250
Net Distance Check	25.2.1	Net Distance >= d_b dan 25 mm?		YES
Number of Layers				1
As use		$n * \pi/4 * d_b^2$	mm^2	2454.37
$As_{min,1}$	9.6.1.2	$(f_c')^{0.5} / (4 * f_y) * b * d$	mm^2	1143.37
$As_{min,2}$	9.6.1.2	$1.4 / (4 * f_y) * b * d$	mm^2	1169
As min Check		As use >= As min?		OK
ρ		As / (b * d)		0.70%
$\rho_{max,1}$	No	$0.75\ \rho_b = 0.75 * 0.85 * \beta_1 * f_c' / f_y * (600/(600 + f_y))$		2.24%
$\rho_{max,2}$	18.6.3.1	2.5%		2.50%
As max Check		$\rho <= \rho$ max?		OK
a	22.2.2.4.1	As * fy / (0.85 * fc' * b)	mm	67.375
M_n	22.2.2.4.1	As * fy * (d - a/2)	kN-m	567.797
c	22.2.2.4.1	$a / \beta 1$	mm	80.619
ε_s	22.2.1.2, 22.2.2.1	(d - c) / c * 0.003		0.019
ϕ	Table 21.2.2	$0.65 <= 0.65 + (\varepsilon s - 0.002) / 0.003 * 0.25 <= 0.9$		0.900
ϕM_n		ϕ * Mn	kN-m	511.017
$M_{u,support}$ (-)			kN-m	461.050
Capacity Check		$\phi M_n > M_u$?		OK
As Req		Mu / [fy * (d - a/2)]	mm^2	1992.94

Table 2. 61 Positive Support Flexural Design

Parameter	**SNI 2847:2019**	**Equation**	**Unit**	**Value**
n		Input		5
d_b			mm	25
Net Distance Between Reinforcements		$(b - 2\ c_c - 2\ d_s - n * d_b) / (n - 1)$	mm	92.250
Net Distance Check	25.2.1	Net Distance >= d_b dan 25 mm?		IYA
Number of Layers				1
As use		$n * \pi/4 * d_b^2$	mm^2	2454.37
As $_{min,1}$	9.6.1.2	$(f_c')^{0.5} / (4 * f_y) * b * d$	mm^2	1143.37
As $_{min,2}$	9.6.1.2	1.4 / (4 * fy) * b * d	mm^2	1169
As $_{min,4}$	18.6.3.2	0.5 * As Negative Support	mm^2	1227.18
As min Check		As use >= As min?		OK
ρ		As / (b * d)		0.70%
$\rho_{max,1}$		$0.75\ \rho_b = 0.75 * 0.85 * \beta_1 * f_c' / f_y * (600/(600 + f_y))$		2.24%
$\rho_{max,2}$	18.6.3.1	2.5%		2.50%
As max Check		ρ <= ρ max?		OK
a	22.2.2.4.1	As * fy / (0.85 * fc' * b)	mm	67.375
M_n	22.2.2.4.1	As * fy * (d - a/2)	kN-m	567.8
c	22.2.2.4.1	a / β1	mm	80.619
ε_s	22.2.1.2, 22.2.2.1	(d - c) / c * 0.003		0.019
ϕ	Table 21.2.2	0.65 <= 0.65 + (εs - 0.002) / 0.003 * 0.25 <= 0.9		0.900
ϕM_n		ϕ * Mn	kN-m	511.017
M_u			kN-m	332.280
Cek $\phi M_n > M_u$		$\phi M_n > M_u$?		OK

Parameter	SNI 2847:2019	Equation	Unit	Value
As Req		Mu / [fy * (d - a/2)]	mm^2	1436.320

Table 2. 62 Negative Span Flexural Design

Parameter	SNI 2847:2019	Equation	Unit	Value
n		Input		3
d_b			mm	25
Net Distance Between Reinforcements		$(b - 2 c_c - 2 d_s - n * d_b) / (n - 1)$	mm	209.5
Net Distance Check	25.2.1	Net Distance >= d_b and 25 mm?		IYA
Number of Layers				1
As use		$n * \pi/4 * d_b^2$	mm^2	1472.62
As $_{min,1}$	9.6.1.2	$(f_c')^{0.5} / (4 * f_y) * b * d$	mm^2	1143.37
As $_{min,2}$	9.6.1.2	1.4 / (4 * fy) * b * d	mm^2	1169
As $_{min,4}$	18.6.3.2	0.25 * As Negative Support	mm^2	613.6
As min Check		As use >= As min ?		OK
ρ		As / (b * d)		0.42%
$\rho_{max,1}$		$0.75 \rho_b = 0.75 * 0.85 * \beta_1 * f_c' / f_y * (600/(600 + f_y))$		2.24%
$\rho_{max,2}$	18.6.3.1	2.5%		2.50%
As max Check		ρ <= ρ max ?		OK
a	22.2.2.4.1	As * fy / (0.85 * fc' * b)	mm	40.425
M_n	22.2.2.4.1	As * fy * (d - a/2)	kN-m	349.012
c	22.2.2.4.1	a / β1	mm	48.372
ε_s	22.2.1.2, 22.2.2.1	(d - c) / c * 0.003		0.033
ϕ	Table 21.2.2	0.65 <= 0.65 + (εs - 0.002) / 0.003 * 0.25 <= 0.9		0.9
ϕM_n		ϕ * Mn	kN-m	314.1
M_u			kN-m	183.63
Cek $\phi M_n > M_u$		$\phi M_n > M_u$?		OK

Parameter	SNI 2847:2019	Equation	Unit	Value
As Req		Mu / [fy * (d - a/2)]	mm^2	774.8

Table 2. 63 Positive Support Flexural Design

Parameter	SNI 2847:2019	Equation	Unit	Value
n		Input		3
d_b			mm	25
Net Distance Between Reinforcements		$(b - 2 c_c - 2 d_s - n * d_b) / (n - 1)$	mm	209.500
Net Distance Check	25.2.1	Net Distance >= d_b dan 25 mm?		IYA
Number of Layers				1
As use		$n * \pi/4 * d_b^2$	mm^2	1472.622
$As_{min,1}$	9.6.1.2	$(f_c')^{0.5} / (4 * f_y) * b * d$	mm^2	1143.371
$As_{min,2}$	9.6.1.2	1.4 / (4 * fy) * b * d	mm^2	1169.000
$As_{min,4}$	18.6.3.2	0.25 * As Negative Support	mm^2	613.592
As min Check		As use >= As min ?		OK
ρ		As / (b * d)		0.42%
$\rho_{max,1}$		$0.75\ \rho_b = 0.75 * 0.85 * \beta_1 * f_c' / f_y * (600/(600 + f_y))$		2.24%
$\rho_{max,2}$	18.6.3.1	2.5%		2.50%
As max Check		ρ <= ρ max ?		OK
a	22.2.2.4.1	As * fy / (0.85 * fc' * b)	mm	40.425
M_n	22.2.2.4.1	As * fy * (d - a/2)	kN-m	349.012
c	22.2.2.4.1	a / β1	mm	48.372
ε_s	22.2.1.2, 22.2.2.1	(d - c) / c * 0.003		0.033
ϕ	Table 21.2.2	0.65 <= 0.65 + (εs - 0.002) / 0.003 * 0.25 <= 0.9		0.900

Parameter	SNI 2847:2019	Equation	Unit	Value
ϕM_n		ϕ * Mn	kN-m	314.111
M_u			kN-m	88.560
Cek $\phi M_n > M_u$		$\phi M_n > M_{u\,?}$		OK
As Req		Mu / [fy * (d - a/2)]	mm^2	373.670

2.11.2 Beam Shear Design

The Midas Gen software is utilized to determine the internal shear force at both support and span locations. Subsequently, this ultimate shear force is compared against the nominal shear strengths of both the steel reinforcement and concrete. The internal force distribution at the span and support is presented in the Force table.

Table 2. 64 Shear Internal Force

Parameter	Unit	Value
$V_{u,support}$	kN	212.45
$V_{u,span}$	kN	187.68

The shear design for a unique moment-resisting frame system considers the potential shear or sway shear resulting from the expected moment. Both the sway shear force and the ultimate shear from the factored load combination are factored in when calculating the earthquake shear force in this system. If the sway shear is greater than half of the earthquake shear force and the axial force is less than one-twentieth of the section axial strength, the concrete shear strength must be considered as zero, and the shear strength is solely dependent on the transversal reinforcement. The shear design calculations at support and span are detailed in Tables 2.65 and 2.66, respectively.

Table 2. 65 Shear Design at Support

Parameter	SNI 2847:2019	Equation	Unit	Value
Vg,Support	R18.6.5	Input [Combination 1.2 D + L]	kN	171.44
As+ Support		Steel Area at Positive Support	mm^2	2454.369

Parameter	**SNI 2847:2019**	**Equation**	**Unit**	**Value**
As- Support		Steel Area at Negative Support	mm^2	2454.369
a_{pr}^{+}		1.25 a (Positive Support)	mm	84.219
a_{pr}^{-}		1.25 a (Negative Support)	mm	84.219
M_{pr}^{+}	R18.6.5	A_s^{+} * (1.25 f_y) * (d - a_{pr}^{+}/2)	N mm	698894237
M_{pr}^{-}	R18.6.5	A_s^{-} * (1.25 f_y) * (d - a_{pr}^{-}/2)	N mm	698894237
V_{sway} or V_{pr}	18.6.5.1	(M_{pr}^{+} + M_{pr}^{-}) / L_n	N	199684
V_e	18.6.5.1	V_g + V_{pr}	N	371124
V_{pr}			N	199684
1/2 V_e			N	185562
Pu			N	238320
Ag fc' / 20			N	585000
V_c Considered?	18.6.5.2	V_c = 0 **if** V_{pr} >= 1/2 V_e **and** P_u < A_g f_c' / 20		No
V_c			N	0
Number of Legs		Input		3
Av		n *π/4 *d_s^2	mm^2	398.197
Spacing		Input	mm	100
Max Spacing 1	18.6.4.4	d / 4	mm	146.13
Max Spacing 2	18.6.4.4	6 d_b	mm	150.00
Max Spacing 3	18.6.4.4	150 mm	mm	150.00
Spacing Check				OK
V_s	22.5.10.5.3	A_v * f_{yv} * d / s	N	977533
V_s Limit	22.5.1.2	0.66 * $(f_c')^{0.5}$ * b * d	N	1267770
ϕ	12.5.3.2, 21.2.4			0.75
V_n		V_c + V_s	N	977533
V_u			N	371124
ϕV_n / V_u				1.975
Capacity Check		ϕV_n / V_u >= 1?		OK

Table 2. 66 Shear Design at Span

Parameter	SNI 2847:2019	Equation	Unit	Value
Number of Legs		Input		3
Av		$n * \pi/4 * d_s^2$	mm^2	398.197
Spacing		Input	mm	100
Max Spacing	18.6.4.6	d / 2	mm	292.25
Spacing Check				OK
V_s	22.5.10.5.3	$A_v * f_{yv} * d / s$	N	977533
V_s Limit	22.5.1.2	$0.66 * (f_c')^{0.5} * b * d$	N	1267770
V_c	22.5.5.1	$0.17 * (f_c')^{0.5} * b * d$	N	326547
ϕ	12.5.3.2, 21.2.4			0.75
V_n		$V_c + V_s$		1304080
V_u			N	187680
$\phi V_n / V_u$				5.211
Capacity Check		$\phi V_n / V_u >= 1?$		OK

2.11.3 Beam Torsion Design

Beams need to undergo inspection for torsion, as shear stresses acting on points of application not aligned with the shear center of the beam section can typically induce torsion in beams. It is essential to examine a beam's torsion capacity, and if necessary, torsion steel reinforcement should be added to prevent torsion or twisting distortion. The section parameters of the beam are provided in Table 2.67..

Table 2. 67 Section Geometry Parameter

Parameter	SNI 2847:2019	Equation	Unit	Value
A_{cp}		b * h	mm^2	390000
P_{cp}		2 * (b + h)	mm	2500
x_o		$b - 2c_c - d_s$	mm	507
y_o		$h - 2c_c - d_s$	mm	557
A_{oh}	R22.7.6.1.1	$x_o * y_o$	mm^2	282399
A_o	22.7.6.1.1	$0.85 A_{oh}$	mm^2	240039
P_h	22.7.6.1	$2 * (x_o + y_o)$	mm	2128
Internal Torsion Force, T_u		Input	kN m	74.85
Cracking Torsion, T_{cr}		$0.33 * (f_c')^{0.5} * A_{cp}^2 / P_{cp}$	N mm	109967353

ϕ	Table 21.2.1			0.75
ϕ T_{cr} / 4			N mm	20618879
Needs Torsion Reinforcement?	Table 22.7.4.1	$T_u > \phi\ T_{cr} / 4$?		Yes

In accordance with SNI 2847:2019, the torsion strength of a beam needs to be evaluated, and this evaluation also encompasses the influence of shear reinforcement on torsion strength. The section requirement checking for the beam is presented in Table 2.68. This table provides an overview of the beam's compliance with the specified section requirements.

Table 2. 68 Section Dimension Requirement Check

Parameter	**SNI 2847:2019**	**Equation**	**Unit**	**Value**
Torsion Type		Certain Static = Equilibrium, Uncertain Static = Compatibility		Compatibility
T_u Use	22.7.3.2, 22.7.5	$\phi\ T_{cr}$ or T_u	N mm	74850000
V_u		From Shear Design Sheet	N	371124
V_c	22.5.5.1	$0.17 * (f_c')^{0.5} * b * d$	N	326547
Ultimate Shear Stress + Torsion	22.7.7.1	$\{ [V_u / b*d]^2 + [T_u P_h / (1.7 A_{oh}^2)]^2 \}^{0.5}$	MPa	1.581
Concrete Stress Capacity	22.7.7.1	$\phi * \{ [V_c / (b * d)] + 0.66 * (f_c')^{0.5} \}$	MPa	3.410
Section Dimensions Check	22.7.7.1	Left Section <= Right Section?		OK
f_y / f_{yt}		Torsion Reinforcement Steel Yield Strength = Flexible and Shear Reinforcement Steel Yield Strengths		1
θ	22.7.6.1.2	θ is taken for non-prestressed structure component beam	°	45
n Support leg		From Shear Design		3
n Field leg		From Shear Design		3

Parameter	SNI 2847:2019	Equation	Unit	Value
s Support		From Shear Design	mm	100
s Field		From Shear Design	mm	100
s max 1	9.7.6.3.3	P_h / 8	mm	266
s max 2	9.7.6.3.3	300 mm	mm	300
Support Spacing Check		s Support >= s max?		OK
Field Spacing Check		s Field >= s max?		OK
Av+t / s Support Use		n *π/4 *d_s^2 / s	mm^2/ mm	3.982
Av+t / s Support Use		n *π/4 *d_s^2 / s	mm^2/ mm	3.982
A_t / s	22.7.6.1	T_u / (2 * ϕ * A_o * f_{yv})	mm^2/ mm	0.495
Av / s Support Req		(Vu Support / φ - Vc) / (fyv * d)	mm^2/ mm	2.016
Av / s Field Req		(Vu Field / φ - Vc) / (fyv * d)	mm^2/ mm	-0.311
Av+t / s Support Req	R9.5.4.3	2 * A_t / s + A_v / s		3.006
Av+t / s Field Req	R9.5.4.3	2 * A_t / s + A_v / s		0.679
A_{v+t} / s min 1	9.6.4.2	0.062 * $(f_c')^{0.5}$ * b / f_{yv}		0.485
A_{v+t} / s min 2	9.6.4.2	0.35 * b / f_{yv}		0.500
Support Shear + Torsion Check		Av+t / s Use >= Av+t / s Req and min?		OK
Field Shear + Torsion Check		Av+t / s Use >= Av+t / s Req and min?		OK

When the torsion strength of a concrete beam is found to be insufficient or its height is excessive, additional longitudinal reinforcing for torsion strength is necessary. Table 2.69 presents the estimate for torsion longitudinal reinforcement, providing guidance on the required reinforcement to address torsion-related considerations in the concrete beam.

Table 2. 69 Torsion Longitudinal Reinforcement

Parameter	**SNI 2847:2019**	**Equation**	**Unit**	**Value**
db or dbt			mm	13
d_b, min	9.7.5.2	0.042 s	mm	4.2
Cek d_b		d_b >= d_b min ?		OK
As Req Top Support		From Shear Design	mm^2	1992.943
As Req Bottom Support		From Shear Design	mm^2	1436.320
As Req Top Field		From Shear Design	mm^2	774.808
As Req Top Field		From Shear Design	mm^2	373.670
A_l	22.7.6.1	A_t / s * P_h	mm^2	1053.273
A_l min	9.6.4.3	0.42 * $(f_c')^{0.5}$ * A_{cp} / f_y - (A_t/s) * P_h	mm^2	1082.845
As + Al Req Support			mm^2	4512.108
As + Al Req Field			mm^2	2231.323
Top Support n		From Flexible Design Sheet		5
Middle Support n		Input (Multiples of 2 Advised)		4
Bottom Support n		From Flexible Design Sheet		5
Vertical Support n		2 + n Tengah / 2		4
Top Field n		From Flexible Design Sheet		3
Middle Field n		Input (Multiples of 2 Advised)		4
Bottom Field n		From Flexible Design Sheet		3
Vertical Field n		2 + n Middle / 2		4
Horizontal Support Spacing		(b - $2c_c$ - $2d_s$ - d_b) / [min(n atas, n bawah) - 1]	mm	117
Vertical Support Spacing		(h - 2cc - 2ds - db) / (n Vertical - 1)	mm	173
Horizontal Field Spacing		(b - $2c_c$ - $2d_s$ - d_b) / [min(n atas, n bawah) - 1]	mm	235
Vertical Field Spacing		(h - 2cc - 2ds - db) / (n Vertical - 1)	mm	173

Parameter	SNI 2847:2019	Equation	Unit	Value
Field Longitudinal Reinforcement Spacing Check		Spacing >= 300 mm ?		OK
Support Longitudinal Reinforcement Spacing Check		Spacing >= 300 mm ?		OK
As + Al Use Support			mm^2	5439.668
As + Al Use Field			mm^2	3476.172
Support Flexibility + Torsion Check		As + Al Use >= As + Al Req ?		OK
Field Flexibility + Torsion Check		As + Al Use >= As + Al Req ?		OK

2.11.4 Conclusion

Based on the calculations and checks for moments, shear, and torsion, the arrangement of beam reinforcement is detailed in Table 2.70. This table provides a comprehensive overview of the configured reinforcement for the beam, ensuring that it meets the necessary strength and stability criteria.

Table 2. 70 Beam Reinforcement Recap

Longitudinal Reinforcement	
Top Support Longitudinal	5 D25
Middle Support Longitudinal	4 D13
Bottom Support Longitudinal	5 D25
Top Field Longitudinal	3 D25
Middle Field Longitudinal	4 D13
Bottom Field Longitudinal	3 D25
Stirrups	
Support Stirrups	3D13-100
Field Stirrups	3D13-100

2.12 Tie Beam Design

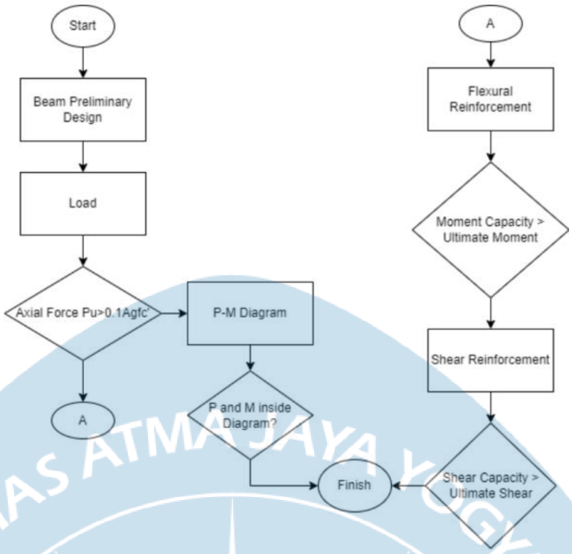


Figure 2. 27 Tie Beam Design Flowchart

Structural concrete beams, known as tie beams, are strategically positioned on the ground to transfer the load from the column to the foundation below. The tie beam design process is illustrated in Figure 2.27, and the parameters for the tie beam are detailed in Table 2.71. This table provides key information about the specifications and characteristics of the tie beam in the structural system.

Table 2. 71 Tie Beam Properties

Parameter	**References**	**Equation**	**Unit**	**Value**
Tie Beam Net Length, L_n		Input	mm	5700
Tie Beam Width, b		Input	mm	300
Tie Beam Height, h		Input	mm	400
Longitudinal Reinforcement Diameter, d_b		Input	mm	16
Transversal Reinforcement Diameter, d_s		Input	mm	10
Concrete Cover, c_c		Input	mm	30
Tie Beam Effective Height, d		$h - c_c - d_s - d_b/2$	mm	352
Concrete Compressive Strength, f_c'		Input	MPa	30
Longitudinal Reinf. Yield Strength, f_y		Input	MPa	420

Transversal Reinf. Yield Strength, f_{yv}		Input	MPa	420
β_1	SNI 2847:2019 Table 22.2.2.4.3	0.65 <= 0.85 - 0.05 * (fc' - 28) / 7 <= 0.85		0.8357
λ		Assuming not using lightweight concrete		1
Minimum Width Requirement	SNI 2847:2019 Article 18.13.3.2	b >= min(Ln/20, 450 mm) ?		OK

2.12.1 Loading

Tie beams are subjected to various axial loads originating from gravitational forces, foundation settlements, and columns. In the design process, the consideration for flexural reinforcement is emphasized instead of computing the axial strength when the axial force (Pu) is less than 10% of the compressive strength multiplied by the cross-sectional area. The axial forces acting on the tie beam are calculated and presented in Table 2.72.

Table 2. 72 Tie Beam Calculations

Axial Force Calculation				
Column Axial Force Due to Factored Gravity Load, P_g		Input [Combination 1.2 D + 1.6 L]	kN	2626.2885
Response Spectrum Acceleration Parameters in Short Periods, S_{DS}		Input	g	0.9
Tie Beam Axial Force, P_u	SNI 1726:2019 Article 7.13.6.2	10% * S_{DS} * P_g	kN	236.366
Axial Force is Considered?		$P_u > 0.1\ A_g\ f_c'$?		Tidak
Internal Forces Calculation Due to Differential Settlement				
Elastic Modulus of Concrete, E_c	SNI 2847:2019 Article 19.2.2	4700 $\sqrt{f_c'}$	MPa	25743
Cross-Section Inertia, I_g		1/12 b h^3	mm^4	1600000000

Differential Settlement, Δ	SNI 8460:2017 Article 9.2.4.3	Input (L_n/300 can be used if there is no data)	mm	2.903
Support Moment Due to Differential Settlement, M_{diff}	Hibbeler, R.C. "Structural Analysis"	$6 * E_c * I_g * \Delta / L_n^2$	kNm	22.081
Gaya Geser Tumpuan Akibat Differential Settlement, V_{diff}		$dM/dx_{(x=L)} = 12\ E_c\ I_g\ \Delta / L_n^3$	kN	7.748
Internal Forces Calculation Due to Gravity Load				
Specific Gravity of Reinforced Concrete, ρ_c		Input	kN/m^3	23.536
Uniform Load Due to Self Weight, q_{DL}		BJc * b * h	kN/m	2.824
Height Level, h_n		Input	m	4
Partition Wall Load per m^2, $q_{A,partition\ wall}$		Input (value beside can be used)	kN/m^2	2.5
Uniform Weight Due to Partition Wall Load, q_{SIDL}		$q_{A,dinding} * h_n$	kN/m	10
Ultimate Uniform Load Due to Gravitational Loads, q_D		$1.4\ (q_{DL} + q_{SIDL})$	kN/m	17.954
Support Ultimate Moment Due to Gravitational Loads, $M_{D,sup}$		$-1/12 * q_D * L_n^2$	kNm	-48.611
Span Ultimate Moment Due to Gravitational Loads, $M_{D,spa}$		$1/24 * q_D * L_n^2$	kNm	24.305
Support Ultimate Shear Force Due to Gravitational Loads, $V_{D,sup}$		$q_D * L_n / 2$	kN	51.169
Span Ultimate Shear Force Due to Gravitational Loads, $V_{D,spa}$		$q_D * L_n / 4$	kN	25.585

2.12.2 Longitudinal Reinforcement

Flexural reinforcement is deemed necessary since the axial force (Pu) is less than 0.1 times the product of the compressive strength and the cross-sectional area. The safety of the tie beams is evaluated by analyzing the current state at the negative support, positive support, negative span, and positive span. The calculation for the longitudinal reinforcement design of the tie beams is presented in Table 2.73. This table provides detailed information about the design and arrangement of the longitudinal reinforcement in the tie beams for ensuring structural integrity.

Table 2. 73 Tie Beam Longitudinal Reinforcement Calculations

Flexural Reinforcement (may be used if Axial Forces are not to be considered)				
Negative Support				
Amount of Support Negative Reinforcement, n		Input		3
d_b			mm	16
Net Spacing for Each Reinforcement		(b - 2 c_c - 2 d_s - n * d_b) / (n - 1)	mm	86.000
Net Distance Check	SNI 2847:2019 Article 25.2.1	Net Distance >= d_b and 25 mm?		OK
Number of Layers				2
As use		n *π/4 *d_b^2	mm^2	603.186
As $_{min,1}$	SNI 2847:2019 Article 9.6.1.2	$(f_c')^{0.5}$ / (4 * f_y) * b * d	mm^2	344.283
As $_{min,2}$	SNI 2847:2019 Article 9.6.1.2	1.4 / (4 * f_y) * b * d	mm^2	352.000
Check As min		As use >= As min ?		OK
a	SNI 2847:2019 Article 22.2.2.4.1	As * fy / (0.85 * fc' * b)	mm	33.116
M_n	SNI 2847:2019 Article 22.2.2.4.1	As * fy * (d - a/2)	kN-m	84.980

c	SNI 2847:2019 Article 22.2.2.4.1	a / β1	mm	39.626
ε_s	SNI 2847:2019 Article 22.2.1.2, 22.2.2.1	(d - c) / c * 0.003		0.024
ϕ	SNI 2847:2019 Table 21.2.2	0.65 <= 0.65 + (εs - 0.002) / 0.003 * 0.25 <= 0.9		0.900
ϕM_n		ϕ * Mn	kN-m	76.482
$M_{u,support}$ (-)			kN-m	70.692
Check Capacity		$\phi M_n > M_u$?		OK
Positive Support				
n		Input		2
d_b			mm	16
Net Spacing for Each Reinforcement		$(b - 2 c_c - 2 d_s - n * d_b) / (n - 1)$	mm	188.000
Net Distance Check	SNI 2847:2019 Article 25.2.1	Net Distance >= d_b dan 25 mm?		OK
Number of Layers				2
As use		$n * \pi/4 * d_b^2$	mm^2	402.124
$As_{min,1}$	SNI 2847:2019 Article 9.6.1.2	$(f_c')^{0.5} / (4 * f_y) * b * d$	mm^2	344.283
$As_{min,2}$	SNI 2847:2019 Article 9.6.1.2	1.4 / (4 * fy) * b * d	mm^2	352.000
Check As min		As use >= As min ?		OK
a	SNI 2847:2019 Article 22.2.2.4.1	As * fy / (0.85 * fc' * b)	mm	22.077
M_n	SNI 2847:2019 Article 22.2.2.4.1	As * fy * (d - a/2)	kN-m	57.586
c	SNI 2847:2019 Article 22.2.2.4.1	a / β1	mm	26.417

ε_s	SNI 2847:2019 Article 22.2.1.2, 22.2.2.1	(d - c) / c * 0.003		0.037
ϕ	SNI 2847:2019 Table 21.2.2	0.65 <= 0.65 + (εs - 0.002) / 0.003 * 0.25 <= 0.9		0.900
ϕM_n		ϕ * Mn	kN-m	51.827
M_u			kN-m	22.081
Check $\phi M_n > M_u$		$\phi M_n > M_u$?		OK
Negative Span				
n		Input		2
d_b			mm	16
Net Spacing for Each Reinforcement		(b - 2 c_c - 2 d_s - n * d_b) / (n - 1)	mm	188.000
Net Distance Check	SNI 2847:2019 Article 25.2.1	Net Distance >= d_b and 25 mm?		OK
Number of Layers				2
As use		n * π/4 *d_b^2	mm^2	402.124
As $_{min,1}$	SNI 2847:2019 Article 9.6.1.2	$(f_c')^{0.5}$ / (4 * f_y) * b * d	mm^2	344.283
As $_{min,2}$	SNI 2847:2019 Article 9.6.1.2	1.4 / (4 * fy) * b * d	mm^2	352.000
Check As min		As use >= As min ?		OK
Positive Span				
n		Input		2
d_b			mm	16
Net Spacing for Each Reinforcement		(b - 2 c_c - 2 d_s - n * d_b) / (n - 1)	mm	188.000
Net Distance Check	SNI 2847:2019 Article 25.2.1	Net Distance >= d_b and 25 mm?		OK
Number of Layers				2
As use		n *π/4 *d_b^2	mm^2	402.124
As $_{min,1}$	SNI 2847:2019 Article 9.6.1.2	$(f_c')^{0.5}$ / (4 * f_y) * b * d	mm^2	344.283

$As_{min,2}$	SNI 2847:2019 Article 9.6.1.2	1.4 / (4 * fy) * b * d	mm^2	352.000
Check As min		As use >= As min ?		OK
a	SNI 2847:2019 Article 22.2.2.4.1	As * fy / (0.85 * fc' * b)	mm	22.077
M_n	SNI 2847:2019 Article 22.2.2.4.1	As * fy * (d - a/2)	kN-m	57.586
c	SNI 2847:2019 Article 22.2.2.4.1	a / β1	mm	26.417
ε_s	SNI 2847:2019 Article 22.2.1.2, 22.2.2.1	(d - c) / c * 0.003		0.037
ϕ	SNI 2847:2019 Table 21.2.2	0.65 <= 0.65 + (εs - 0.002) / 0.003 * 0.25 <= 0.9		0.900
ϕM_n		ϕ * Mn	kN-m	51.827
M_u			kN-m	24.305
Check $\phi M_n > M_u$		$\phi M_n > M_u$?		OK

2.12.3 Transversal Reinforcement

The transversal reinforcement in the tie beam is intended to resist shear forces acting on the structure. The shear strength of the beam is determined by both the shear strength of the concrete (Vc) and the shear strength of the steel (Vs). The calculation for shear strength is performed at two critical points: the support and the span. Table 2.74 displays the transversal reinforcement calculation for the tie beam, outlining the details of the design and arrangement of reinforcement to ensure adequate shear strength.

Table 2. 74 Tie Beam Transversal Reinforcement Calculations

Shear Reinforcement				
Support				
Amount of Footing		Input		2
Av		$n * \pi/4 * d_s^2$	mm^2	157.080

Spacing		Input	mm	150
V_u			kN	58.917
ϕ	SNI 2847:2019 Article 12.5.3.2, 21.2.4			0.75
V_u / ϕ			kN	78.556
Maximum Spacing Specifier Limit	SNI 2847:2019 Table 9.7.6.2.2	$0.33 * (f_c')^{0.5} * b * d$	kN	190.870
Max Spacing 1	SNI 2847:2019 Table 9.7.6.2.2	d / 4 or d / 2	mm	176.0
Max Spacing 2	SNI 2847:2019 Table 9.7.6.2.2	300 mm or 600 mm	mm	600.0
Check Spacing				OK
V_s	SNI 2847:2019 Article 22.5.10.5.3	$A_v * f_y * d / s$	kN	154.818
Boundary V_s	SNI 2847:2019 Article 22.5.1.2	$0.66 * (f_c')^{0.5} * b * d$	kN	381.741
V_c	SNI 2847:2019 Article 22.5.5.1	$0.17 * (f_c')^{0.5} * b * d$	kN	98.327
V_n		$V_c + V_s$	kN	253.145
ϕV_n				189.859
Check Capacity		$\phi V_n >= V_u$?		OK
Span				
Amount of Footing		Input		2
Av		$n * \pi/4 * d_s^2$	mm^2	157.080
Spacing		Input	mm	150
V_u			kN	25.585
ϕ	SNI 2847:2019 Article 12.5.3.2, 21.2.4			0.75
V_u / ϕ			kN	34.113

Maximum Spacing Specifier Limit	SNI 2847:2019 Table 9.7.6.2.2	$0.33 * (f_c')^{0.5} * b * d$	kN	190.870
Max Spacing 1	SNI 2847:2019 Table 9.7.6.2.2	d / 4 or d / 2	mm	176.0
Max Spacing 2	SNI 2847:2019 Table 9.7.6.2.2	300 mm or 600 mm	mm	600.0
Check Spacing				OK
V_s	SNI 2847:2019 Article 22.5.10.5.3	$A_v * f_y * d / s$	kN	154.818
Boundary V_s	SNI 2847:2019 Article 22.5.1.2	$0.66 * (f_c')^{0.5} * b * d$	kN	381.741
V_c	SNI 2847:2019 Article 22.5.5.1	$0.17 * (f_c')^{0.5} * b * d$	kN	98.327
V_n		$V_c + V_s$	kN	253.145
ϕV_n				189.859
Check Capacity		$\phi V_n >= V_{u\ ?}$		OK

2.12.4 Conclusion

The summary of the tie beam reinforcement setup is presented in Table 2.75. The reinforced concrete tie beam has a cross-sectional dimension of 300 x 400 mm. The reinforcement features a hook length of 75 mm, and the bending angle is set at 135°. This table provides a consolidated overview of the key details regarding the configuration of reinforcement for the tie beam.

Table 2. 75 Recapitulation of Tie Beam Reinforcement

Reinforcement Recap		
Longitudinal	Support Top	3D16
	Support Bottom	2D16
	Span Top	2D16
	Span Bottom	2D16
Transversal	Support	1D10-150
	Span	1D10-150
Transversal Hook Length		

Hook Type	135	°
l_{ext}	75	mm

2.13 Column Design

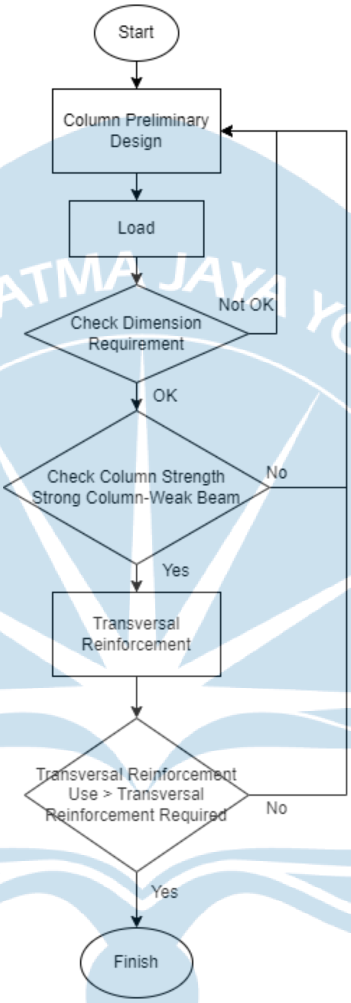


Figure 2. 28 Column Design Flowchart

The column design for the SRPMK building must adhere to the requirements outlined in SNI 28147:2019 chapter 18.7. The initial design dimensions for the column are detailed in Table 2.76. The column design process is visually illustrated in Figure 2.28, providing a step-by-step depiction of the design considerations and calculations involved in determining the column dimensions.

Table 2. 76 Material and Section Properties of Column

Parameter	**Equation**	**Unit**	**Value**
Column Length/Height, L	Input	mm	4000

Parameter	Equation	Unit	Value
Column Short Side, b	Input	mm	800
Column Long Side, h	Input	mm	800
Longitudinal Reinf. Diameter, d_b	Input	mm	25
Stirrup Diameter, d_s	Input	mm	13
Concrete Cover, c_c	Input	mm	40
Concrete Compressive Strength, f_c'	Input	MPa	30
Longitudinal Reinf. Yield Strength, f_y	Input	MPa	420
Stirrup Yield Strength, f_{yv}	Input	MPa	420
Beam Height, h_b	Input	mm	600
L_n	L - hb	mm	3400

It is essential to verify that the column's dimensions comply with the criteria specified in SNI 2847:2019. The variables to consider include the width-height ratio (b/h), steel area (Ast), column section area (Ag), axial force (Pu), and compressive strength of the concrete (fc'). When the axial load acting on a column exceeds 30% of the column's cross-sectional area multiplied by its compressive strength, SNI 2847:2019 must be used for consideration and analysis of the axial load. The force and geometry checks are presented in Table 2.77, outlining the verification process for these critical parameters.

Table 2. 77 Force and Geometry Requirement

Parameter	SNI 2847:2019	Equation	Value	Note
Axial Force Requirement	Not required. Read R18.7.1	Pu > 0.1 A_g f_c'?	Pu=2685.77 kN > 1920 kN	OK
Shortest Side Requirement	18.7.2.1	b >= 300 mm?	b=800 mm > 300 mm	OK
Section Dimension Ratio Requirement	18.7.2.1	b/h >= 0.4?	b/h=1 > 0.4	OK

2.13.1 Internal Forces

The dead load, live load, and seismic load are the internal forces in the column. The upper column, planned column, and lower column loads originate from the MIDAS Output. Table 2. 78 presents the internal forces for moments and axial force. The internal shear force is displayed in Table 2. 79.

Table 2. 78 Internal Axial Force and Moment

Condition	P (kN)	M2 (kN-m)	M3 (kN-m)
P max	84.35	0	0
P min	-2685.77	-9.39	56.46
M2 Max	-247.46	472.19	194.81
M2 Min	-1026.59	-446.62	-107.87
M3 Max	-392.12	188.37	498.37
M3 Min	-441.85	-189.56	-683.7

Table 2. 79 Internal Shear Force

Support	
V2 (kN)	-681.420
V3 (kN)	-266.550
Field	
V2 (kN)	-143.400
V3 (kN)	-213.910

The SP Column is then used to verify the internal axial force of the column in order to determine the minimum necessary steel reinforcement ratio. Table 2. 80 displays the ratio of steel reinforcement checking.

Table 2. 80 Internal Forces for Axial Checking from SP Column

Parameter	SNI 2847:2019	Equation	Unit	Value
Number of Reinforcements, n		Input		16
Longitudinal Reinf. Area, A_s		$n * \pi/4 * d_b^2$	mm^2	7854
Reinforcement Ratio, ρ		As / (b * h)		1.23%
ρ_{min} and ρ_{max} Check	18.7.4.1	1% <= ρ <= 6%		OK

2.13.2 Strong Column-Weak Beam Requirement

In SRPMK construction according to SNI 2847:2019 chapter 18.7.3.2, a prerequisite is that the column must be stronger than the beam. This ensures that the building's earthquake resistance starts from the foundation and progresses through the beam-column joint, column, and beam. The criterion for satisfaction is $\sum Mnc \geq (1.2)\sum Mnb$. If this criterion is met, the first potential failure is expected to be in the beam, which is considered less hazardous than failures in the column, beam-column joint, or foundation.

As depicted in Figure 2.29, the longitudinal reinforcement of the column needs to be examined considering positive moment (Mn+), negative moment (Mn-), and bottom edge moment (Mnc) of the column. Table 2.81 presents the strong column-weak beam checking, ensuring compliance with the specified requirements.

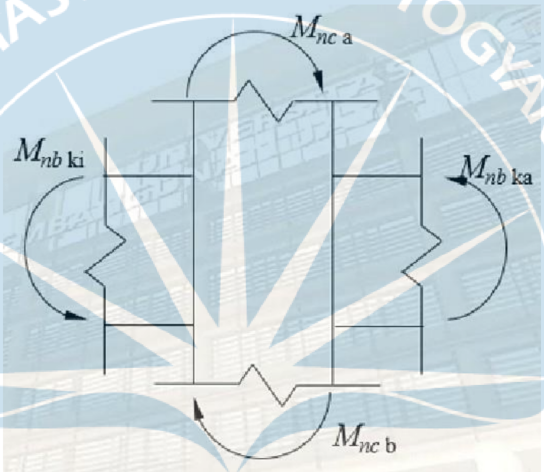


Figure 2. 29 Moment acts on column

Table 2. 81 Strong Column-Weak Beam Check

Parameter	**SNI 2847:2019**	**Equation**	**Unit**	**Value**
Column Nominal Moment, M_{nc}		Input (M_n from P_{max} dan P_{min} condition)	kN m	1190.4
M_n^- Beam Support		Input	kN m	659.02
M_n^+ Beam Support		Input	kN m	494.54
SCWB Check	18.7.3.2	$2 * M_{nc} >= 1.2 * (M_n^- + M_n^+)$		OK

From the calculation, it shows that the column and beam relationship is strong column-weak beam because $\sum Mnc = 2381$ kNm and $(1.2)\sum Mnb = 1384.3$ kNm.

2.13.3 Plastic Hinge Zone

The development of the column's plastic hinge is a critical consideration. Therefore, it is essential to determine the length of the column's plastic hinge zone and provide sufficient support. Table 2.82 presents the length of the plastic hinge, providing important information for the thoughtful design and support of the plastic hinge in the column.

Table 2. 82 Plastic Hinge Zone

Parameter	SNI 2847:2019	Equation	Unit	Value
l_{o1}	18.7.5.1	h	mm	800.0
l_{o2}	18.7.5.1	L_n / 6	mm	566.7
l_{o3}	18.7.5.1	450 mm	mm	450
l_o	18.7.5.1	Max (l_{o1}; l_{o2}; l_{o3})	mm	800.0

There needs to be sufficient transversal reinforcement for the column's plastic hinge. As earthquakes can occur in two directions, this is explained in detail for the minor and main axes of the column. The spacing and configuration of confinement have an impact on this. As a result, a precise design is required. The transversal reinforcement design and spacing at plastic hinge length are displayed in Table 2. 83.

Table 2. 83 Plastic Hinge/Support Transversal Reinforcement

Parameter	SNI 2847:2019	Equation	Unit	Value
Number of Short Side Legs, n1		Input		4
Number of Long Side Legs, n2		Input		4
Spacing, s		Input	mm	100
Largest Leg Spacing, $x_{i\ max}$	R18.7.5.2	Input	mm	300
A_{sh} 1		$n *\pi/4 *d_s^2$	mm^2	530.93
A_{sh} 2		$n *\pi/4 *d_s^2$	mm^2	530.93
A_{sh} / s, 1			mm^2 / mm	5.31
A_{sh} / s, 2			mm^2 / mm	5.31

Parameter	SNI 2847:2019	Equation	Unit	Value
Plastic Hinge Zone Confinement				
Concrete Core Section Width, b_c	R18.7.5.2	$b - 2c_c$	mm	720
Concrete Core Section Length, h_c	R18.7.5.2	$h - 2c_c$	mm	720
Column Section Area, A_g		$b * h$	mm^2	640000
Concrete Core Section Area, A_{ch}		$b_c * h_c$	mm^2	518400
Short Side/Minor Axis				
A_{sh}/s min, 1	18.7.5.4	$0.3 (b_c * f_c' / f_{yv}) * (A_g / A_{ch} - 1)$	mm^2	3.62
A_{sh}/s min, 2	18.7.5.4	$0.09 * b_c * f_c' / f_{yv}$	mm^2	4.63
Cek A_{sh}/s 1		A_{sh}/s 1 >= Ash/s min ?		OK
Long Side/Major Axis				
A_{sh}/s min, 1	18.7.5.4	$0.3 (h_c * f_c' / f_{yv}) * (A_g / A_{ch} - 1)$	mm^2	3.62
A_{sh}/s min, 2	18.7.5.4	$0.09 * h_c * f_c' / f_{yv}$	mm^2	4.63
Cek A_{sh}/s 2		A_{sh}/s 2 >= Ash/s min ?		OK
Spacing Check				
$s_{max,1}$	18.7.5.3	$b / 4$	mm	200
$s_{max,2}$	18.7.5.3	$6 * d_b$	mm	150
h_x	18.7.5.3	$x_{i\ max}$	mm	300
$s_{max,3} = s_o$	18.7.5.3	100 <= 100 + (350 - hx) / 3 <= 150	mm	116.67
s_{max}	18.7.5.3	Min (s_{max1}, s_{max2}, s_{max3})	mm	116.67
Spacing Check				OK

According to SNI 2847:2019, the shear strength at the plastic hinge zone should consider the potential increase in yield strength up to 1.25 times the initial yield strength. Achieving the greatest probable moment involves enhancing the yield. Figure 2.30 illustrates the use of SP Column to determine this. The greatest moment capacity, 1799 kNm, is divided by the reduction factor, 0.871, resulting in 2065 kNm, which represents the probable moment. The shear strength at the plastic hinge zone is presented in Table 2.84, taking these considerations into account.

Factored Loads and Moments with Corresponding Capacities

No	Pu	Mux	Muy	ϕMnx	ϕMny	ϕMn/Mu	NA Depth	dt Depth	εt	ϕ
	kN	kNm	kNm	kNm	kNm		mm	mm		
1	-84.35	0.00	0.00	1290.35	0.00	999.999	125	738	0.01467	0.900
2	2685.77	-9.39	56.46	-299.20	1799.02	31.864	335	861	0.00473	0.871
3	247.46	472.19	194.81	1249.02	515.31	2.645	277	934	0.00716	0.900
4	1026.59	-446.62	-107.82	-1495.66	-361.07	3.349	269	876	0.00685	0.900
5	392.12	188.37	498.37	489.69	1295.58	2.600	278	925	0.00707	0.900
6	441.85	-189.56	-683.70	-373.93	-1348.70	1.973	250	885	0.00774	0.900

Figure 2. 30 Largest Moment Capacity

Table 2. 84 Plastic Hinge Zone/Support Shear Strength with $f_{pr} = 1.25 f_y$

Parameter	SNI 2847:2019	Equation	Unit	Value
Column M_{pr}		Input, (largest value)	kN m	2065.465
$V_{u\,1}$	18.7.6.1	2 * M_{pr} Column / Ln	N	1214979
Shear Force Result of Structural Analysis				
Vu 2, Minor Axis		From Internal Forces	N	681420
Vu 2, Major Axis		From Internal Forces	N	266550
Minor Axis Concrete Shear Resistance				
V_u		Max (V_{u1}, V_{u2})	N	1214979
ϕ	Table 21.2.1			0.75
V_c	22.5.6.1	$0.17\ (1 + N_u/(14\ A_g))\ (f_c')^{0.5}\ h\ d$; $d = b - c_c - d_s - d_b / 2$	N	547131
Vs Req	22.5.10.1	$V_u / \phi - V_c$	N	1072842
As/s Req	22.5.10.5.3	$V_s / (f_{yv} * d)$; $d = b - c_c - d_s - d_b / 2$	mm^2 / mm	3.48
A_s/s Min 1	10.6.2.2	$0.062\ (f_c')^{0.5}\ h / f_{yv}$	mm^2 / mm	0.65
A_s/s Min 2	10.6.2.2	$0.35\ h / f_{yv}$	mm^2 / mm	0.67
A_s/s Check		Ash/s 1 >= Max (As/s Req, As/s Min) ?		OK
Major Axis Concrete Shear Resistance				
V_u		Max (V_{u1}, V_{u2})	N	1214979
ϕ	Table 21.2.1			0.75

Parameter	SNI 2847:2019	Equation	Unit	Value
V_c	22.5.6.1	$0.17\ (1 + N_u/(14\ A_g))\ (f_c')^{0.5}\ b\ d;\ d = h - c_c - d_s - d_b / 2$	N	547131
Vs Req	22.5.10.1	$V_u / \phi - V_c$	N	1072842
As/s Req	22.5.10.5.3	$V_s / (f_{yv} * d);\ d = h - c_c - d_s - d_b / 2$	mm^2 / mm	3.48
A_s/s Min 1	10.6.2.2	$0.062\ (f_c')^{0.5}\ b / f_{yv}$	mm^2 / mm	0.65
A_s/s Min 2	10.6.2.2	$0.35\ b / f_{yv}$	mm^2 / mm	0.67
A_s/s Check		Ash/s 2 >= Max (As/s Req, As/s Min) ?		OK

2.13.4 Outside Plastic Hinge Zone

The maximum spacing and steel area for transversal reinforcement are provided by SNI 2847:2019 outside the plastic hinge zone or at the bridge position. Table 2. 85 displays the design for the transversal reinforcement of the column at the span position, and Table 2. 86 shows the results of the shear strength check at the column span.

Table 2. 85 Out of Plastic Hinge Zone/Span Transversal Reinforcement

Parameter	SNI 2847:2019	Equation	Unit	Value
Number of Short Side Legs, n1		Input		2
Number of Long Side Legs, n2		Input		2
Spacing, s		Input	mm	150
Av Minor Axis		$n *\pi/4 *d_s^2$	mm^2	265.46
Av Major Axis		$n *\pi/4 *d_s^2$	mm^2	265.46
Max Spacing 1	18.7.5.5	$6\ d_b$	mm	150.0
Max Spacing 2	18.7.5.5	150 mm	mm	150.0
Spacing Check		Spacing <= Max Spacing?		OK

Table 2. 86 Out of Plastic Hinge Zone/Span Shear Strength

Parameter	SNI 2847:2019	Equation	Unit	Value
Minor Axis Concrete Shear Resistance				

Parameter	SNI 2847:2019	Equation	Unit	Value
V_u		From Internal Forces	N	143400
ϕ	Table 21.2.1			0.75
V_c	22.5.6.1	$0.17\ (1 + N_u/(14\ A_g)]\ (f_c')^{0.5}$ $h\ d;\ d = b - c_c - d_s - d_b\ /\ 2$	N	547131
Vs Req	22.5.10.1	Max $(V_u/\phi - V_c;\ 0)$		0
Av/s Req	22.5.10.5.3	$V_s\ /\ (f_{yv} * d);\ d = b - c_c - d_s - d_b\ /\ 2$		0
A_s/s Min 1	10.6.2.2	$0.062\ (f_c')^{0.5}\ b\ /\ f_{yv}$	mm^2 / mm	0
A_s/s Min 2	10.6.2.2	$0.35\ b\ /\ f_{yv}$	mm^2 / mm	0
A_s/s Check		Av/s >= Av/s Req?		OK
Major Axis Concrete Shear Resistance				
V_u		From Internal Forces Sheet	N	213910
ϕ	Table 21.2.1			0.75
V_c	22.5.6.1	$0.17\ (1 + N_u/(14\ A_g)]\ (f_c')^{0.5}$ $b\ d;\ d = h - c_c - d_s - d_b\ /\ 2$	N	547131
Vs Req	22.5.10.1	Max $(V_u/\phi - V_c;\ 0)$		0
Av/s Req	22.5.10.5.3	$V_s\ /\ (f_{yv} * d);\ d = h - c_c - d_s - d_b\ /\ 2$		0
A_s/s Min 1	10.6.2.2	$0.062\ (f_c')^{0.5}\ b\ /\ f_{yv}$	mm^2 / mm	0
A_s/s Min 2	10.6.2.2	$0.35\ b\ /\ f_{yv}$	mm^2 / mm	0
Check A_s/s		Av/s >= Av/s Req?		OK

2.13.5 Conclusion

Table 2.87 presents the column reinforcement configuration based on the moment and shear force checking to produce strong column-weak beam.

Table 2. 87 Column Reinforcement Recap

Longitudinal Reinforcement	
Longitudinal	16 D25
Transversal/Stirrup Reinforcement at Support	
Minor Axis	4D13-100
Major Axis	4D13-100
Transversal/Stirrup Reinforcement at Span	
Minor Axis	2D13-150
Major Axis	2D13-150

2.14 Beam-Column Joint

The beam-column joint in an SRPMK building needs to be built with transversal reinforcement and a development length for the longitudinal rebar. Table 2. 88 presents the beam-column parameters that were determined in the earlier preliminary design. Properties of Beam-Column. The effective area of the beam-column joint is displayed in Table 2. 89.

Table 2. 88 Beam-Column Parameters

Beam-Column Properties SRPMK1		
bb	350	mm
hb	600	mm
Lb	8000	mm
bc	800	mm
hc	800	mm
Lc	4000	mm
fc'	30	Mpa
fy	420	Mpa
λ	1	

Beam-Column Parameters SRPMK2

bb	600	mm
hb	650	mm
Lb	8000	mm
bc	1000	mm
hc	1000	mm
Lc	4000	mm
fc'	30	Mpa
fy	400	Mpa
λ	1	

Table 2. 89 Effective Dimension of Beam-Column

Beam Column Properties			
b		mm	SNI 2847:2019 chapter18.8.4.3
x	225	mm	
hj	800	mm	
b+h	1025	mm	
b+2x	800	mm	
bj	800	mm	
Aj	64000	mm2	SNI 2847:2019 figure 18.8.4

2.14.1 Reinforcement

While the longitudinal reinforcement is merely an extension of the column's longitudinal reinforcement, the reinforcement for the beam-column connection is not intended for confined relation. Nonetheless, only longitudinal reinforcement need to be examined. The reinforcement strength computation is shown in Table 2. 90.

Table 2. 90 Beam-Column Transversal Requirements

Check Transversal Requirements			
Description	Value	Unit	Note
lo	800	mm	SNI 2847:2019 figure 18.9.2.2
Confirmed Beams	0	beams	
bb>=3/4bc	OK		
Used Confinement = Column Support			SNI 2847:2019 chapter 18.8.3.2 at plastic hinge

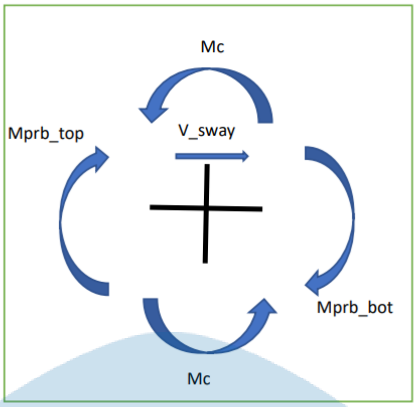


Figure 2. 31 Shear Force on Column due to Probability Moment of Beam and Column

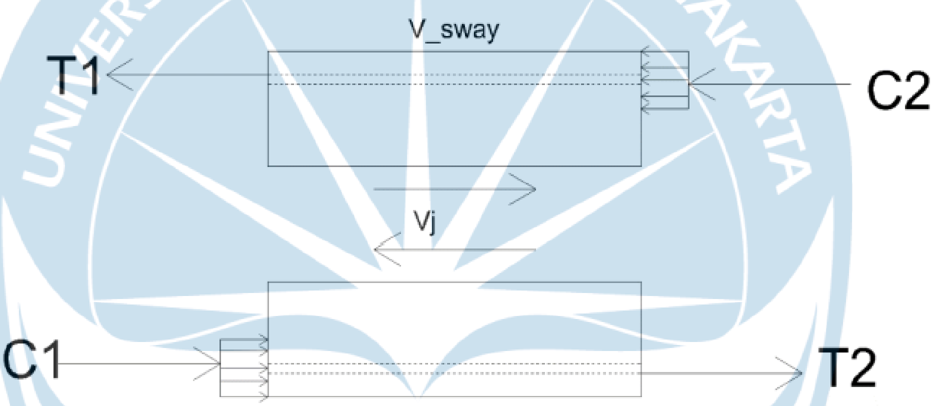


Figure 2. 32 Tension (T) and Compression on Beam-Column Joint

Table 2. 91 Shear Strength Check

Check Shear Strength			
Mprb top	787.3168327	kNm	
Mprb bot	599.5757832	kNm	
DF	0.5		Distribution factor
Mc	693.446308	kNm	
Vsway	510.880	kN	
fy	420	Mpa	
Top Longitudinal Rebar			
As	3436.117	mm2	
T1	1803.961407	kN	
C1	1803.961407	kN	
Bottom Longitudinal Rebar			

As	2454.369	mm2	
T2	1288.543862	kN	
C2	1288.543862	kN	
Vj	2581.625329	kN	max(T+C-Vsway)
Vn	3505.424368	kN	
φ	0.85		
φVn	2979.610713	kN	
Check φVn>Vj	OK		

Table 2. 92 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'))	338.1003441	mm	SNI 2847:209 eq. 18.8.5.1
ldh	355.005	mm	
Take Development Length	360	mm	
12db	300	mm	

2.14.2 Conclusion

Table 2. 93 Reinforcement Recap on Beam-Column Joint

Recap		
Longitudinal	7D25	
Confinement	4D13	
Development Length, *ldh*	360	mm
Hook Length	300	mm

Table 2. 94 Transversal Hook Length

Transversal Hook Length		
Hook Type	135	O
1ext	150	mm

The distance between each longitudinal bar in a beam-column joint must be less than 350 mm, per SNI 2847:2019 figure R18.7.5.2. This space needs to be verified using the drawing and shown in Table 2. 95.

Table 2. 95 Check Rebar Spacing

Check xiy and xix		Note
xiy	255	From Drawing
xix	255	From Drawing
Check xiy	OK	
Check xix	OK	

2.15 Slab Design

This design is based on SNI 2847:2019 and the PBI'71 Slab Moment Table Method. It is necessary to choose between a one-way slab and a two-way slab for the slab's dimension. A one-way slab is one that has a lengthy span that is greater than twice as long as it is short. A two-way slab is one with a long span that is not twice as long as its short span. While reinforcement in a two-way slab is placed in both directions, reinforcement in a one-way slab is only done in one direction. The following options are available for slab design:

2.15.1 Determination of Dimension and Slab Type

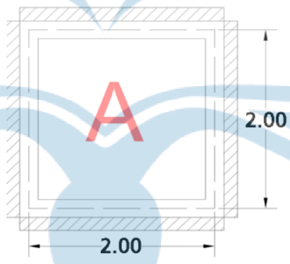


Figure 2. 33 Slab Type A

The slab will be checked for one way or two ways. Given that the long span (Ly) and short span (Lx) are both 2 meters, the long span to short span ratio is 1 and the structure is a two-way slab. This interior slab is enclosed on all four sides by beams. The slab is 140 mm in thickness. According to SNI 2847:2019 table 20.6.1.3.1, the concrete cover of the slab is 40 mm.

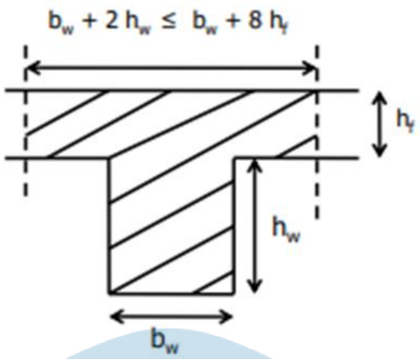


Figure 2. 34 Interior Slab-Beam Cross-Section

Table 2. 96 Slab Load

Parameter	Value	Unit
Live Load	4.79	kN
Self-Weight	3.5	kN/m2
Reinforced		
Ceramic	0.2	kN/m2
Ceiling	0.18	kN/m2
Total DL	3.88	kN/m2
Comb Load 1.2D + 1.6L	12.32	kN/m2

2.15.2 Shear Strength

The shear force (Vu) that acts on the slab is $\frac{1.15quXln}{2}$ = 38.2536 kN, the clear distance from top slab to the middle of steel reinforcement in short span (dx) is 666 mm, and the clear distance of long span steel reinforcement (dy) is 658 mm. The shear strength of slab is with for shear. Because the shear strength is larger than the shear force, it is safe.

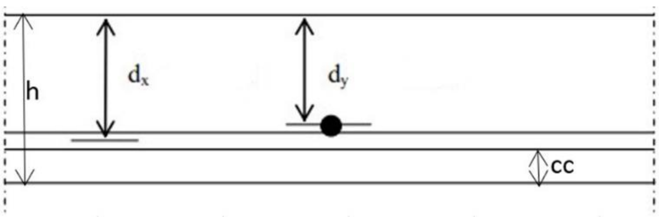


Figure 2. 35 Clear Distance Dx and Dy on Slab

2.15.3 Moment Strength

By multiplying the moment load by the coefficient in PBI 1971 table 13.3.1, it is possible to determine the moment load in the slab. Short span moment (mlx), long span moment (mly), short support moment (mtx), and long support moment (mty) all have coefficients of 21, 21, and 52, respectively. The slab is subject to the moment $M = 0.01q_u\ l_2 = 0.1405$ kNm. The previously mentioned coefficient moment will be added to this moment.

To calculate the main reinforcement, the coefficient of flexural resistance (k) must be calculated first. The general eq. for k showed in Figure 2. 36, with Mu is mlx, mly, mtx, or mty and d is dx or dy. The values of Klx, Kly, Ktx, and Kty are 0.0259, 0.0266, 0.0642, and 0.0658, respectively.

$$k_{lx} = \frac{M_{lx}}{\emptyset b {d_x}^2}$$
$$k_{ly} = \frac{M_{ly}}{\emptyset b {d_y}^2}$$
$$k_{tx} = \frac{M_{tx}}{\emptyset b {d_x}^2}$$
$$k_{ty} = \frac{M_{ty}}{\emptyset b {d_y}^2}$$

Figure 2. 36 General Equation for Coefficient of Flexural Resistance (k)

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 (ØMn) of ØMnlx, ØMnly, ØMntx, and ØMnty is 65.7729 kNm, 65.7729 kNm, 66.9753 kNm, and 66.9753 kNm, respectively with $\phi = 0.9$. This is still bigger than moment load mlx, mly, mtx, and mty which is 4.1395 kNm, 4.1395 kNm, 10.2502 kNm, and 10.2502 kNm, respectively.

2.15.4 Reinforcement

Calculating the spacing for reinforcement uses the formula $\frac{0.25\pi D^2 b}{As_{req}}$. The reinforcement spacing is 280.4993 mm for short span (slx), long span (sly), short support (stx), and long support (sty). To make the steel arrangement easier to 50 mm down, this space must be rounded down. According to SNI 2847:2019 chapter

8.7.2.3, the maximum separation is 450 mm and three times the slab thickness (3h). D10-250 is the reinforcement used for Dlx, Dly, Dtx, and Dty.

2.15.5 Conclusion

The slab type A of 2000 x 2000 mm will use fc'25 MPa and fy 400 MPa with reinforcement as in Table 2. 97 Slab Reinforcement.

Table 2. 97 Slab Reinforcement

Slab Reinforcement	
Dlx	D10-250
Dly	D10-250
Dtx	D10-250
Dty	D10-250

2.16 Stair Design

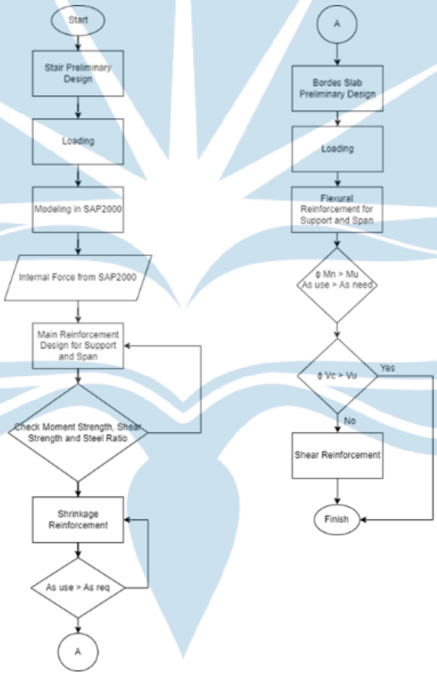


Figure 2. 37 Stair Design Flowchart

For every story in the building, there are stairs. The design of stairs accommodates both the stair and the slab (landing slab). Reinforced concrete will be used for the stair and landing slab. Figure 2. 37 depicts the process for designing stairs and landing slab slabs.

2.16.1 Stair Data

The data of stair design is presented as follows:

- Height of each floor (Hlt) = 4 m
- Stair width = 2 m
- Landing slab width = 2 m
- Optrede (Op) = 0.16 m
- Antrede (An) = 0.3 m
- Number of stair (Ntg) = 24 stairs
- Stair length (Ltg) = 2.88 m
- Stair slope (α) = 28.0724
- Thickness of the stair = 0.15 m
- Thisness of landing slab = 0.25 m
- Stair concrete cover = 30 mm
- Landing slab concrete cover = 40 mm
- Fc' = 25 MPa
- Fy = 370 MPa
- Equivalent thickness of the stair (tt) = 0.070588235 m = 70.588235 mm
- Total equivalent of the stair (t') = 0.25 m = 250 mm
- Weight of concrete = 24 kN/m3
- Weight of tile = 21 kN/m3

2.16.2 Loading

Self-weight, tiles, railing, and live load all contribute to the weight of the steps and the border slab.

Table 2. 98 Stair Loading

Stair Loading (qtg)		
-Slab Weight	6	kN/m
-Tiles and Plaster	1.05	kN/m
-Railing Weight	1	kN/m
Total Dead Load (qd)	7.37	kN/m
Live Load (qL)	4.79	kN/m

Table 2. 99 Landing Slab Loading

Landing Slab Loading (qbd)		
-Slab Weight	3.6	kN/m
-Tiles and Plaster	1.05	kN/m
-Railing Weight	1	kN/m
Total Dead Load (qd)	5.0737	kN/m
Live Load (qL)	4.79	kN/m

2.16.3 Reinforcement

Midas Gen is used to obtain the internal force of staircases. Maximum moments due to dead and live loads are 1.21 kNm and 4.79 kNm respectively. Maximum shear forces due to dead and live loads are 1.473 kN and 4.79 kN respectively. For Mur = 26.7 kNm and Vur = 38.79 kN, 1.2D + 1.6L is the greatest combination that can be used. Assuming that the maximum moment is 0.5 Mur, Mu for the support site is equal to 13.36 kNm. The maximum moment is taken as 0.8Mur for the span position, making Mu = 19.49 kNm.

	Mu (kNm)	**D reinf. (mm)**	**ϕMn (kNm)**	**Vu (kN)**	**ϕVc (kN)**	**Shrinkage reinf. (mm)**
Support	26.7	D13-175	29.37	38.79	119.5	D8-D125
Span	13.36	D13-400	13.25			

2.16.4 Landing Slab Data

Below in Table 2. 100 are presented the dimension and properties of landing slab. Table 2. 101 presents the load of stair landing slab.

Table 2. 100 Landing Slab Dimension and Properties

Parameter	Equation	Value	Unit
Landing Slab long span (Ly)	=	2	m
Landing Slab short span (Lx)	=	2	m
Landing Slab thickness (hb)	=	160	mm
d reinforcement support (dbs)	=	13	mm
d reinforcement span (dbf)	=	13	mm
d distribution/shrinkage (dsh)	=	8	mm
concrete cover (cc)	=	40	mm
d	hb-cc-1/2*db =	113.5	mm

Parameter	Equation	Value	Unit
Fc	=	30	MPa
Fy main	=	400	MPa
Fy distribution	=	400	MPa
Es	=	200000	MPa
Light brick Unit Weight	=	650	kg/m3
		6.3765	kN/m3
β	=	0.835714286	
єty main	Fy main/Es =	0.002	
єty main	Fy distr/Es =	0.002	

Table 2. 101 Landing Slab Load

Parameter	Equation	Value	Unit
Dead Load			
Slab weight	ts*wc =	3.6	kN/m2
Tiles and Plaster Weight	ttl*wt+tp*wcm =	1.473792	kN/m2
Total Dead Load (qD)		5.073792	kN/m2
Wall Height	=	0.96	m
Wall Thickness	=	0.15	m
Wall Weight	=	0.918216	kN/m
Live Load	=	4.79	kN/m2

2.16.5 Landing Slab Reinforcement

Table 2. 102 shows the reinforcement arrangement for stairs and the corresponding capacities.

Table 2. 102 Reinforcement Recap and Capacity

	Mu (kNm)	D reinf. (mm)	ϕMn (kNm)	Vu (kN)	ϕVc (kN)	Shrinkage reinf. (mm)
Support	26.7	D16-450	29.66	38.79	119.5	D8-100
Span	13.36	D12-550	13.44	19.49	120.9	D8-100

2.16.6 Landing Beam Data

The data for landing beam is shown in Table 2. 103 and the loads are in Table 2. 104.

Table 2. 103 Landing Beam Data

Parameter	Equation	Value	Unit
b	=	200	mm
h	=	200	mm
Ag	b*h =	40000	mm2
Fc	=	30	MPa
Ec	4700*sqrt(fc) =	25742.9602	MPa
Fy	=	420	MPa
Es	=	200000	MPa
ety	fy/Es =	0.0021	
db support	=	19	mm
db span	=	19	mm
dv	=	8	mm
cc	=	40	mm
d	=	142.5	mm
β	=	0.835714286	
λ	=	1	

Table 2. 104 Landing Beam Load

Parameter	Equation	Value	Unit
Tile and Plester		1.473792	kN/m
Wall Weight	=	0.918216	kN/m
Live Load	=	4.79	kN/m2
Total Live	=	9.58	kN/m

2.16.7 Landing Beam Reinforcement

The reinforcement arrangement for landing beam and the corresponding capacities are presented in Table 2. 105

Table 2. 105 Landing Beam Reinforcement Recap

	Mu (kNm)	D reinf. (mm)	ϕMn (kNm)	Vu (kN)	ϕVc (kN)	Shrinkage reinf. (mm)
Support	26.7	4D19	29.66	38.79	29.2	D8-100
Span	13.36	4D19	29.66	19.49	21.7	D8-100