CHAPTER II
DESIGN OF UPPER STRUCTURE

2.1 General Description of The Building

Upper structure design is one of the most essential steps in the construction because it will result an economical, safe, and strong building (Nugroho et al.,2020). To achieve those objections, it is necessary to understand the general information of the building that will be planned.

It will be modelized a three-story building that will be design according to Special Moment Frame (SRPMK) based on SNI 1726:2019. The building will be located in Jalan Kenari, Kecamatan Umbulharjo, Kota Yogyakarta, Daerah Istimewa Yogyakarta and functioned as library. Therefore, seismic design category that should be chose is KDS IV.

Generally, the building uses reinforced concrete as main material of the structure. The building will be modelized using MIDAS Gen. Tables and graphics below shows the description of the building.

Table 2.1 Structure Data of The Building

Function	: Library
Location	: Yogyakarta
Total floors	: 3 floors
Height of Floor	: 17.25 m

Table 2.2 The Height of Each Floors

Floor	**Height of Floor (m)**
1^{st} Floor	4
2^{nd} Floor	4
3^{rd} Floor	4
Roof	5.25

Description below shows the specification of the used materials, including the quality of the concrete and its reinforcement.

Quality of Steel Reinforcement (f_y)	= 420 MPa
Quality of Concrete (fc’)	= 30 MPa
Modulus of Elasticity (E)	= 4700 $\sqrt{fc'}$ (MPa)
	= 25742.9602 MPa

2.2 The Information of Load

2.2.1 Gravitational Load

Gravitational load consists of dead load (DL), Super-Imposed Dead Load (SIDL), and Live Load (LL) that assigned based on SNI 1727:2020.

a. Dead Load (DL)

Dead loads (DL) are weight of all structural components, including slabs, beams, and columns. Dead loads, in this modelling, are calculated automatically using MIDAS Gen software, using unit weight of 24 kN/m^3.

b. Super-Imposed Dead Load (SIDL)

Super Imposed Dead Load (SIDL) are weight of non-structural components (usually contains MEP and architectural components). In this design, SIDL that are used are:

Ceiling and its frame	: 0.18 kN/m^2
MEP	: 0.25 kN/m2
Walls (batako)	: 2 kN/m2
Spacy (1 cm) + Sand	: 0.8063 kN/m2 (for Floor plate)
	: 0.3221 kN/m2 (For deck)
Ceramics	: 0.24 kN/m2
Waterproofing	: 0.28 kN/m2

c. Live Load (LL)

Live loads are moving load due to the usage of the users of the building due to

check the irregularities oving components, like goods or occupant of the building. In this design, SNI 1727:2020 was used as a guidance according to the room functional. Reduction of live load is also allowed for any live load with minimum of 4.79 kN/m^2. Magnitude of reduction that is allowed is 25%.

2.2.2 Earthquake Load

Earthquake loads in the construction of building used the response spectrum analysis, according to the location of the building. The parameter of the response spectrum is obtained from soil data, then inputted to Desain Spektra Indonesia-RSA by Kementerian Pekerjaan Umum dan Perumahan Rakyat Indonesia, that can be accessed on their official website (Ciptakarya, 2021).

Table 2.3 Parameter Response Spectrum

Site Classification		IV
Importance of Earthquake	I_e	1.5
Spectral acceleration to the mapped maximum earthquake for short period	Ss	1.1070
Spectral acceleration to the mapped maximum earthquake for 1 second period	S1	0.5070
Amplification factor for short period	Fa	1.0572
Amplification factor for 1 second period	Fv	1.7930
Acceleration for short period	Sms	1
Acceleration for 1 second period	Sm1	0.507
Designed acceleration for short period	Sds	0.78
Designed acceleration for 1 second period	Sd1	0.6060 g
Coefficient Modification Factor	R	8
3Overstrength Factor	Ωo	3

Deflection Enlarge Factor	Cd	5.5

2.2.3 Wind Load

Calculation of Roof Pressure Coefficient

h (average elevation) $= \frac{1}{2}$ x (Ring Balk Elevation + Roof Top Elevation)

$= \frac{1}{2}$ x (12 m +17.25 m)

= 14.625 m

L (width of the building) = 55.5

h/L = 0.26

Roof Pressure coefficient is calculated based on SNI 1727-2020. Double interpolation is conducted, with h/L interpolation as the first step, continued by α interpolation.

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

Arah Angin	***h/L***	**Jarak horizontal dari tepi sisi angin datang**	**C_p**
Tegak lurus terhadap bubungan untuk $\theta < 10^0$ sejajar bubungan untuk semua θ	≤0,5	0 sampai dengan *h*/2	-0,9, -0,18
		h/2 sampai dengan *h*	-0,9, -0,18
		h sampai dengan 2*h*	-0,5, -0,18
		> 2*h*	-0,3, -0,18
	≥1,0	0 sampai dengan *h*/2	-1,3[b], -0,18
		>*h*/2	-0,7, -0,18

[a]Nilai disediakan untuk keperluan interpolasi.
[b]Nilai dapat direduksi secara linier dengan luas yang sesuai berikut ini:
[c]Untuk kemiringan atap lebih besar dari 80°, gunakan C_p = 0,8

Figure 2.1 Wind Load Coefficient

By h/L interpolation, then continued by α Interpolation coefficient for the wind-face that will be calculated is -0.200 and 0.298. For purlin, tension coefficient (Cp) can be taken as 0.298. As a result, wind face can be computed as:

Wind-face:

$G \times Cp = 0.85 \times 0.298 = 0.253$

Wind-back:

$G \times Cp = 0.85 \times (-0.6) = -0.51$

Calculation of Velocity Pressure

To calculate the velocity pressure (q_z), SNI 1727-2020 provides the equation as follows:

$q_z = 0.613\, K_z\, K_{zt}\, K_d\, K_e\, V^2$ (N/m^2) (Eq. 2.2.1.)

Note:

K_d = wind direction factor

K_z = coefficient of exposure of velocity pressure

K_{zt} = topography factor

K_e = ground face elevation factor

V = velocity of wind

Therefore, determination of the coefficients and factors should be done first. SNI 1727-2020 Table 26.10.1. and Table 26.6.1. provides the table that can be used as a guidance.

As a result, information below mentions the coefficients and factors that will be used.

- According to SNI 1727-2020 Article 26.10.1., K_z is calculated by interpolation. For exposure B, with average elevation of 14.625 m, K_z is taken 0.80.

Z (m)	B
12.20	0.76
15.20	0.81
14.625	0.80

- According to SNI 1727-2020 Article 26.6, Kd can be taken as 0.85.

- According to SNI 1727:2020 Article 26.8.2, Kzt is determined based on the site plan situation. Since the construction isn't conducted on the hill, Kzt can be taken as 1.
- According to SNI 1727:2020 Article 26.9, Ke can be taken as 1.
- General wind velocity that is used in the planning is 20 m/s.

By inputting values above, using Eq. 2.2.1, qz can be computed below.

qz = 0.613 (0.8) (1) (0.85) (1) (20)2 (N/m2)

qz = 166.8228 N/m2

Calculation of design wind pressure

According to SNI 1727-2020, for half-opened building, internal pressure coefficient (GC_{pi}) can be taken +0.55 and -0.55. Observing the result of the calculation of wind face and wind back, +0.253 will be taken as GC_p and -0.55 will be taken as GC_{pi}.

To calculate the design wind pressure, SNI 1727:2020 Article 27.3.1 provides the equation that can be used as follows:

$p = qGC_p – q_i(GC_{pi})$ (Eq. 2.2.2.)

Note:

q = qz for wall in wind face sides that measures in the elevation of z

G = Wind effect factor

Cp = Coefficient of external pressure

GCpi = Coefficient of internal pressure

Using Eq. 2.2.2., the value of design wind pressure is:

p = 166.8228 (0.253) – 166.8228 (-0.55)

p = 134.0379 N/m2 = 0.134028 kN/m2

The calculation of the wind pressure for the modelling will be helped by the MIDAS Gen's "Wind Load" menu, by inputting the coefficient of internal pressure for wind face and wind back.

2.2.4 Rain Load

According to PBI 1987, Rain load can be calculated as follow.

$$R = 40 - 0{,}8\alpha \text{ (kg/m}^2\text{)} \leq 20 \text{ kg/m}^2$$
$$= 13.38 \text{ kg/m}^2 \leq 20 \text{ kg/m}^2$$

2.2.5 Load Combination

Load combinations are ultimate factored load that affects the load based on assigned load. In this design, SNI 2847:2019 are used as a guidance to determine the factored load for each parameter. The combinations that are used are:

1. 1.4 DL
2. 1.2 DL + 1.6 LL+ 0.5(Lr or R)
3. 1.2 DL + 1.6 (Lr or R) + (1.0 L or 0.5 W)
4. 1.2 DL + 1.0 W + 1.0 LL + 0.5 (Lr or R)
5. 1.2 DL + 1.0 E + 1.0 LL
6. 0.9 DL + 1.0 W
7. 0.9 DL + 1.0 E

For combination 5 and 7, due to earthquake load that are ruled out on SNI 1726:2019, nominal dead load, nominal live load, and nominal earthquake load are:

1. (1.2+0.2 Sds) DL + 1 LL $\pm$ 0.3 ρ EX $\pm$ 1 ρ EY
2. (1.2+0.2 Sds) DL + 1 LL $\pm$ 1 ρ EX $\pm$ 0.3 ρ EY
3. (0.9-0.2 Sds) DL $\pm$ 0.3 ρ EX $\pm$ 1 ρ EY
4. (0.9-0.2 Sds) DL $\pm$ 1 ρ EX $\pm$ 0.3 ρ EY

The abbreviations of the load combinations are explained below:

DL : Dead load, includes Super Imposed Dead Load

LL : Live Load

Lr : Roof Live Load

R : Rain Load

W : Wind Load

EX : Earthquake load- X direction

EY : Earthquake load- Y direction

ρ : Redundant factor (equals to 1.3, according to SNI 1726:2019 Article 7.3.4)

As a conclusion, table below shows the load combination used in the modelling.

Table 2.4 Load Combination Table

Combination	**D**	**SIDL**	**L**	**R**	**W_x**	**W_y**	**Lr**	**R_x**	**R_y**
Comb1	1.4	1.4							
Comb2A	1.2	1.2	1.6				0.5		
Comb2B	1.2	1.2	1.6	0.5					
Comb3A	1.2	1.2	1				1.6		
Comb3B	1.2	1.2			0.5	0.5	1.6		
Comb3C	1.2	1.2	1	1.6					
Comb3D	1.2	1.2		1.6	0.5	0.5			
Comb4A	1.2	1.2	1		1	1	0.5		
Comb4B	1.2	1.2	1	0.5	1	1			
Comb5XA	1.356	1.356	1					1.39	0.39
Comb5XB	1.356	1.356	1					1.39	-0.39
Comb5XC	1.356	1.356	1					-1.39	0.39
Comb5XD	1.356	1.356	1					-1.39	-0.39
Comb5YA	1.356	1.356	1					0.39	1.39
Comb5YB	1.356	1.356	1					-0.39	1.39
Comb5YC	1.356	1.356	1					0.39	-1.39
Comb5YD	1.356	1.356	1					-0.39	-1.39
Comb6	0.9	0.9		1	1				
Comb7XA	0.744	0.744						1.39	0.39
Comb7XB	0.744	0.744						1.39	-0.39
Comb7XC	0.744	0.744						-1.39	0.39
Comb7XD	0.744	0.744						-1.39	-0.39
Comb7YA	0.744	0.744						0.39	1.39

Comb7YB	0.744	0.744						-0.39	1.39
Comb7YC	0.744	0.744						0.39	-1.39
Comb7YD	0.744	0.744						-0.39	-1.39

2.3 Preliminary Design and Structural Modelling

Preliminary design of the structure is computed based on SNI 2847:2019. Preliminary design will be conducted in all structural elements, including beam, column, and slab.

2.3.1 Preliminary Design of Beam Element

Beam is a structural element that will transfer loads from slab to the column. In the construction of the building, beam design will be divided into main beam, secondary beam and cantilever beam. While main beam transfers the load from secondary beam to the column, secondary beam itself will transfer the load from slab to the main beam. The preliminary design of beam will be conducted based on SNI 2847:2019 Article 9.3.1 (see Table 2.5). Therefore, for main beam, height of the beam will be calculated as L/16, while secondary beam will be calculated L/21, and cantilever as L/8.

Therefore, the dimension that is used on the building is listed in Table 2.5 below.

Table 2.5 Calculation of Preliminary Design of Beam

Type	L(mm)	Hmin	hused	bmin1	bmin2	Bused	Type
Main Beam	8000	500	600	180	250	350	B600/350
Main Beam	8000	500	700	180	250	350	B700/350
Main Beam	8000	500	550	180	250	300	B550/300
Main Beam	9000	562.5	800	180	250	400	B800/400
Secondary Beam	8000	380.9524	500	150	250	250	BA500/250
Cantilever	3250	406.25	500	150	250	250	BK500/250

2.3.2 Preliminary Design of Column Element

Column is a vertical element that receive axial loads. Column transfers the load from its own weight, live load, and super-imposed dead load from slabs, beams, and their own to the lower structure until it is transferred to the foundation. In the calculation of preliminary design of column, it is calculated by tributary area method, which all the loads per m^2 will be calculated. At the final, to determine the gross area of column, equation below is used.

$$Ag \geq \frac{Pu}{0.35\, fc'} \tag{2.3.1.}$$

While Pu will be calculated below, and f'c is the compression strength in MPa. Therefore, Table 2.6 Shows the loading system for column at the first floor.

Table 2.6 Calculation of Preliminary Design of Column

Dead Load Type	**H**	**B**	**L**	**Unit Weight**	**N**	**Floor Total**	**Weight**	
Beam 1	0.65	0,35	4,5	2400	1	3	7371	kg
Beam 2	0.65	0,35	4	2400	1	3	6552	kg
Beam 3	0.7	0,35	4	2400	1	3	7056	kg
Beam 4	0.7	0,35	4,5	2400	1	3	7938	kg
Column	0.6	0,6	4	2400	1	2	6912	kg
Upper Slab	0.12	4.63	8	2400			11100	kg
Bottom Slab	0.12	4	8	2400			9600	kg
Total Dead Load							**55701**	**kg**
SIDL Type	A	Load (kg/m²)		Floor Total			Weight	
Spacy	69	21		3			4347	kg
Ceramic	69	24		3			4968	kg
Ceiling	69	18		3			3726	kg
MEP	69	30		3			6210	kg
Total Super Imposed Dead Load							**19251**	kg
Live Load	A	Load (kg/m²)		Floor Total			Weight	
Collection room	69	7.18		3			151553.9322	kg
Total Live Load							**151553.9322**	kg
TOTAL LOAD							218612.9	kg
Ag							212394.8452	mm
Column side							460.8631524	mm
Dimension Column taken							K600x600	
							K500X500	

Therefore, the column will use two dimensions, K600X600 for first and second floor, while K500X500 will be assigned third floor.

2.3.3 Preliminary Design of Slab Element

Slab is the element that will hold a load directly based on its function. For one way slab, the preliminary provision is the same as the beam's (see Table 2.6). Therefore, in the design, slab with thickness of 125 mm will be used.

2.3.4 Structural Modelling

Structural modelling for Public Library and Co-working space is helped by MIDAS Gen's software. Figure 2.2 shows the building modelling that is done by using MIDAS Gen, applying the dimensions that assumed before by preliminary design of beam, column, and slab.

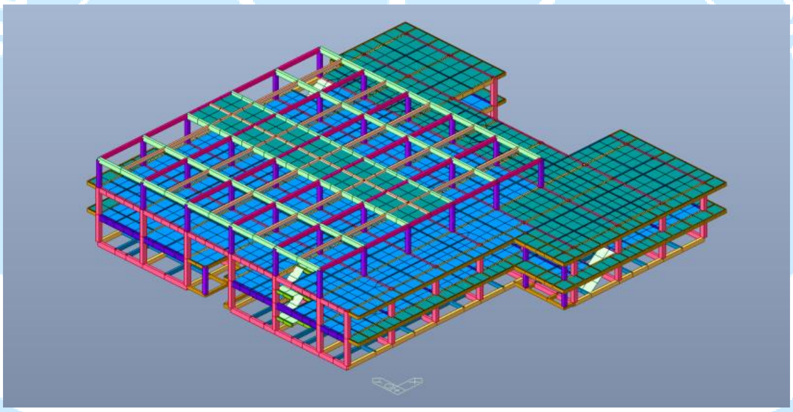


Figure 2.2 Structural Modelling on MIDAS Gen

However, it is important to notice that moment of inertia of structural component should be factored according to SNI 2847:2019 ps. 6.6.3.1.1 (see Figure 2.3). Stiffness modifier or stiffness modification factor is the allowable moment of inertia that modelled in software for elastic analysis at factored load. Figure 2.4 shows the application of the code in MIDAS Gen software.

Bagian dan kondisi		Momen inersia	Luas penampang
Kolom		0,70 I_g	1,0A_g
Dinding	Tidak retak	0,70 I_g	
	Retak	0,35 I_g	
Balok		0,35 I_g	
Pelat datar dan *slab* datar		0,25 I_g	

Figure 2.3 Allowable Moment of Inertia and Sectional-Area for Elastic Analysis for Factored Load Level

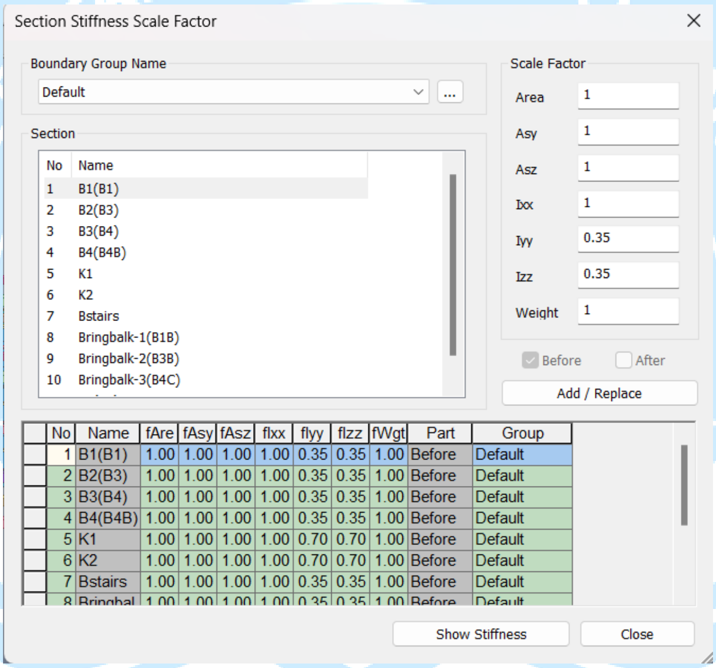


Figure 2.4 Section Stiffness Scale Factor Menu on MIDAS Gen

2.4 Lateral Restraint System Against Earthquake Loads

2.4.1 Modal Participation Ratio Control

According to the SNI 1726:2019, modal participation masses of the designed structure should utilize the number of variances so that the participation of the variance impacts to the participation ratio for 100% of structural masses. However, in the real situation, another alternative is provided by observing the participation ratio of the structure until it reaches 90% of the structural masses.

During the structural design using MIDAS Gen, the number of variances that is taken for the analysis is 30 modes, as it is shown in table 2.7 below.

Table 2.7 Mass Participation Ratio Result from MIDAS Gen

Mode No	TRAN-X		TRAN-Y		TRAN-Z		ROTN-X		ROTN-Y		ROTN-Z	
	MASS (%)	SUM (%)	MASS (%)	SUM (%)	MASS (%)	SUM (%)	MASS (%)	SUM (%)	MASS (%)	SUM (%)	MASS (%)	SUM (%)
1	64.2	64.2	2.4	2.4	0.0	0.0	2.0	2.0	39.3	39.3	5.8	5.8
2	5.7	69.9	54.0	56.4	0.0	0.0	36.3	38.3	2.7	42.0	12.0	17.8
3	2.2	72.0	15.4	71.8	0.0	0.0	3.6	41.8	0.3	42.2	55.3	73.1
4	0.0	72.0	5.6	77.3	0.0	0.0	1.8	43.6	0.0	42.2	0.1	73.2
5	0.1	72.1	1.0	78.3	0.0	0.0	0.7	44.4	0.0	42.3	2.0	75.2
6	11.1	83.2	0.1	78.4	0.0	0.0	0.1	44.5	15.2	57.5	0.0	75.2
7	0.0	83.2	0.1	78.5	0.0	0.0	0.2	44.7	0.1	57.5	0.3	75.5
8	0.1	83.4	0.5	79.0	0.0	0.0	1.1	45.8	0.5	58.0	6.6	82.1
9	0.0	83.4	4.2	83.3	0.0	0.0	9.8	55.5	0.0	58.0	0.5	82.6
10	0.0	83.4	1.3	84.6	0.0	0.0	4.3	59.8	0.0	58.0	0.4	83.1
11	0.0	83.4	0.4	85.0	0.0	0.0	1.1	60.9	0.0	58.0	0.4	83.5
12	1.0	84.4	0.0	85.0	0.0	0.0	0.0	60.9	2.8	60.8	0.9	84.4
13	0.3	84.6	0.0	85.0	0.0	0.0	0.0	60.9	0.6	61.4	0.3	84.6
14	0.1	84.8	0.0	85.0	0.0	0.0	0.1	61.0	0.5	61.9	0.1	84.7
15	1.2	85.9	0.0	85.0	0.0	0.0	0.0	61.0	3.3	65.2	0.4	85.1
16	0.0	85.9	0.3	85.3	0.0	0.0	0.9	62.0	0.0	65.2	0.1	85.2
17	1.0	87.0	0.0	85.4	0.0	0.0	0.1	62.0	2.6	67.8	0.0	85.2
18	0.0	87.0	0.4	85.7	0.0	0.0	1.0	63.0	0.0	67.8	0.0	85.3
19	0.1	87.1	0.7	86.4	0.0	0.0	1.4	64.4	0.1	67.9	0.0	85.3
20	4.8	91.9	1.0	87.4	0.0	0.0	2.4	66.8	9.6	77.5	1.1	86.3
21	1.0	92.9	3.7	91.1	0.0	0.0	8.7	75.5	2.0	79.5	1.0	87.3
22	0.1	93.0	0.6	91.7	0.0	0.0	1.6	77.1	0.3	79.8	1.0	88.2
23	0.7	93.7	0.0	91.7	0.0	0.0	0.0	77.1	2.1	81.9	0.1	88.3
24	0.0	93.7	0.3	92.0	0.0	0.0	0.9	78.0	0.0	82.0	0.1	88.4
25	0.0	93.8	0.4	92.4	0.0	0.0	0.6	78.6	0.1	82.1	0.7	89.1
26	0.0	93.8	0.7	93.1	0.0	0.0	2.0	80.6	0.0	82.1	0.6	89.7
27	0.0	93.8	0.4	93.5	0.0	0.0	0.8	81.3	0.2	82.2	4.2	93.9
28	0.0	93.8	0.0	93.5	0.0	0.0	0.0	81.3	0.0	82.2	0.0	93.9
29	0.0	93.8	0.2	93.7	0.0	0.0	0.6	81.9	0.0	82.3	0.0	93.9
30	0.3	94.1	0.0	93.7	0.0	0.0	0.0	81.9	0.8	83.0	0.1	94.1

There are several observations that should be conducted to check the behavior of the structure. From the Table 2.7, it is known that mass participation ratio fulfilled the requirements that stated in the paragraph before (94.1%). The behavior of the mode should be check by observing the motion of the structure. The first mode shows the Y-dir. translation, while second mode shows X-dir. translation and third mode shows the rotational translation (see Figure 2.5, Figure 2.6, and Figure 2.7).

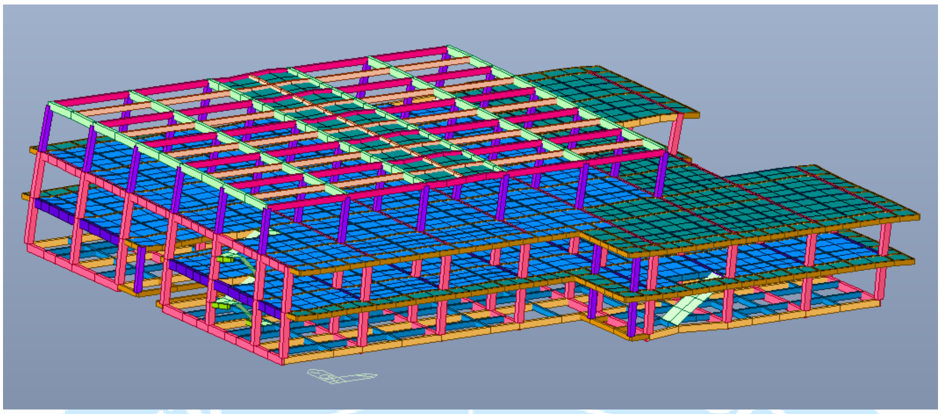


Figure 2.5 Mode Shape 1 (X-Dir)

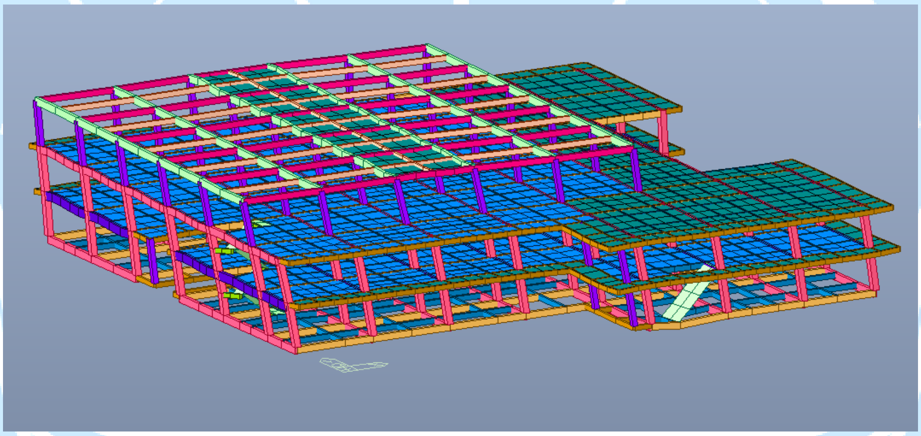


Figure 2.6 Mode Shape 2 (Y-Dir)

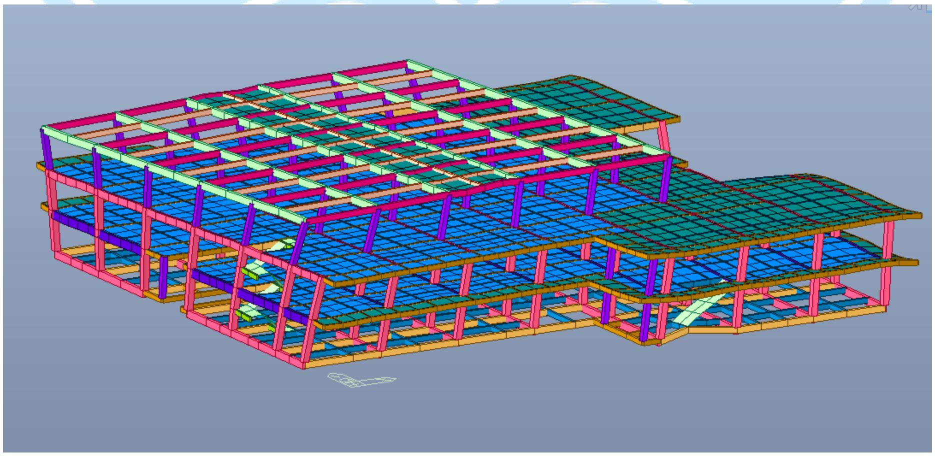


Figure 2.7 Mode Shape 3 (Rotation)

2.4.2 Period of the Structure

SNI 1726-2019 regulates the structural period that will be used to determine the periods of the structure. In the following calculation, it will be obtained two types of periods, which is the maximum period (T_{max}) and Fundamental Approach Period (T_a). For further explanation, data below will be used as calculation for period of X-dir. and Y-dir.

Design acceleration for 1 sec (SD1) = 0.6060 g

Period Boundary Coefficient

Cu (SNI 1727-2019 Table-17) = 1.4

Ct (SNI 1727-2019 Table-18) = 0.0466

x (SNI 1727-2019 Table-18) = 0.9

Height of the building = 17.25 m

From MIDAS Gen calculation, data below will be obtained based on the structural behavior at each mode.

$T_{c,x}$ = 0.595 s

$T_{c,y}$ = 0.559 s

Therefore, the calculation of maximum period (T_{max}) and Fundamental Approach Period (T_a) can be conducted as follow.

Fundamental Approach Period (T_a) $= C_t \times h^x$

$= 0.0466 \times 17.25^{0.9}$

$= 0.6046$ s

Maximum period (T_{max}) $= C_u \times T_a$

$= 1.4 \times 0.6046$ s

$= 0.846$ s

Since both of the period resulted in MIDAS Gen is smaller than T_{max}, $T_{c,x}$ and $T_{c,y}$ will be directly used for the next calculation.

2.4.3 Seismic Base Shear

Before determining the seismic base shear, the seismic response coefficient that are regulated in SNI 1726-2019 Article 7.8.1.1. Therefore, by using the guidance, the calculation can be conducted as follow.

$$C_s = \frac{S_{DS}}{(\frac{R}{I_e})} = \frac{0.7802}{(\frac{8}{1.5})} = 0.146$$

There are also upper and bottom limit for the seismic response coefficient, both for X-dir. and Y-dir for maximum coefficient. Later, the seismic response coefficient that is used will be determined by provisions. Calculation of the upper and lower coefficient limit are shown below.

$$\mathrm{C_{s,max}} = \frac{\mathrm{S_{D1}}}{(\mathrm{T} \times \frac{\mathrm{R}}{\mathrm{I_e}})}$$

$$\mathrm{C_{s,max,X}} = \frac{0.6060}{(0.595 \times \frac{8}{1.5})} = 0.1908$$

$$\mathrm{C_{s,max,Y}} = \frac{0.6060}{(0.559 \times \frac{8}{1.5})} = 0.2034$$

$$\mathrm{C_{s,min}} = 0.044 \times \mathrm{S_{Ds}} \times \mathrm{I_e}$$

$\mathrm{C_{s,min}} = 0.044 \times 0.7802 \times 1.5 = 0.0515 > 0.01$ (OK)

Since Cs is bigger than Cs,min and smaller than Cs,max, then Cs will be used, both Cs,used,X and Cs,used,Y.

Seismic weight of the structure is also needed to be determined. MIDAS Gen Analysis provided the mass of the structure and can be seen at the Table 2.8.

Table 2.8 Structure Mass

Story	Mass (kN/g)
Roof	727.7635
Story2	2293.0292
Story1	2442.3483
Total	5463.141
Effective Weight	**5463.141× 9.80665 = 53,575 kN**

To calculate the seismic base shear, SNI 1726-2019 provided a provision that can be seen in the calculation below.

$$V = C_s \times W = 0.1463 \times 53{,}575 \text{ kN} = 7837.51 \text{ kN}$$

V will be checked with the static shear force that is inputted into MIDAS Gen's analysis. According to the result of the reaction for static force, it is obtained $V_x = V_y = 7019.65$ kN. Therefore, it is showed that the calculation is nearly the same as the building modelling input.

2.4.4 Scale Factor of Forces

Scale Factor for earthquake loads is depended on the result of structural analysis result. However, initial scale factor can be computed as follow.

$$SF = \frac{G}{\left(\frac{R}{I}\right)} = \frac{9.80665}{\left(\frac{8}{1.50}\right)} = 1.839$$

Based on MIDAS Gen calculation, result of support due to dynamic earthquake load that effected by response spectrum, inputting 1.839 as a scale factor, are shown as follow.

Vi,X = 999.538 kN

Vi,Y = 1011.27 kN

Therefore, the reduction of the initial scale factor can be done by computation as follow.

$$fx = \frac{7019.65}{999.538} = 7.023$$

$$fy = \frac{7019.65}{1011.27} = 6.941$$

It is need to be noticed that value of g = 9.80 m^2/s is already computed automatically in MIDAS Gen's menu for scale factor. Therefore, final scale factor can be computed as follow (see Figure 2.8)

$$SFx = \frac{7.023 \times 0.625}{9.80665} = 1.32 \text{ m/s}^2$$

$$SFy = \frac{6.941 \times 0.625}{9.80665} = 1.31 \text{ m/s}^2$$

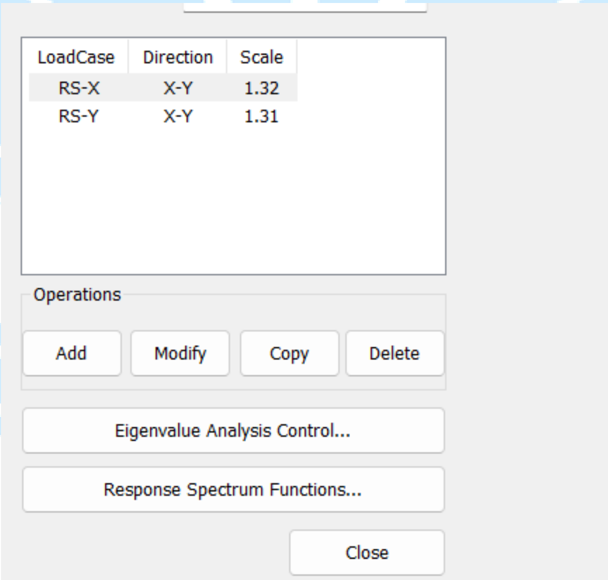


Figure 2.8 Scale Factor for Response Spectrum

2.4.5 Story Drift

Story drift can be computed from the biggest differences of deflection of all points above and below the story. The determination of displacement uses the center of mass method, then calculate the inelastic drift and check with drift limit by equation below

$$\Delta = \frac{\delta \times Cd}{Ie}$$

$$\Delta max = \frac{\Delta \times h}{\rho}$$

H is the length of column, ρ is redundant factor. Δ is the allowable story drift that taken from SNI 1726:2019 Table 20, taken as 0.001, then Table 2.9 shows the result of the calculation and Figure 2.9 shows the graph of the story drift.

Table 2.9 The Story Drift Calculation

Story	***Displacement***		***Elastic Drift***		***h***	***Inelastic Drift***		***Drift Limit***	***Cek***
	δe_X	δe_Y	δe_X	δe_Y		Δ_X	Δ_Y		
	(mm)	**(mm)**	**(mm)**	**(mm)**	**(mm)**	**(mm)**	**(mm)**	**(mm)**	
3	14.809	11.373	6.627	4.882	4000	24.300	17.900	30.769	OK
2	8.182	6.491	4.064	3.164	4000	14.900	11.600	30.769	OK
1	4.118	3.327	4.118	3.327	4000	15.100	12.200	30.769	OK

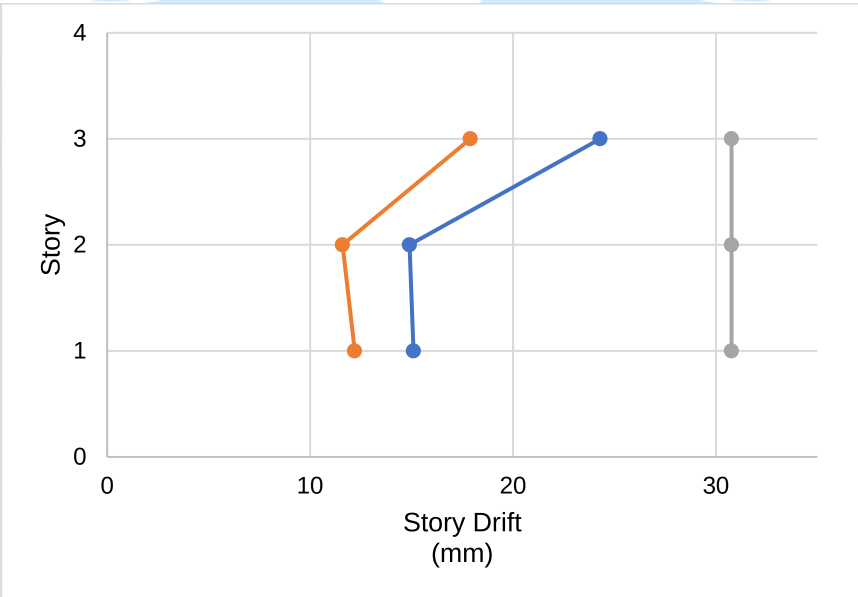


Figure 2.9 Story Drift Graph

2.4.6 P-Delta Effect

According to SNI 1726-2019 ps.7.8.7, P-Delta effect at shear and moment of story level, structure output (includes story force, element's moment, and story drift) it doesn't need to be calculated if the stability coefficient is less than 0.1. Stability coefficient is calculated as follow.

$$\theta = \frac{\mathrm{Px}\Delta I_e}{V_x h_{sx} C_d}$$

P_x is the service load at the story level, which load factor doesn't need to be exceeded 1, Δ is the modified story drift (inelastic drift), I_e is the importance of earthquake, V_x is the seismic shear force, h_{sx} is the story height, and C_d is deflection enlarge factor.

Table 2.10 shows the result of the P-Delta analysis control, and Figure 2.10 shows the P-Delta graph. Both of them shows that P-Delta effect doesn't need to be included on the analysis.

Table 2.10 The P-Delta Analysis Control

Story	*Inelastic Drift*		*Story Forces*			*h*	*Stability Coefficent*		*Limit of P-Delta Effect*	*Limit of Structure Stability*	*Cek*
	Δ_X	*Δ_Y*	*P*	*V_x*	*V_y*						
	(mm)	(mm)	(kN)	(kN)	(kN)	(mm)	*θX*	*θY*			
3	24,300	17,900	7867,3	2399,31	2176,34	4000	0,0054	0,0044	0,1	0,0909	OK
2	14,900	11,600	46237,93	6157,49	5620,58	4000	0,0076	0,0065	0,1	0,0909	OK
1	15,100	12,200	84994,60	8326,30	7645,08	4000	0,0105	0,0092	0,1	0,0909	OK

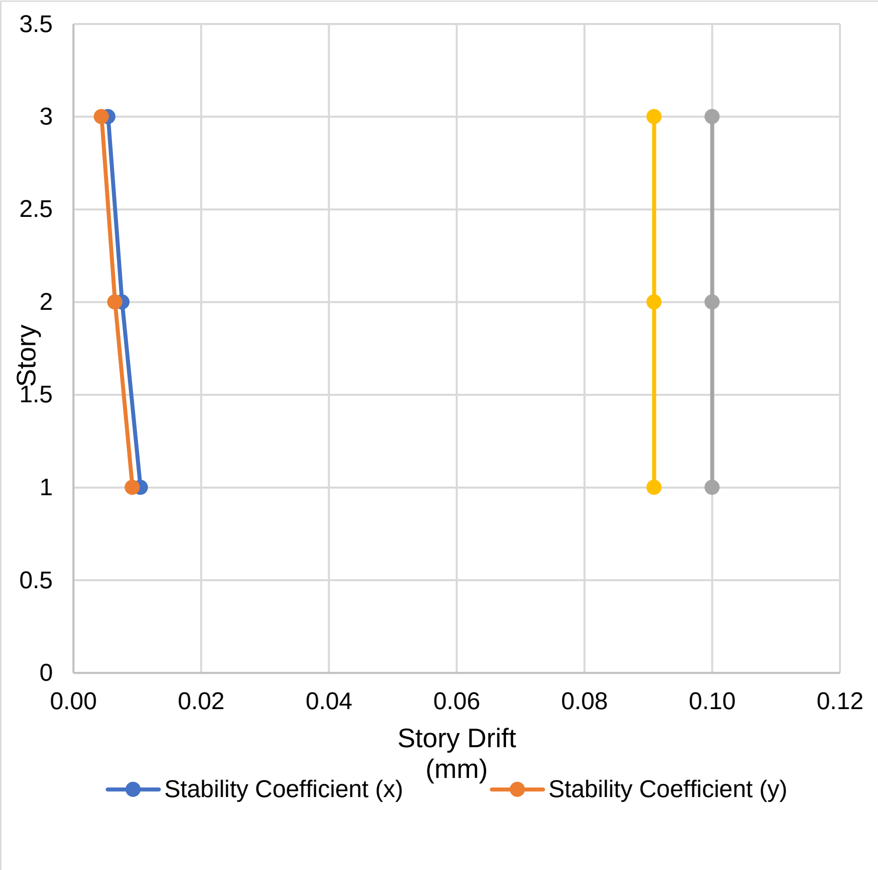


Figure 2.10 P-Delta Analysis

2.4.7 Control to the Irregularity of the Building

According to the SNI 1726:2019, structure should be classified as regular or irregular structure based on the criteria that is ruled in ps. 7.3.2. Irregularity of building will be divided by horizontal and vertical irregularity. If the requirements below are not fulfilled, then consequences of irregularity should be mentioned during structural modelling (2nd running on the MIDAS Gen).

1. Horizontal Irregularity

   <u>Type IA</u>

   Torsion irregularity is defined to exist if the maximum inter-story deviation, calculated including unexpected torsion of 1.0, at one end of the structure transverse to an axis is more than 1.2 times the average inter-story deviation at both ends of the structure. The torsional irregularity requirements in the reference articles apply only to structures where the diaphragm is rigid or semi-rigid.

Type IB

Excessive torsional irregularity is defined to exist if the calculated maximum inter-story deviation including the result of unexpected torsion with A_x =1.0 at one end of the structure transverse to an axis is more than 1.4 times the average inter-story deviation at both ends of the structure. The requirement for excessive torsional irregularity in the reference articles applies only to structures where the diaphragm is rigid or semi-rigid.

Figure 2.11 shows the illustration of horizontal irregularity type IA and IB, while Table 2.11 shows the result of the investigation.

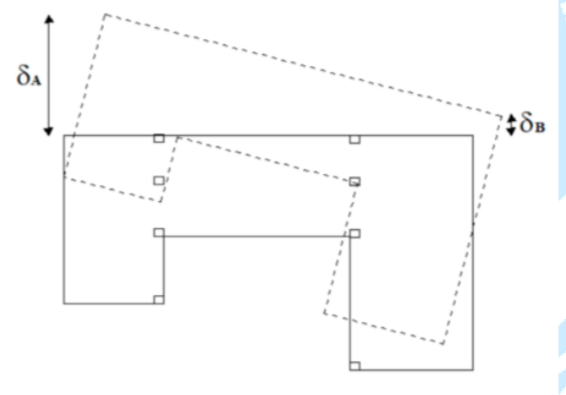


Figure 2. 11 Horizontal Irregularity Type IA and IB

Table 2.11 Horizontal Irregularity Type IA and IB

Story	Dir.X		Dir.Y	
	$\Delta_{max}/\Delta_{avg}$	Check	$\Delta_{max}/\Delta_{avg}$	Check
3	1.0459	OK	1.228	IA
2	1.1268	OK	1.377	IA
1	1.1951	OK	1.3084	IA

It is inspected that the horizontal irregularity type IA is occurred to the structure. Table 2.17 later will show the recapitulation of consequences that should be done on the modelling.

<u>Type II</u>

An internal corner irregularity is defined as existing if both dimensions of the structural plan projection from the internal corner location are greater than 15% of the structural plan dimensions in the direction considered.

Figure 2.12 shows the illustration of horizontal irregularity type II.

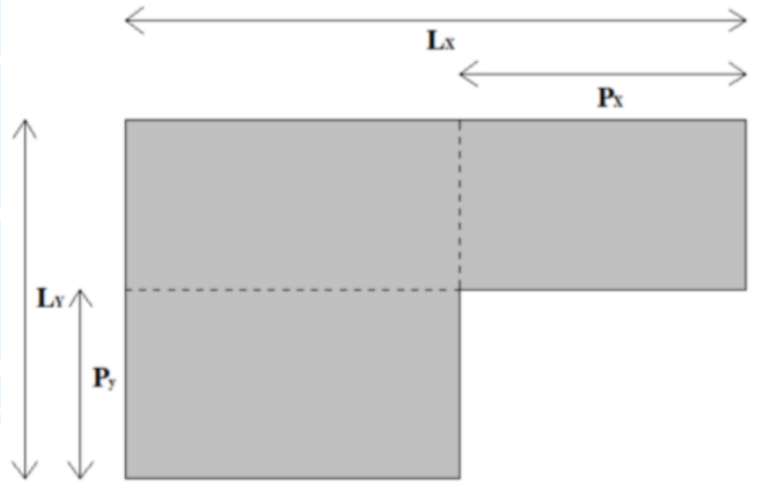


Figure 2.12 Horizontal Irregularity Type II

From the structural floor plan, it is obtained that:

Lx = 55.5 m

Px = 43.5 m

Ly = 19.25 m

Py = 3.25 m

Lx/Px = 0.3468 m

Ly/Py = 0.0747 m

Conclusion = Type II occurred (Lx/Px>0.15)

It is inspected that the horizontal irregularity type II is occurred to the structure. Table 2.17 later will show the recapitulation of consequences that should be done on the modelling.

<u>Type III</u>

A diaphragm discontinuity irregularity is defined as existing if there is a diaphragm that has a discontinuity or sudden variation in stiffness, including having a cut or open area greater than 50% of the gross closed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one level to the next.

Figure 2.13 shows the illustration of horizontal irregularity type III, and Table 2.12 shows the result of the inspection.

Figure 2.13 Horizontal Irregularity Type III

Table 2.12 Horizontal Irregularity Type III

A_{total}	:	2640	m^2
$A_{opening}$	:	33.6	m^2
Cek		OK	

From the result, horizontal irregularity type III is not inspected occurred.

<u>Type IV</u>

An irregularity due to displacement perpendicular to the plane is defined to exist if there is a discontinuity in the path of lateral force resistance, such as a displacement perpendicular to the plane of at least one vertical element resisting the lateral force. In the structure, there is found an- offset column so that both of those centroids don’t meet at one point, that creates the shear force perpendicular to the plane area of the building. Figure 2.14 shows the illustration of the horizontal irregularity, while Table 2.17 later will show the recapitulation of consequences that should be done on the modelling.

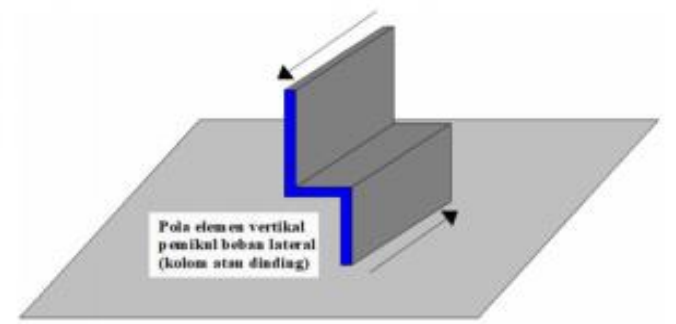


Figure 2.14 Horizontal Irregularity Type IV

Type V

A nonparallel system irregularity is defined to exist if the vertical elements carrying lateral forces are not parallel to the main orthogonal axes of the seismic force system. From the structure, it is not found the nonparallel irregularity (see Figure 2.15 for the illustration).

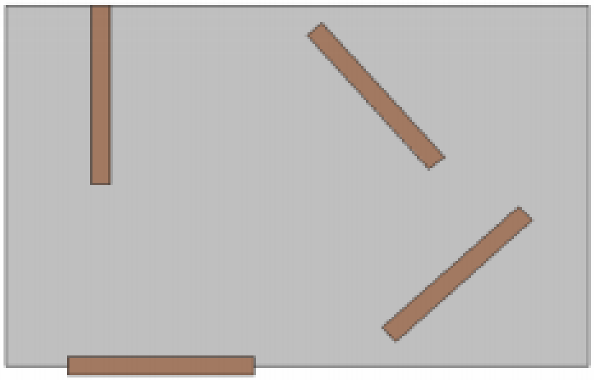


Figure 2.15 Horizontal Irregularity Type V

2. Vertical Irregularity

Type IA

Soft Story Stiffness Irregularities are defined as existing if there is a story whose lateral stiffness is less than 70% of the lateral stiffness of the story above it or less than 80% of the average stiffness of the three stories above it.

Type IB

Excessive Soft Story Stiffness Irregularities are defined as existing if there is a story whose lateral stiffness is less than 60% of the lateral stiffness of the story above it or less than 70% of the average stiffness of the three stories above it.

Stiffness of the structure can be obtained from MIDAS Gen. Figure 2.16 shows the illustration of vertical irregularity type IA and IB, while Table 2.13 shows the result of the investigation.

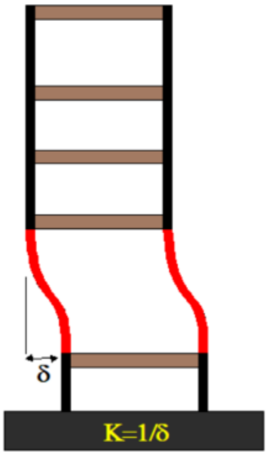


Figure 2.16 Vertical Irregularity Type IA and IB

Table 2.13 Vertical Irregularity Type IA and IB

Story	Dir X		Dir Y	
	Stiffness kN/m	Check	Stiffness kN/m	Check
3	603.41	not considered	819.04	not considered

2	986.11	OK	1264.83	OK
1	971.73	OK	1200.72	OK

<u>Type II</u>

Mass Irregularities are defined to exist if the effective mass on any level is more than 150% of the effective mass of the adjacent level. A roof that is lighter than the floor below does not need to be reviewed.

Mass of each floor can be obtained from MIDAS Gen. Figure 2.17 shows the illustration of vertical irregularity type II, while Table 2.14 shows the result of the investigation.

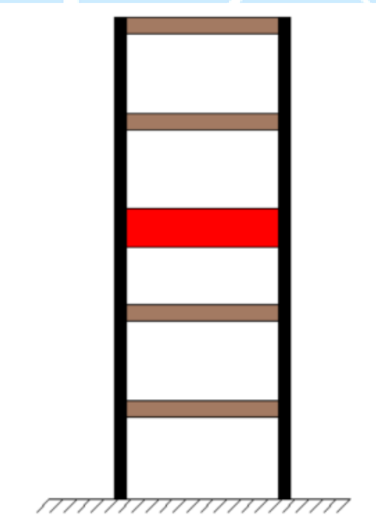


Figure 2.17 Vertical Irregularity Type II

Table 2.14 Vertical Irregularity Type II

Story	**Mass**	**Check**
	kg	
3	22485.444	OK
2	23949.667	V.2
1	8729.559	OK

From the inspection result, it is known that vertical irregularity occurred. Therefore, the consequences need to be mention as Table 2.17 shows later.

Type III

Vertical Geometric Irregularities are defined to exist if the horizontal dimension of the seismic force-bearing system at any level is more than 130% of the horizontal dimension of the seismic force-bearing system at the adjacent level.

Figure 2.18 shows the illustration of vertical irregularity type III, while Table 2.15 shows the result of the investigation.

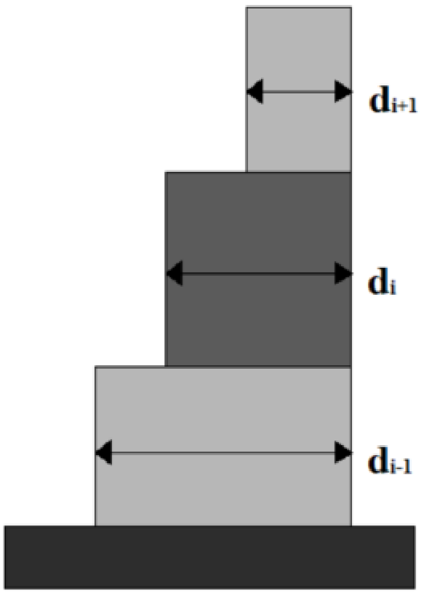


Figure 2.18 Vertical Irregularity Type III

Table 2.15 Vertical Irregularity Type III

Story	L	Check
	mm	
RT	4000	
3	4000	OK
2	4000	OK
1	4000	OK

Type IV

Irregularity Due to Plane Discontinuity on Vertical Elements Bearing Lateral Forces is defined as existing if the shift in the direction of the field of the lateral force bearing elements is greater than the length of the element or there is a reduction in the stiffness of the bearing elements in the floor below. From the observation of the structure's shape, vertical irregularity is not found in the structure (see Figure 2.19 for the illustration).

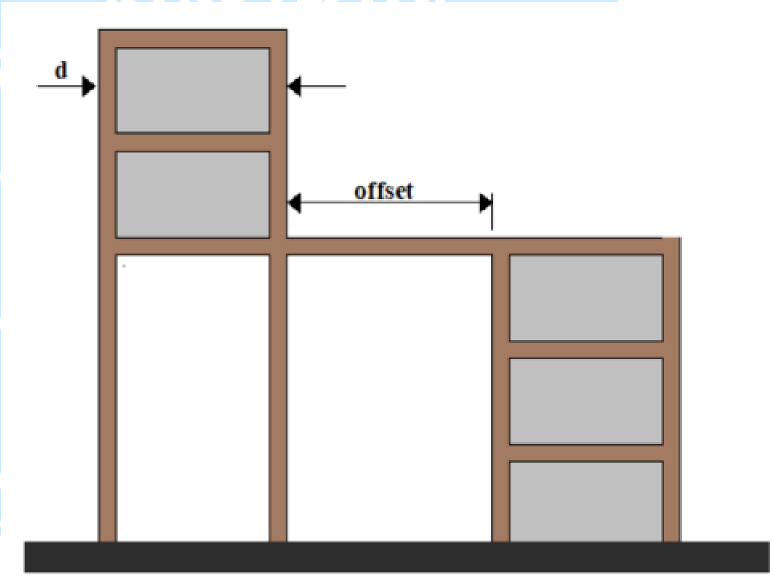


Figure 2.19 Vertical Irregularity Type IV

Type VA

Weak Story Irregularities Due to Discontinuities in the Lateral Strength of a Story are defined to exist if the lateral strength of a story is less than 80% of the lateral strength of the story above it. The lateral strength of a story is the total strength of all seismic resisting elements that share a story shear in the direction under consideration.

Type VB

Excessive Weak Story Irregularities Due to Discontinuities in Lateral Strength A story is defined to exist if the lateral strength of a story is less than 65% of the lateral strength of the story above it. The lateral strength of a story is the total strength of all seismic resisting elements that share a story shear in the direction under consideration.

Vertical irregularity type V can be checked by finding the story forces on each floor (by using MIDAS Gen). Figure 2.20 shows the illustration of vertical irregularity type III, while Table 2.16 shows the result of the investigation.

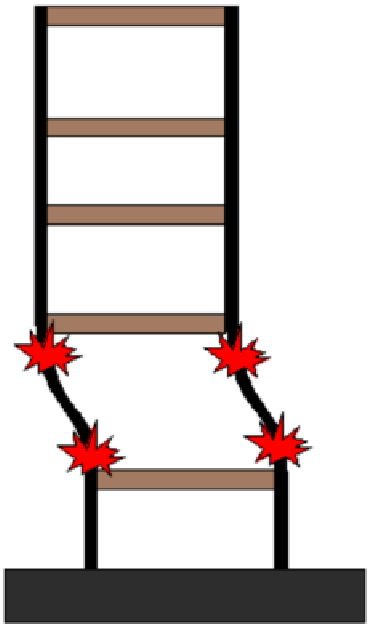


Figure 2.20 Vertical Irregularity Type VA and VB

Table 2.16 Vertical Irregularity Type VA and VB

Story	Dir.X		Dir.Y	
	Forces kN	Check	Forces kN	Check
3	2407.6056		2245.2476	
2	6197.4951	OK	5964.6405	OK
1	8380.1744	OK	8126.1811	OK

3. Consequences of Irregularity

From the result of the inspection, it is found that horizontal irregularity type IA, II, III, IV, and vertical irregularity type IV occurred on the structure. Therefore, consequences of the irregularity should be mention (see Table 2.17).

Table 2.17 Irregularity Consequences

Type	Reference of consequences (see SNI 1726:2019)						
HIA	11.3.4	7.12.1		7.3.3.4	7.7.3	7.8.4.3	Table 16
H2				7.3.3.4			Table 16
H3				7.3.3.4			Table 16
H4	11.3.4		7.3.3.3	7.3.3.4	7.7.3		Table 16
V2							Table 16

Shortly, main idea of the reference of consequences can be seen in points below.

a. SNI 1726:2019 ps. 11.3.4 mentions about the accidental torsion of 5%.

b. SNI 1726:2019 ps. 7.12.1 mentions about the redundancy factor (1.3) that should be pointed out in story drift investigation.

c. SNI 1726:2019 ps. 7.3.3.3 mentions about strengthen factor (Ω_0) in load combination.

d. SNI 1726:2019 ps. 7.3.3.4 mentions that in diaphragm design, the forces should be enlarged by 25%, especially in connector of diaphragm or other elements.

e. SNI 1726:2019 ps. 7.3.3 mentions that the analysis should be done by three dimensions analysis (investigations of three degree of freedoms).

f. SNI 1726:2019 ps. 7.8.4.3 mentions that torsional moment should be enlarged by factor (A_x) between 1.0 and 3.0.

g. SNI 1726:2019 Table 16 briefly mentions that for other structural type on seismic design category D, static analysis equivalent is not allowable.

4. Building Dilatation

Building dilatation is the separation of structural or connector that creates the irregularity shape of a building. It is done to decrease the center of mass into two separated building, and mass building will be smaller continued by smaller effect from earthquake.

Building dilatation is usually done by the layout of building in the shape of H, T, C, X, or U. It will decrease the potential crack due to lateral forces of earthquake, since other element of a structure may collide to each other.

Building dilatation method that is used in the structure is column dilatation. Dilatation should mention the displacement or story drift of a building (Amalia et al., 2022). From those data, size of a dilatation can be obtained. However, connection on the void due to dilatation should be concerned. Usually, it is used elastomer Polychloroprene rubber to fill the void so that it can decrease the effect due to collision.

From floor displacement, the largest displacement is obtained by 17.7 cm. Therefore, the building will be separated as at least 2 times of the displacement, which in the design of structure, it is taken 35.5 mm (see Figure 2.21). It will use the help of MIDAS Gen to do the second analysis (2^{nd} running) to determine the internal forces for design.

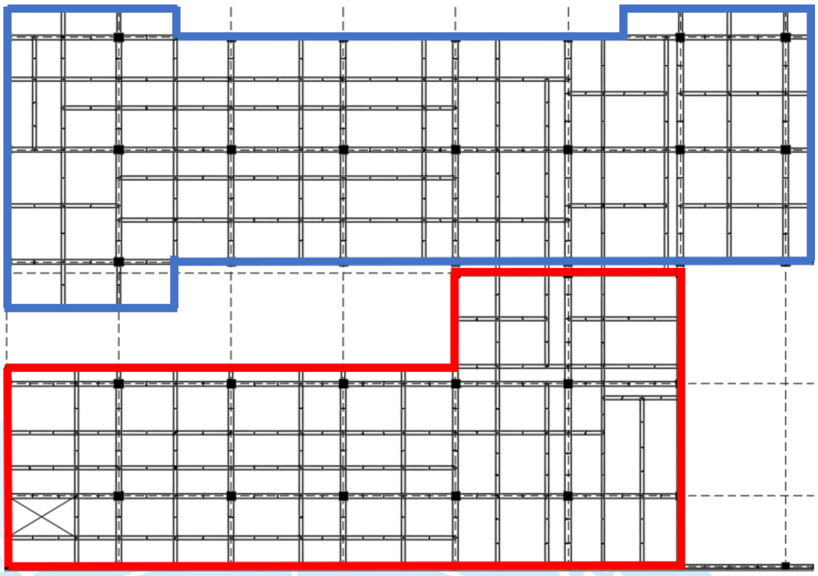


Figure 2. 21 Building Dilatation

2.5 The Design of Roof

2.5.1 Calculation of Purlin

<u>Planning Data</u>

Profile C 150X65X20X2.5 is used as a purlin.

t	= 2.5 mm
A	= 7.59 cm2
Weight Unit	= 5.96 kg/m
Ix	= 267 cm4
Iy	= 44 cm4
Zx	= 35.6 cm3
Zy	= 10 cm3
rx	= 5.93 cm
ry	= 2.41 cm
Cy	= 2.12 cm
Xo	= 5.15 cm
J	= 1581 cm4
Cw	= 2148 cm6
Material data	
Material of ceiling	= 5.556 kg/m2

Material of lambersering = 9.9 kg/m2

Calculation of Load

Dead Load:

Weight of the roof closer = 13.38488 kg/m

Weight of Ceiling = 29.57917 kg/m

Weight of Purlin = 5.96 kg/m

Qd = 13.3848788507778 + 29.5791660734376 + 5.96 =

Qd = 48.92404 kg/m

Resultant of the load calculation:

Qd,x = 48.92404 kg/m × cos(33.27) = 40.902 kg/m

Qd,y = 48.92404 kg/m × sin (33.27) = 26.843 kg/m

Live Load:

LL for purlin design is taken 100 kg or 0.96 kN.

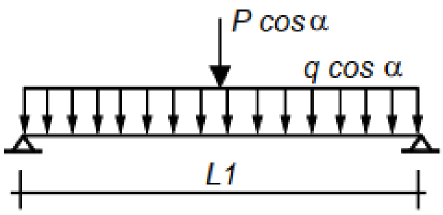
Rain Load:

R = 40 – 0,8α (kg/m^2) ≤ 20 kg/m^2

= 13.38 kg/m^2 ≤ 20 kg/m^2

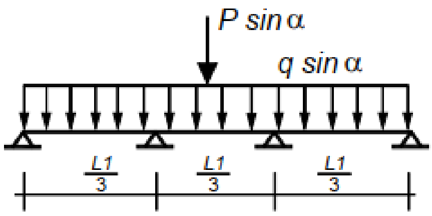
Calculation of Moment

By inputting distance between truss, L= 4000 m, moment for X-dir. and Y-dir. can be obtained in the Table 2.18 below.



$$M_{3,D} = \frac{1}{8}\, q \cos\alpha\, L^2$$

$$M_{3,L} = \frac{1}{4}\, P \cos\alpha\, L$$



$$M_{Y,D} = \frac{1}{8}\, q \sin\alpha\, (\frac{L}{3})^2$$

$$M_{Y,L} = \frac{1}{4}\, P \sin\alpha\, (\frac{L}{3})$$

Table 2.18 Load Calculation for Purlin

Moment	**X-dir**	**Y-dir**	**Type of Load**
DD	44.88321937	4.987024	Uniform Load
R	43.01466501	4.779407	Uniform Load
LL	4.586113494	23.2992	Point Load
W	0.228778157	0.348614	Uniform Load

(in kgM)

Load combination that controls the calculation of purlin is:

1. 1.4 D
2. 1.2 D + 1.6L + 0.5(L or R)
3. 1.2 D + 1.6(L or R) + L or 0.5W
4. 1.2 D + 1 W + 0.5(L or R)
5. 0.9 D + 1 W

Then, the moment calculation can be taken in the Table 2.19 and Table 2.20 below.

Table 2.19 Moment Calculation of X direction

No. Comb	D	R	LL	W	Result
1	**1.4**	**0**	**0**	**0**	62.83651 kgm
	62.83650712	0	0	0	
2a	**1.2**	**0**	**1.6**	**0**	61.19764 kgm
	53.85986324	0	7.337782	0	
2b	**1.2**	**0.5**	**1.6**	**0**	82.70498 kgm
	53.85986324	21.50733	7.337782	0	

3a	**1.2**	**0**	**1.6**	**0.5**	61.31203 kgm
	53.85986324	0	7.337782	0.114389	
No. Comb	D	R	LL	W	Result
3b	**1.2**	**1.6**	**1**	**0**	127.2694 kgm
	53.85986324	68.82346	4.586113	0	
3c	**1.2**	**1.6**	**0**	**0.5**	122.7977 kgm
	53.85986324	68.82346	0	0.114389	
4a	**1.2**	**0**	**0.5**	**1**	56.3817 kgm
	53.85986324	0	2.293057	0.228778	
4b	**1.2**	**0.5**	**0**	**1**	75.59597 kgm
	53.85986324	21.50733	0	0.228778	
5	**0.9**	**0**	**0**	**1**	40.62368 kgm
	40.39489743	0	0	0.228778	
					690.7197 kgm

Table 2.20 Moment Calculation of Y direction

No. Comb	D	R	LL	W	Result
1	**1.4**	**0**	**0**	**0**	6.981834 kgm
	6.981834124	0	0	0	
2a	**1.2**	**0**	**1.6**	**0**	43.26316 kgm
	5.984429249	0	37.27873	0	
2b	**1.2**	**0.5**	**1.6**	**0**	45.65286 kgm
	5.984429249	2.389704	37.27873	0	
3a	**1.2**	**0**	**1.6**	**0.5**	43.43746 kgm
	5.984429249	0	37.27873	0.174307	
3b	**1.2**	**1.6**	**1**	**0**	36.93068 kgm
	5.984429249	7.647052	23.2992	0	
3c	**1.2**	**1.6**	**0**	**0.5**	13.80579 kgm
	5.984429249	7.647052	0	0.174307	
4a	**1.2**	**0**	**0.5**	**1**	17.98265 kgm
	5.984429249	0	11.6496	0.348614	
4b	**1.2**	**0.5**	**0**	**1**	8.722747 kgm
	5.984429249	2.389704	0	0.348614	
5	**0.9**	**0**	**0**	**1**	4.836936 kgm
	4.488321937	0	0	0.348614	
					221.6141 kgm

Moment of X and Y shoud be checked with equation below.

$$\frac{Mu,x}{\emptyset b \times Mnx} + \frac{Mu,u}{\emptyset b \times \frac{Mny}{2}} \leq 1 \qquad \text{(Eq. 2.1)}$$

Using Eq 2.1, it will be obtained a calculation below.

$$\frac{127.2694}{0.9\times 690.7197}+\frac{45.65286}{0.9\times\frac{221.6141}{2}}=0.66\leq 1\ \text{(Safe)}$$

Material Control (Stress on the profile C)

To check the stress of profile, equation below is used.

$$f_b=\frac{M_{3,U}}{\emptyset W_3}+\frac{M_{2,U}}{\emptyset W_w}\leq F_y\ ;\emptyset=0{,}9 \qquad \text{(Eq. 2.2)}$$

Using Eq 2.2, it is obtained as follow.

$$f_b=\frac{127.2694\times 0.0098}{0.9\times 35.6\times 1000}+\frac{45.65286\times 0.0098}{0.9\times 10\times 1000}\leq 240$$

$$f_b=49.745\ \text{MPa}\leq 240\ \text{MPa}$$

Therefore, the profile is safe in stress condition.

Material Control (Deflection of profile C)

To check the deflection of profile, equation below is used.

$$\delta_2=\frac{5}{384}\times\frac{q\cos\alpha\, L^4}{EI}+\frac{1}{48}\times\frac{P\cos\alpha\, L^3}{EI} \qquad \text{(Eq. 2.3)}$$

$$\delta_3=\frac{5}{384}\times\frac{q\sin\alpha}{EI}\times\left(\frac{L}{3}\right)^4+\frac{1}{48}\times\frac{P\sin\alpha}{EI}\times\left(\frac{L}{3}\right)^3 \qquad \text{(Eq. 2.4)}$$

$$\delta=\sqrt{{\delta_3}^2+{\delta_2}^2}\leq\frac{1}{240}L \qquad \text{(Eq. 2.5)}$$

Using the Eq 2.3- Eq 2.5, it is determined calculation below.

$$\delta_2=\frac{5}{384}\times\frac{40.903\times 0.0098\times\ \cos(33.27)\ (4000)^4}{200000\times 267\times 10000}+\frac{1}{48}\times$$
$$\frac{83.605\ \cos\ (33.27)\ 4000^3}{200000\times 267\times 10000}=2.51$$

$$\delta_3=\frac{5}{384}\times\frac{26.843\times 0.0098\times\ \sin(33.27)}{200000\times 44\times 10000}\times\left(\frac{4000}{3}\right)^4+\frac{1}{48}$$
$$\times\frac{54.866\ \cos\ (33.27)}{EI}\times\left(\frac{4000}{3}\right)^3=0.123$$

$$\delta = \sqrt{(2.51)^2 + (0.123)^2} = 2.501 \ \leq \frac{1}{240}L = 16.67$$

Therefore, the profile is safe in deflection.

2.5.2 Calculation of Sag rod

In order to calculate the sag rod, it is calculated based on the forces that occurred. It is also factored with load combination below.

1. 1.4 $F_{T,D}$
2. 1.2 $F_{T,D}$ + 1.6 $F_{T,L}$

$F_{T,D}$ and $F_{T,L}$ are calculated using equation below.

$$F_{t,D} = n(\frac{L}{3} \times q \times sin\,\alpha) \qquad \text{(Eq. 2.6)}$$

$$F_{t,L} = \frac{n}{2} \times P \times sin\,\alpha \qquad \text{(Eq. 2.7)}$$

By taking the number of purlin (n) below sag rod is 5 an using Eq 2.6 – Eq 2.7, forces on sag rod are calculated below.

$$F_{t,D} = 5\left(\frac{5000}{3} \times 26.842 \times 0.0098 \times sin\,33.27\right) = 1.7549 \text{ kN}$$

$$F_{t,L} = \frac{5}{2} \times 54.866 \times sin\,33.27 = 1.34512 \text{ kN}$$

By calculating the factored load in the load combination, it is known the computation as below.

1. 1.4 (1.7549 kN) = 2.457 kN
2. 1.2 (1.7549 kN) + 1.6 (1.34512 kN) = 4.258 kN (taken as F_T).

Then, diameter of sagrod can be calculated by using below.

$$A_{sr} = \frac{F_t . 10^3}{\emptyset F_y} \qquad \text{(Eq. 2.8)}$$

By using Eq. 2.8, it can be calculated as follow.

$$A_{sr} = \frac{4.258.10^3}{0.9 \times 240} = 19.7133 \text{ mm}^2$$

By using area of circle's principle, diameter of sag rod that is needed is computed to be 5 mm. Therefore, it is used diameter of 10 mm.

2.5.3 Calculation of Truss Structure

Planning Data

Distance between truss	= 4000 mm
Distance between purlin	= 1913.77 mm
Length of Truss	= 8000 mm
Slope of Truss	= 33.27o
Hypothenuse of roof	= 9568.83 mm
Weight of roof	= 6.994 kg/m2
Weight of Purlin	= 5.96 kg/m
Quality of Steel	= BJ37
Yield Strength	= 240 MPa
Fu	= 370 MPa
Modulus of Elasticity	= 200000 MPa
Unit Weight	= 7850 kg/m3
Modulus of Shear of Steel	= 77.200 MPa
Poisson ratio, U	= 0.3

Profile of 2L 90x90x9x9 is used for the Truss Planning.

A	= 50 mm	Iy	= 55.41 cm4
B	= 50 mm	ix	= 1.52 mm
t	= 5 mm	iy	= 2.4 mm
T	= 9 mm	Sx	= 6.18 cm^3
r1	= 6.5 mm	Sy	= 10.17 cm^3
r2	= 3 mm	x_0	= 0 mm
A	= 9.6 cm2	y_0	= 11.6 mm
W	= 7.54 kg/m	r_0	= 943.9975 mm
Ix	= 22.2 cm4	H	= 0.857

J $= 3958.33\ mm^3$

Calculation of Forces of Truss

The calculation of internal forces of the truss is done with MIDAS Gen. Using these combinations, table below describes the result of the structural analysis.

Table 2.21 Result of Structural Analysis of Truss Structure by MIDAS Gen

Combination	Compression (kN)	Tension (kN)
1,4D	-22.640591	5.395205
1.2D + 1,6L+ 0,5R	-33.247411	7.659326
1.2D + 1,6L+ 0,5W	-22.640591	5.395205
1.2D + 1W+ 1L + 0,5W	-28.326299	6.602311

Therefore, it is obtained that:

Compression Force (Max) = -33.2474 kN

Tension Force = 7.6593 kN

Calculation of Load Distribution

Dead Load

Calculation of dead load distribution

The input of self-weight of truss is done using MIDAS Gen's menu.

Load P1

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

$= \left(\frac{1.6}{2} + 1\right) m \times 4\ m \times 5.556\ \text{kg/m}^2$

$= 40.0032$ kg $= 0.392$ kN

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

$= \left(\frac{1.6}{2} + 1\right) \text{m} \times \frac{4\text{ m}}{\cos(33.27)}\ \text{m} \times\ 6.994$ kg/m^2

$= 60.232$ kg $= 0.591$ kN

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

= 4 m × 5.96 kg/m

= 23.84 kg = 0.234 kN

Other Load (taken 10%) = 10% (Weight of Roof + Weight of Purlin)

= 10% × (0.591 + 0.234) kN

= 0.082 kN

D = (0.392+0.591+0.234+0.082) kN

= 1.299 kN

<u>Load P2</u>

Weight of Ceiling = $a \times L \times$ Weight of Ceiling

= $1.6\ m \times 4\ m \times 5.556$ kg/m^2

= 35.5584 kg = 0.349 kN

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

$= \frac{1.6 \times 4\ \text{m}}{\cos(33.27)}$ m × 6.994 kg/m^2

= 53.54 kg = 0.525 kN

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

= 4 m × 5.96 kg/m

= 23.84 kg = 0.234 kN

Other Load (taken 10%) = 10% (Weight of Roof + Weight of Purlin)

= 10% × (0.525 + 0.234) kN

= 0.055 kN

D = (0.349+0.525+0.234+0.055) kN

= 1.163 kN

<u>Load P3</u>

Weight of Ceiling = $a \times L \times$ Weight of Ceiling

= $1.6\ m \times 4\ m \times 5.556$ kg/m^2

= 35.5584 kg = 0.349 kN

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

$= \frac{1.6 \times 4 \text{ m}}{\cos(33.27)}$ m $\times$ 6.994 kg/m^2

= 53.54 kg = 0.525 kN

Weight of Purlin = 2 × L × Weight of Purlin per m'

= 2 × 4 m × 5.96 kg/m

= 47.68 kg = 0.466 kN

Other Load (taken 10%) = 10% (Weight of Roof + Weight of Purlin)

= 10% × (0.525 + 0.468) kN

= 0.099 kN

D = (0.349+0.525+0.466+0.099) kN

= 1.439 kN

<u>Live Load</u>

L = 100 kg = 1000 N

Rain Load

R $= 40 - 0{,}8\alpha$ (kg/m^2) $\leq$ 20 kg/m^2 × D_{Purlin}

= 13.38 kg/m^2 x 1.91377 m

= 25.60624 kg/m = 0.251 kN/m

R_x = 0.251 kN/m x sin (33.27)

= 0.138 kN/m

Ry = 0.251 kN/m x cos (33.27)

= 0.21 kN/m

<u>Compression Member Control</u>

In order to check the profile used of the compression member of the truss structure, these following steps were calculated sequentially.

1. Check the maximum slenderness ratio. The limit ratio of the b/t will be compared to the equation below.

$$\lambda_r = 0.45\sqrt{\frac{E}{F_y}} \qquad \text{(Eq. 2.9)}$$

If the b/t is less than the result value of Eq. 2.9, the section of profile is non-slender. Otherwise, it will be considered as slender section.

$\frac{b}{t} = \frac{50}{5} = 10.$

$$\lambda_r = 0.45\sqrt{\frac{200000}{240}} = 12.990$$

Therefore, the section of compression member is non-slender section.

2. Check the flexural buckling of the structure. The value of KL/ix, is compared to the Fe and F_{cr}, using equation below.

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{i_x}\right)^2} \qquad \text{(Eq. 2.10)}$$

Then, it will be cheked using equation below.

$$4{,}71\sqrt{\frac{E}{F_y}}. \qquad \text{(Eq. 2.11)}$$

Using Eq 2.10-Eq 2.11, $\frac{KL}{i_x}$ and F_e is calculated as follows.

$$\frac{KL}{i_x} = \frac{1(1913.77)}{1.52} = 125.9057$$

$$F_e = \frac{\pi^2(200000)}{(125.9057)^2} = 124.52 \text{ MPa}$$

$$4{,}71\sqrt{\frac{E}{F_y}} = 4{,}71\sqrt{\frac{200000}{240}} = 135.97$$

Since the value of $\frac{KL}{i_x} = 125.9057 < 4{,}71\sqrt{\frac{E}{F_y}} = 135.97$, the equation below can be used to calculate the F_{cr}.

$$F_{cr} = 0.658^{\frac{F_y}{F_e}} \times F_y \qquad \text{(Eq. 2.12)}$$

Thefore, using Eq 2.12, F_{cr} can be calculated as follow.

$$F_{cr} = 0.658^{\frac{240}{135.97}} \times 240 = 113.724 \text{ MPa.}$$

3. Check the torsional buckling, with comparing the ratio of the length of the compression member (a) to the radius of gyration with the modified slenderness ratio $\left(\frac{KL}{r}\right)_m$, that expressed with equation below.

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{i_x}\right)^2 + \left(\frac{K_i a}{r_i}\right)^2} \qquad \text{(Eq 2.13)}$$

With K_i=0.5 for double angle section.

Therefore, by using Eq. 2.13, the calculation can be conducted as follow.

$$\frac{a}{i_x} = \frac{1913.77}{1.52} = 125.906$$

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

$$\left(\frac{KL}{i_x}\right)_m = \sqrt{\left(\frac{(1)(1913.77)}{1.52}\right)^2 + 0.25(125.906)^2} = 178.06$$

Since the value of $\left(\frac{KL}{i_x}\right)_m = 178.06 > 4{,}71\sqrt{\frac{E}{F_y}} = 135.97$, F_{cr} is calculated using Eq 2.5.14 Then, the value of F_{cry} can be calculated using Eq. 2.5.15. To calculate the F_{cr} due to torsional buckling, it is used the equation below.

$$F_{cr} = \left(\frac{F_{cry}+F_{crz}}{2H}\right)\left[1-\sqrt{1-\frac{4.F_{cry}.F_{crz}.H}{\left(F_{cry}+F_{crz}\right)^2}}\right] \qquad \text{(Eq. 2.14)}$$

$$F_{cry} = 0.877F_e \qquad \text{(Eq. 2.15)}$$

$$F_{crz} = \frac{GJ}{A \times \overline{r0}} \qquad \text{(Eq. 2.16)}$$

With J is torsional constant. For double angle profile, it can be calculated with:

$$J = \frac{(b+h-t)\times t^3}{3} \quad \text{(Eq. 2.17)}$$

Using Eq. 2.14 - Eq. 2.17, F_{cr} can be calculated as follow.

$$F_{crz} = \frac{77200 \times 3958.33}{960 \times 942.9975} = 337.558 \text{ MPa}$$

$$F_{cry} = 0.877\ (124.52) = 109.204 \text{ MPa}$$

$$F_{cr} = \left(\frac{337.558 \text{ MPa} + 109.204 \text{ MPa}}{2(0.857)}\right) \times \left[1 - \sqrt{1 - \frac{4.(109.204 \text{ MPa})(337.558 \text{ MPa})(0.857)}{(337.558 \text{ MPa} + 109.204 \text{ MPa})^2}}\right]$$

F_{cr} = 102.783 MPa

4. Check the design compression strength, by observing the value of F_{cr} due to flexural buckling and F_{cr} due to torsional flexural buckling, so it can be chosen the smallest value from both criterias.

   F_{cr} due to flexural buckling = 113.724 MPa

   F_{cr} due to torsional flexural buckling = 102.783 MPa

According to SNI 1727:2020, in order to check the design compression strength, equation below is used.

$$\emptyset_c P_n = 0{,}9 \times F_{cr} \times A_g \quad \text{(Eq. 2.18)}$$

Then, it will be compared to the maximum compression forces from the structural analysis. If the $\emptyset_c P_n$ exceeds the maximum compression forces, it can be said that the profile is safe.

Therefore, using Eq. 2.18, the calculation is conducted as follow.

$\emptyset_c P_n$=0,9×113.7235 MPa × 960 =88.80464 kN > 33.2474 kN

Therefore, designed compression forces of the truss is safe.

Tension Member Control

During tension member analysis, profile should be checked based on yielding limit condition, fracture limit condition, and block shear condition. However, slenderness of the tensile member should be checked too.

1. Check the slenderness of the profile using requirement below.

$$\lambda = \frac{L}{ix} < 300 \qquad \text{(Eq. 2.19)}$$

Using Eq 2.19, it is obtained the computation below.

$\lambda = \frac{1913.77}{1.52} = 125.9057 < 300$, so that the tension member is in slender condition.

2. Check the yielding limit condition of the profile, by using equation below.

$$Rn = Fy.Ag \qquad \text{(Eq. 2.20)}$$

Ag is the gross area of the member.

Therefore, it is obtained:

Rn = 240 MPa × 960 mm2 = 230.4 kN

ØRn = 0.9× 230.4 kN = 270.36 kN

3. Check the fracture limit condition of the profile, by using equation below.

$$Rn = fu.Ae \qquad \text{(Eq. 2.21)}$$

fu is the ultimate tensile strength, while Ae is effective area of the section. For double angle, Ae can be taken as 0.75 A_n, with A_n is the netto area of the section. For initial calculation, A_n can be taken as its largest value, which is 85% of the gross area of section. Therefore, the calculation can be done as follow.

$Rn = 370\ MPa \times 0.75 \times 0.85 \times 960\ mm2 = 226.44\ kN$

$ØRn = 0.75 \times 226.44\ kN = 169.83\ kN$

4. Check the block shear condition with equations below.

$Rn = 0,6\ Fu \times Anv + Ubs \times Fu \times Ant$ (Eq. 2.22)

$Rn = 0,6\ Fy \times Agv + Ubs \times Fu \times Ant$ (Eq. 2.23)

Calculation of gross area of shear section (Agv), net area of shear section (Anv) and net area of tensile section (Ant) is expressed in the steps below.

$Agv = 5\ mm \times 70\ mm = 350\ mm2$

$Anv = 5\ mm \times (70\ mm - (22.175\ mm + 11.0875\ mm)) = 176.18175\ mm2$

$Ant = 5\ mm \times (25\ mm – (0.5 \times 22.175\ mm)) = 67.0625\ mm2$

Using Eq. 2.22 and Eq. 2.23, these calculations are obtained.

$Rn = 0,6 \times 370 \times 176.18175 + 1 \times 370 \times 67.0625 = 63.926\ kN$

$Rn\ max = 0,6 \times 240 \times 350 + 1 \times 370 \times 67.0625 = 75.213\ kN$

The smallest value is chosen, Rn = 63.926 kN.

$ØRn = 0.75 \times 63.926\ kN = 47.945\ kN$

5. Check the tensile strength of the member by comparing the tension force with smallest design tension strength. Based on yielding limit condition, fracture limit condition, and block shear condition, block shear condition resulted the minimum design strength, which is 47.945 kN.

Therefore, to check the safety of the profile, the tension forces should be smaller than the design tension strength to be considered safe.

ØRn= 47.945 kN >Tension forces = 7.6593 kN.

Therefore, designed tension forces of the truss is safe.

2.5.4 Design of Over stack

Data Planning

Profil of 2L 90x90x9x9 is used for the over stack.

A	= 50 mm	ix	= 1.52 mm
B	= 50 mm	iy	= 2.4 mm
t	= 5 mm	Sx	= 6.18 cm^3
T	= 9 mm	Sy	= 10.17 cm^3
r1	= 6.5 mm	x_0	= 0 mm
r2	= 3 mm	y_0	= 11.6 mm
A	= 9.6 cm^2	r_0	= 943.9975 mm
W	= 7.54 kg/m	H	= 0.857
Ix	= 22.2 cm^4	J	= 3958.33 mm^3
Iy	= 55.41 cm^4		

From MIDAS Gen Analysis, it is obtained the force of the over stack that is expressed in Table 2.22 below.

Table 2.22 The Forces of Over Stack

Element	**Node-i**	**Node-j**
Left Over stack	15.223699	15.281329
Right Over stack	16.544179	16.601809

Therefore, the control of the over stack member will be considered as tension member, with tension forces of 16.544179 kN.

1. Check the slenderness of the profile using requirement below.

$$\lambda = \frac{L}{ix} < 300 \qquad \text{(Eq. 2.19)}$$

Using Eq. 2.19, it is obtained the computation below.

$\lambda = \frac{1196.351}{1.52} = 78.71 < 300$, so that the tension member is in slender condition.

2. Check the yielding limit condition of the profile, by using equation below.

$$Rn = Fy.Ag \qquad \text{(Eq. 2.20)}$$

Ag is the gross area of the member.

Therefore, it is obtained:

Rn = 240 MPa × 960 mm2 = 230.4 kN

ØRn = 0.9× 230.4 kN = 270.36 kN

3. Check the fracture limit condition of the profile, by using equation below.

$$Rn = fu.Ae \qquad \text{(Eq. 2.21)}$$

fu is the ultimate tensile strength, while Ae is effective area of the section. For double angle, Ae can be taken as 0.75 A_n, with A_n is the netto area of the section. For initial calculation, A_n can be taken as its largest value, which is 85% of the gross area of section. Therefore, the calculation can be done as follow.

Rn = 370 MPa × 0.75 × 0.85 × 960 mm^2 = 226.44 kN

ØRn = 0.75 × 226.44 kN = 169.83 kN

4. Check the block shear condition with equations below.

$$Rn = 0{,}6\ Fu \times Anv + Ubs \times Fu \times Ant \qquad \text{(Eq. 2.22)}$$

$$Rn = 0{,}6\ Fy \times Agv + Ubs \times Fu \times Ant \qquad \text{(Eq. 2.23)}$$

Calculation of gross area of shear section (Agv), net area of shear section (Anv) and net area of tensile section (Ant) is expressed in the steps below.

Agv = 5 mm × 70 mm = 350 mm^2

Anv = 5 mm × (70 mm - (22.175 mm+11.0875 mm))

= 176.18175 mm^2

Ant = 5 mm × (25 mm – (0.5 × 22.175 mm)) = 67.0625 mm^2

Using Eq. 2.5.22 and Eq. 2.5.23, these calculations are obtained.

$$Rn = 0{,}6 \times 370 \times 176.18175 + 1 \times 370 \times 67.0625 = 63.926 \text{ kN}$$

$$Rn\ max = 0{,}6 \times 240 \times 350 + 1 \times 370 \times 67.0625 = 75.213 \text{ kN}$$

The smallest value is chosen, Rn = 63.926 kN.

ØRn = 0.75 × 63.926 kN = 47.945 kN

5. Check the tensile strength of the member by comparing the tension force with smallest design tension strength. Based on yielding limit condition, fracture limit condition, and block shear condition, block shear condition resulted the minimum design strength, which is 47.945 kN.

Therefore, to check the safety of the profile, the tension forces should be smaller than the design tension strength to be considered safe.

ØRn= 47.945 kN >Tension forces = 16.544179 kN.

Therefore, designed over stack of the truss is safe.

2.5.5 Design of Roof Connection

In order to calculate the bolt connection, SNI 1729-2020 is used as a guideline of the calculation.

Bolt Data

Quality of bolt	= A325
Diameter of bolt, db	= 20 mm
Diameter of hole, ϕ	= 19 mm
Area of bolt (Ab)	= 201.06 mm2
Thickness of Gusset Plate	= 8 mm
Fy	= 250 MPa
Fu	= 410 MPa
Fnt	= 620 MPa
Fnv	= 372 MPa

1. Compression Member

For compression member, MIDAS GEN structural analysis obtained:

Vmax = 33.247 kN

Spacing and Distance of Edge Calculation

Spacing of bolt is calculated based on SNI 1729-2020, Article J3.3 and J3.5.

$2\frac{2}{3}$ db < Sb < 24 times of Thickness of Gusset Plate.

$2\frac{2}{3}(16) < Sb < 24(8)$

42.67 mm < Sb < 192 mm

Distance of edge of bolt is based on SNI 1729-2020, Article J3.4. and J3.5.

22 mm < Sedge < 12 times of thickness of gusset plate.

22 mm < Sedge < 12(8)

22 mm < Sedge < 96 mm

Therefore, the spacing between bolts and distance of bolt to the edge is taken as follows.

S_b = 45 mm

S_{edge} = 40 mm

Calculation of Nominal Strength of Bolt: Shear Strength

Calculation of shear strength of bolt is conducted based on SNI 1729-2020; Article J3.6.

$$R_n = F_{nv} A_b \qquad \text{(Eq. 2.24)}$$

Therefore, using Eq. 2.24, it can be obtained:

$R_n = 372\ (201.06\ mm^2) = 74.8\ kN$

Calculation of Nominal Strength of Bolt: Bearing Strength

Calculation of shear strength of bolt is used equation below.

$$R_n = 1.2 \times L_c \times t \times F_u \leq 2{,}4 \times db \times t \times Fu \qquad \text{(Eq. 2.25)}$$

Therefore, using Eq. 2.25., it can be obtained:

$R_n = 1.2 \times (100\text{-}16)\ mm \times 8\ mm \times 410\ N/mm^2 \leq 2{,}4 \times 16\ mm \times 8\ mm \times 410\ N/mm^2$

$R_n = 390.624\ kN \leq 125.952\ kN$ (It will be taken as R_n)

Check the required number of bolts

$$N = \frac{V}{\emptyset R_n}$$

$$N = \frac{33.247\ kN}{0.75\ (125.952)} = 0.352 \approx 2\ bolts$$

2. Tension Member

For tension member, MIDAS Gen structural analysis obtained:

$V_{max} = 7.567181\ kN$

Spacing and Distance of Edge Calculation

Spacing of bolt is calculated based on SNI 1729-2020, Article J3.3 and J3.5.

$2\frac{2}{3}\ db < S_b <$ 24 times of Thickness of Gusset Plate.

$2\frac{2}{3}(16) < S_b < 24(8)$

$42.67 \text{ mm} < S_b < 192 \text{ mm}$

Distance of edge of bolt is based on SNI 1729-2020, Article J3.4. and J3.5.

$22 \text{ mm} < S_{edge} < 12$ times of thickness of gusset plate.

$22 \text{ mm} < S_{edge} < 12(8)$

$22 \text{ mm} < S_{edge} < 96 \text{ mm}$

Therefore, the spacing between bolts and distance of bolt to the edge is taken as follows.

$S_b = 45 \text{ mm}$

$S_{edge} = 40 \text{ mm}$

Calculation of Nominal Strength of Bolt: Shear Strength

Calculation of shear strength of bolt is conducted based on SNI 1729-2020; Article J3.6.

$$R_n = F_{nv} A_b \qquad \text{(Eq. 2.24)}$$

Therefore, using Eq. 2.5.24, it can be obtained:

$R_n = 372 \, (201.06 \text{ mm}^2) = 74.8 \text{ kN}$

Calculation of Nominal Strength of Bolt: Bearing Strength

Calculation of shear strength of bolt is used equation below.

$$R_n = 1.2 \times L_c \times t \times F_u \leq 2{,}4 \times db \times t \times Fu \qquad \text{(Eq. 2.25)}$$

Therefore, using Eq. 2.5.25, it can be obtained:

$R_n = 1.2 \times (100\text{-}16) \text{ mm} \times 8 \text{ mm} \times 410 \text{ N/mm}^2 \leq 2{,}4 \times 16 \text{ mm} \times 8 \text{ mm} \times 410 \text{ N/mm}^2$

$R_n = 390.624 \text{ kN} \leq 125.952 \text{ kN}$ (It will be taken as R_n)

Check the required number of bolts

$N = \frac{V}{\emptyset R_n}$

$N = \frac{7.567181\ kN}{0.75\ (125.952)} = 0.08 \approx 2$ bolts.

2.6 The Design of Slab

2.6.1 Provision of Slab Design

When designing one-way slab, there are limitations that should be followed according to SNI 2847:2019.

1. Limitation of thickness of the slab

   SNI 2847:2019 ps.7.3.1.1. explained the limitation of the slab's thickness for non-prestressed solid slab that isn't supported or attached to partitions or other construction which may be damaged by large deflections. Table 2.23 shows the minimum thickness for non-prestressed solid slab that is categorized by the condition of the support. The table can be used for normal weight concrete, with the value of f_y = 420 MPa.

Table 2.23 Minimum Thickness for Non-Prestressed One-Way Solid Plate

Condition of Support	**Minimum thickness (h)**
Simple support	*l*/20
Continuous one side	*l*/24
Continuous two sides	*l*/28
Cantilever	*l*/10

2. The limitations for flexural reinforcement

   The minimum area of flexural reinforcements for non-prestressed one-way slab (it will be stated as $A_{s,min}$) is provided by SNI 2847:2019 ps. 7.6.1. Table 2.24 shows the provision that should be fulfilled that is categorized based on the type of the reinforcement.

Table 2.24 A_{smin} Non-Prestressed One-Way Solid Plate

<table>
<tr><th>Type of reinforcement</th><th>f_y (MPa)</th><th colspan="2">$A_{s,min}$</th></tr>
<tr><td>Threaded bar</td><td>< 420</td><td colspan="2">0.0020 A_g</td></tr>
<tr><td rowspan="2">Threaded bar or welding wire</td><td rowspan="2">≥ 420</td><td rowspan="2">Largest from:</td><td>$\frac{0.0018 \times 420}{f_y} \times A_g$</td></tr>
<tr><td>0.0014 A_g</td></tr>
</table>

3. Minimum shear reinforcement

According to SNI 2847:2019 ps. 7.6.3, $A_{v,min}$ should be provided in all sections if $V_u > \phi V_c$. For precast hollow slabs without concrete cover, the h should be more than 315 mm. $A_{v,min}$ also should be provided in all sections if $V_u > 0.5\ \phi V_{cw}$.

4. Minimum shrinkage reinforcement and temperature

According to SNI 2847:2019 ps. 24.4, should be installed perpendicular to the flexural reinforcement. For one way slab, table 2.25 shows the minimum of reinforcement ratio based on the type of the reinforcement.

Table 2.25 The Ratio of Area of Shrinkage Reinforcement and Temperature to the Gross Concrete Cross-Sectional Area

<table>
<tr><th>Type of reinforcement</th><th>f_y (MPa)</th><th colspan="2">Minimum Reinforcement Ratio</th></tr>
<tr><td>Threaded bar</td><td>< 420</td><td colspan="2">0.0020 A_g</td></tr>
<tr><td rowspan="2">Threaded bar or welding wire</td><td rowspan="2">≥ 420</td><td rowspan="2">Largest from:</td><td>$\frac{0.0018 \times 420}{f_y}$</td></tr>
<tr><td>0.0014</td></tr>
</table>

5. Maximum spacing of reinforcement in one way slab

SNI 2847:2019 ps. 7.7.2.3 explained that the maximum spacing for reinforcement of one wat slab should be less than 3h and 450 mm, with h is the thickness of the slab. The spacing that is installed also should not be more to the requirement that is listed on table 2.26 Below.

Table 2.26 Maximum Spacing of Bonded Reinforcement in One-Way Slabs and Non-Prestressed and Prestressed Beams Class C

Type of reinforcement	**Maximum Spacing (s)**	
Threaded or welding wire	Smallest of:	$\frac{380 \times \left(\frac{280}{f_s}\right) - 2.5\ C_C}{300 \times \left(\frac{280}{f_s}\right)}$
		$\frac{2}{3}\left[380 \times \left(\frac{280}{\Delta f_{Ps}}\right) - 2.5\ C_C\right]$

6. Minimum spacing of reinforcement in one-way slab

The minimum spacing of reinforcement in one-way slab is also the same as the minimum spacing of reinforcement in beams, which ruled out in SNI 2847:2019 ps. 25.2. It is listed that for non-prestressed reinforcement that is parallel to one horizontal layer, the clear spacing (noted as $s_{1,\ min}$) of reinforcement can't be less than the largest value of 25 mm, d_b, and $\frac{4}{3}\ d_{agg}$.

For parallel non-prestressing reinforcement that is installed in two or more horizontal layers, the reinforcement of the top layer should be placed directly above the bottom layer, with the clear spacing (noted as $s_{2,\ min}$) at least 25 mm.

2.6.2 Calculation of Reinforcement of One-Way Slab

The calculation of the moment of slab can use the approach method (*Metode Pendekatan*) that is ruled out in SNI 2847:2019 ps. 6.5. The load that can be considered during the analysis is gravitational loads. There are several requirements that should be followed so that the approach method can be applied during analysis.

1. Prismatic structural components,
2. The load is uniformly distributed,
3. The unfactored live load does not exceeding three times the unfactored dead load (L < 3D),
4. There are 2 spans or more, and
5. Ratio of the greatest span length to the smallest span length of two spans can't be more than 20%.

However, moment also can be determined by taking out the MIDAS Gen's output. During the reinforcement design, output of MIDAS Gen will be used.

According to the structural analysis that conducted using MIDAS Gen, it is obtained the moment and shear force as follows.

Mu(-) Field	: 5.702 kNm
Mu(+) Support	: 1.835 kNm
Vc	: 32.963 kN

Slab will be designed as 125 mm of thickness, using concrete cover of 20 mm, with longitudinal reinforcement of 10 mm. Therefore, the reinforcement will be calculated using these following steps that were conducted sequentially.

1. Calculate the effective height (d)

   $$d = 125 \text{ mm} - 20 \text{ mm} - \frac{10}{2} \text{ mm}$$
   $$= 100 \text{ mm.}$$

2. Calculate the shear strength of concrete (Vc) by equation below:

   $$\text{Vc} = 0.17 \times \lambda \times \sqrt{f'c} \times b_w \times d \qquad \text{(Eq. 2.26)}$$
   $$\text{ØVc} = 0.75 \text{ Vc} \qquad \text{(Eq. 2.27)}$$

   By using Eq. 2.26 and Eq. 2.27, the calculation is shown as follows:

$Vc \quad = 0.17 \times 1 \times \sqrt{30} \times 1000 \text{ mm} \times 100\ mm$

$= 93{,}113$ kN

$Vc = 0.75\ (93{,}113) = 69{,}835$ kN

According to the MIDAS Output, design shear strength of the slab is bigger than the shear force resulted by MIDAS Gen. Therefore, the slab is compatible to resist the shear force.

3. Calculation of β1

Value of β1 is calculated according to SNI 2847:2019 Ps. 22.2.2.4.3.

$$\beta 1 = 0.85 - \frac{0.05(30-28)}{7} = 0.84$$

4. Calculation of longitudinal reinforcement in support area

Calculation of coefficient of flexural resistance

To calculate the coefficient of flexural resistance, it can use the equation below:

$$k = \frac{M_n}{bd^2} = \frac{M_u}{\emptyset bd^2} \qquad \text{(Eq. 2.28)}$$

By using Eq. 2.28, the coefficient of flexural resistance can be determined as follow.

$$k = \frac{5{,}702{,}000}{0.9(1000)(100^2)} = 0.634$$

Calculation of flexural reinforcement ratio

To calculate the coefficient of flexural reinforcement ratio, it can use the equation below:

$$\rho = \frac{0.85f'_c}{f_y}\left(1 - 1\sqrt{1 - \frac{2k}{0.85f'_c}}\right) \qquad \text{(Eq. 2.29)}$$

Then, check it with ρmax with equation below:

$$\rho_{max} = 0.36\frac{0.85f'_c \times \beta_1}{f_y} \qquad \text{(Eq. 2.30)}$$

By using Eq. 2.29 and Eq. 2.30, the flexural reinforcement ratio can be calculated as follow.

$$\rho = \frac{0.85(30)}{420}\left(1 - 1\sqrt{1 - \frac{2(0.634)}{0.85(30)}}\right) = 0.0015$$

$$\rho_{max} = 0.36\frac{0.85(30) \times 0.84}{420} = 0.018$$

Since the value of ρ is less than ρmax, ρ can be used in the next calculation.

<u>Calculation of the required area of tensile reinforcement</u>

To calculate the required area of tensile reinforcement, it can use the equation below:

$$A_{s.req} = \rho\ b\ d \qquad \text{(Eq. 2.31)}$$

Then, check with the minimum limit of reinforcement area according to Table 2.25, then As,used can be determined.

Using Eq. 2.31, $A_{s.req}$ can be calculated as follow.

$$A_{s.req} = 0.0015 \times 1000 \times 100 = 152{,}769 \text{ mm}^2$$

$A_{s,min}$ is calculated as follow:

$$A_{s,min} = 0.002\ (125 \times 1000) = 250 \text{ mm}^2$$

Since the $A_{s,min} > A_{s,req}$, $A_{s,min}$ will be used.

<u>Calculation of spacing of installed flexural reinforcement</u>

To calculate the spacing of installed flexural reinforcement, the equation below can be used, then check the maximum spacing according to SNI 2847:2019 ps. 7.7.2.3

$$s = \frac{0.25 \times \pi \times D^2 \times b}{A_{s.used}} \qquad \text{(Eq. 2.32)}$$

Using Eq. 2.32, the spacing can be determined as follow.

$$s = \frac{0.25 \times \pi \times (10)^2 \times 1000}{250 \ \text{mm}^2} = 314.159 \text{ mm}$$

According to SNI 2847:2019, Article 7.7.2.3, maximum spacing for slab should be less than 3h = 375 mm, or 450 mm. Therefore, for support area, it is used **D10-200**.

5. Calculation of shrinkage reinforcement in the support area

For shrinkage reinforcement, diameter of reinforcement that is used is 12 mm.

Calculation of spacing

To calculate the spacing, equation below can be used.

$$s = \frac{0.25 \times \pi \times D^2 \times b}{A_{s.used}} \quad \text{(Eq. 2.32)}$$

Using Eq. 2.32, the spacing can be determined as follow.

$$s = \frac{0.25 \times \pi \times (10)^2 \times 1000}{250 \ \text{mm}^2} = 314.159 \text{ mm}$$

SNI 2847:2019 Article 7.7.7.4 limits the maximum spacing of shrinkage reinforcement, that is 5h = 5(125) = 625 mm, and 450 mm.

Therefore, for support area, it is used **D10-200**.

6. Calculation of longitudinal reinforcement in field area

Calculation of coefficient of flexural resistance

To calculate the coefficient of flexural resistance, it can use the equation below:

$$k = \frac{M_n}{bd^2} = \frac{M_u}{\emptyset bd^2} \quad \text{(Eq. 2.28)}$$

By using Eq. 2.28, the coefficient of flexural resistance can be determined as follow.

$$k = \frac{1{,}835{,}000}{0.9(1000)(100^2)} = 0.204$$

Calculation of flexural reinforcement ratio

To calculate the coefficient of flexural reinforcement ratio, it can use the equation below:

$$\rho = \frac{0.85f'_c}{f_y}\left(1 - 1\sqrt{1 - \frac{2k}{0.85f'_c}}\right) \qquad \text{(Eq. 2.29)}$$

Then, check it with ρ_{max} with equation below:

$$\rho_{max} = 0.36\frac{0.85f'_c \times \beta_1}{f_y} \qquad \text{(Eq. 2.30)}$$

By using Eq. 2.29 and Eq. 2.30, the flexural reinforcement ratio can be calculated as follow.

$$\rho = \frac{0.85(30)}{420}\left(1 - 1\sqrt{1 - \frac{2(0.204)}{0.85(30)}}\right) = 0.005$$

$$\rho_{max} = 0.36\frac{0.85(30) \times 0.84}{420} = 0.0186$$

Since the value of ρ is less than ρmax, ρ can be used in the next calculation.

Calculation of the required area of tensile reinforcement

To calculate the required area of tensile reinforcement, it can use the equation below:

$$A_{s.req} = \rho\ b\ d \qquad \text{(Eq. 2.31)}$$

Then, check with the minimum limit of reinforcement area according to Table 2.25, then As,used can be determined.

Using Eq. 2.31, $A_{s.req}$ can be calculated as follow.

$A_{s.req} = 0.005 \times 1000 \times 100 = 48.74$ mm^2

$A_{s,min}$ is calculated as follow:

$A_{s,min}$ = 0.002 (125 × 1000) = 250 mm^2

Since the $A_{s,min} > A_{s,req}$, $A_{s,min}$ will be used.

Since the $A_{s,min2} < A_{s,min1} < A_{s,req}$, $A_{s,req}$ will be used.

Calculation of spacing of installed flexural reinforcement

To calculate the spacing of installed flexural reinforcement, the equation below can be used, then check the maximum spacing according to SNI 2847:2019 ps. 7.7.2.3

$$s = \frac{0.25 \times \pi \times D^2 \times b}{A_{s.used}} \quad \text{(Eq. 2.32)}$$

Using Eq. 2.32, the spacing can be determined as follow.

$$s = \frac{0.25 \times \pi \times (10)^2 \times 1000}{250\ mm^2} = 314.159 \text{ mm}$$

According to SNI 2847:2019, Article 7.7.2.3, maximum spacing for slab should be less than 3h = 375 mm, or 450 mm. Therefore, for support area, it is used **D10-200**.

2.6.3 Calculation of Reinforcement for Two-Way Slab

The calculation of the moment will be used the output result from MIDAS Gen.

According to the structural analysis that conducted using MIDAS Gen, it is obtained the moment and shear force as follows.

Mmax of X dir	: 4.351 kNm
Mmin of X dir	: -1.395 kNm
Mmax of Y dir	: 1.909 kNm
Mmin of Y dir	: -8.298 kNm
Vc	: 18.585 kN

Slab will be designed as 125 mm of thickness, using concrete cover of 20 mm, with longitudinal reinforcement of 10 mm. Therefore, the reinforcement will be calculated using these following steps that were conducted sequentially.

Slab that will be designed is 4x3 m floor slab, then with the same calculation to determine effective thickness of the slab, it is obtained d=100 mm.

1. Flexural reinforcement (Field area for bottom reinforcement) for Positive Moment in X-direction

   Reinforcement spacing, s = 200 mm

   Maximum spacing, s_{max} = 2t = 250 mm, and 450 mm (SNI 2847:2019 ps. 8.7.2.2)

   Total reinforcement = 1000/s = 5

   Clear distance of reinforcement = $S-d_b$

   = 200 – 10

   = 190 mm

   According to SNI 2847:2019 ps 25.2.1, the minimal clear spacing of reinforcement is 25 mm. Therefore, clear distance of reinforcement is already fulfilling requirements.

   $$A_{s,\ min\ 1} = \frac{0.18\%(420)(b)(t)}{fy}$$

   $$= \frac{0.18\%(420)(1000)(125)}{125}$$

   $$= 225\ mm^2$$

   $$A_{s,installed} = n\ (0.25)\ (d_b^2)$$

   $$= 5\ (0.25)\ (10)^2$$

   $$= 392.699\ mm^2$$

   Since $A_{s,installed}$ exceeds the minimum area, then it can be concluded that reinforcement is installed properly.

   $$a = \frac{As(fy)}{0.85(f'c)(b)}$$

a $= \frac{392.699\,(420)}{0.85(30)(1000)}$

a = 6.468 m

Flexural capacity, Mn $= A_s\,(fy)\,(d-\frac{a}{2})$

$= 392.699\,(420)\,(100-\frac{6.468}{2})$

= 15.960 kNm

c $= \frac{a}{\beta 1}$

$= \frac{6.468}{0.84}$

= 7.739

ε_s $= \frac{0.003(d-c)}{c}$

$= \frac{0.003(100-7.739)}{7.739}$

= 0.036

Reduction factor, ϕ = 0.9

ϕMn = 14.364 kNm

Observing that the Mu = 4.351 kNm is less than ϕMn = 14.364 kNm, therefore the moment capacity of the slab reinforcement is fulfilling the requirement.

2. Flexural reinforcement (Support area for upper reinforcement) for Negative Moment in X-direction

Reinforcement spacing, s = 200 mm

Maximum spacing, s_{max} = 2t = 250 mm, and 450 mm (SNI 2847:2019 ps. 8.7.2.2)

Total reinforcement = 1000/s = 5

Clear distance of reinforcement $= S-d_b$

= 200 – 10

= 190 mm

According to SNI 2847:2019 ps 25.2.1, the minimal clear spacing of reinforcement is 25 mm. Therefore, clear distance of reinforcement is already fulfilling requirements.

$A_{s,\ min\ 1}$ $= \frac{0.18\%(420)(b)(t)}{fy}$

$= \frac{0.18\%(420)(1000)(125)}{125}$

$= 225\ mm^2$

$A_{s,installed}$ $= n\ (0.25)\ (d_b^2)$

$= 5\ (0.25)\ (10)^2$

$= 392.699\ mm^2$

Since $A_{s,installed}$ exceeds the minimum area, then it can be concluded that reinforcement is installed properly.

a $= \frac{As(fy)}{0.85(f'c)(b)}$

a $= \frac{392.699\ (420)}{0.85(30)(1000)}$

a $= 6.468$ m

Flexural capacity, Mn $= A_s\ (fy)\ (d-\frac{a}{2})$

$= 392.699\ (420)\ (100-\frac{6.468}{2})$

$= 15.960$ kNm

c $= \frac{a}{\beta 1}$

$= \frac{6.468}{0.84}$

$= 7.739$

ε_s $= \frac{0.003(d-c)}{c}$

$= \frac{0.003(100-7.739)}{7.739}$

$= 0.036$

Reduction factor, ϕ $= 0.9$

ϕMn $= 14.364$ kNm

Observing that the Mu = 1.395 kNm is less than ϕMn = 14.364 kNm, therefore the moment capacity of the slab reinforcement is fulfilling the requirement.

3. Flexural reinforcement (Field area for bottom reinforcement) for Positive Moment in Y-direction

Reinforcement spacing, s = 200 mm

Maximum spacing, s_{max} = 2t = 250 mm, and 450 mm (SNI 2847:2019 ps. 8.7.2.2)

Total reinforcement = 1000/s = 5

Clear distance of reinforcement = $S-d_b$

= 200 – 10

= 190 mm

According to SNI 2847:2019 ps 25.2.1, the minimal clear spacing of reinforcement is 25 mm. Therefore, clear distance of reinforcement is already fulfilling requirements.

$A_{s,\ min\ 1}$ $= \frac{0.18\%(420)(b)(t)}{fy}$

$= \frac{0.18\%(420)(1000)(125)}{125}$

= 225 mm^2

$A_{s,installed}$ = n (0.25) (d_b^2)

= 5 (0.25) (10)^2

= 392.699 mm^2

Since $A_{s,installed}$ exceeds the minimum area, then it can be concluded that reinforcement is installed properly.

a $= \frac{As(fy)}{0.85(f'c)(b)}$

a $= \frac{392.699\ (420)}{0.85(30)(1000)}$

a = 6.468 m

Flexural capacity, Mn $= A_s\ (fy)\ (d-d_b-\frac{a}{2})$

$= 392.699\ (420)\ (100-10-\frac{6.468}{2})$

= 14.311 kNm

c $= \frac{a}{\beta 1}$

$= \frac{6.468}{0.84}$

= 7.739

ε_s $= \frac{0.003(d-c)}{c}$

$= \frac{0.003(100-7.739)}{7.739}$

= 0.036

Reduction factor, ϕ = 0.9

ϕMn = 12.880 kNm

Observing that the Mu = 1.395 kNm is less than ϕMn = 1.909 kNm, therefore the moment capacity of the slab reinforcement is fulfilling the requirement.

4. Flexural reinforcement (Support area for upper reinforcement) for Negative Moment in Y-direction

Reinforcement spacing, s = 200 mm

Maximum spacing, s_{max} = 2t = 250 mm, and 450 mm (SNI 2847:2019 ps. 8.7.2.2)

Total reinforcement = 1000/s = 5

Clear distance of reinforcement $= S-d_b$

= 200 – 10

= 190 mm

According to SNI 2847:2019 ps 25.2.1, the minimal clear spacing of reinforcement is 25 mm. Therefore, clear distance of reinforcement is already fulfilling requirements.

$A_{s,\ min\ 1}$ $= \frac{0.18\%(420)(b)(t)}{fy}$

$= \frac{0.18\%(420)(1000)(125)}{125}$

= 225 mm^2

$A_{s,installed}$ = n (0.25) (d_b^2)

$= 5\ (0.25)\ (10)^2$

$= 392.699\ mm^2$

Since $A_{s,installed}$ exceeds the minimum area, then it can be concluded that reinforcement is installed properly.

a $= \frac{As(fy)}{0.85(f'c)(b)}$

a $= \frac{392.699\ (420)}{0.85(30)(1000)}$

a = 6.468 m

Flexural capacity, Mn $= A_s\ (fy)\ (d-d_b-\frac{a}{2})$

$= 392.699\ (420)\ (100-10-\frac{6.468}{2})$

= 14.311 kNm

c $= \frac{a}{\beta 1}$

$= \frac{6.468}{0.84}$

= 7.739

ε_s $= \frac{0.003(d-c)}{c}$

$= \frac{0.003(100-7.739)}{7.739}$

= 0.036

Reduction factor, ϕ = 0.9

ϕMn = 12.880 kNm

Observing that the Mu = 1.395 kNm is less than ϕMn = 8.298 kNm, therefore the moment capacity of the slab reinforcement is fulfilling the requirement.

5. Minimum reinforcement for upper and lower reinforcement in X and Y- direction

Reinforcement spacing, s = 200 mm

Maximum spacing, s_{max} = 2t = 250 mm, and 450 mm (SNI 2847:2019 ps. 8.7.2.2)

Total reinforcement = 1000/s = 5

Clear distance of reinforcement = $S\text{-}d_b$

= 200 – 10

= 190 mm

According to SNI 2847:2019 ps 25.2.1, the minimal clear spacing of reinforcement is 25 mm. Therefore, clear distance of reinforcement is already fulfilling requirements.

Layer = 2

$A_{s,\ min\ 1}$ $= \frac{0.18\%(420)(b)(t)}{fy}$

$= \frac{0.18\%(420)(1000)(125)}{125}$

= 225 mm^2

$A_{s,installed}$ = n (0.25) (d_b^2)

= 5 (0.25) (10)^2

= 392.699 mm^2

Since $A_{s,installed}$ exceeds the minimum area, then it can be concluded that reinforcement is installed properly.

6. Shear capacity check

V_c $= 0.17 \times \lambda \times \sqrt{f'c} \times b_w \times d$

$= 0.17 \times 1 \times \sqrt{30} \times 1000 \times 100$

= 93113 N = 93.113 kN

Φ = 0.75

0.5 (ϕ) (V_c) = 39.917 kNm

Since V_u = 18.585 kNm < 0.5 (ϕ) (V_c) = 39.917 kNm, then the shear capacity has been fulfilled the requirement.

2.7 The Design of Stairs

2.7.1 Geometrical Design of Stairs

Floor Height	= 4000 mm
Stair Width	= 2850 mm
Optrade, O	= 180 mm = 0,18 m
Aptrede, A	= 280 mm
Bordes Width	= 1200 mm
Thickness	= 130 mm = 0,13 m

<u>Stair Calculation (Based on SNI 1727-2020 table 4.3-1)</u>

Optrede	= 22,22222222 ~ 22
Aptrede	= 21
Stair Length	= ΣOptrede×Aptrede width
	= 22×280= 6160 mm

Slope of Stairs

Slope	= O/A= 0,642857143
α	= 32,73522627 ~ 33 (25 < α < 40)

Therefore, the slope of stairs is fulfilled the requirement.

Stair Ramps

2O + A = 640 mm (60 < 2O + A < 65)

Therefore, the stair ramps are fulfilled the requirement.

<u>Calculation of Load</u>

Calculation of Stairs Loading

1. Stairplate load

   <u>Dead load</u>

   Stairplate load = t/(cos∝)×24 kN /m³

   = 0.154545086 x 24 kN/m³

   = 3.709082069 kN/m²

Staircase load = ½ O x 24 kN/ m^3

= 2.16 kN/m^2

Dead Load = 5.869082069 kN/m^2

Super imposed dead load

Space load = 0.21 kN/m

Tile load = 0.24 kN/m

Railing load = 1 kN/m

SIDL = 1.45 kN/ m^2

Live load

Emergency path = 4.79 kN/ m^2

LL = 4.79 kN/m^2

Total Dead Load

DL + SIDL = 7.319 kN/m2

Load Conclusion

qDL = 7,319 kN/m2

qLL = 4,79 kN/m2

q ultimate = 1,2qDL + 1,6qLL

= 16.447 kN/m2

2. Bordes load

Dead load

Bordes load = t×24 kN/m^3

= 0,13 x 24 kN/m^3

= 3.12 kN/m^2

DL = 3.12 kN/m^2

Super imposed dead load

Space load = 0,21 kN/m

Tile load = 0,24 kN/m

Railing load = 1 kN/m

SIDL = 1,45 kN/ m^2

Live load

Emergency path = 4.79

LL = 4.79 kN/m^2

Total Dead Load

DL + SIDL = 4.57 kN/m^2

Conclusion:

qDL = 4.57 kN/m^2

qLL = 4.79 kN/m^2

q ultimate = 1.2qDL + 1.6qLL

= 13.148 kN/m^2

2.7.2 Reinforcement Design of Stairs

In order to design the reinforcement of stairs, internal forces from structural analysis are needed to be obtained. From MIDAS Gen, it will be obtained both M_{xx} and M_{yy}, while M_{xx} will be needed for transverse reinforcement and M_{yy} will be needed for longitudinal reinforcement. For reinforcement design, it is known that the output from MIDAS Gen is as follows.

$M_{xx, support}$ = 3.4 kNm

$M_{xx, field}$ = 1.1 kNm

$M_{yy, support}$ = 13.5 kNm

$M_{yy, field}$ = 10 kNm

Longitudinal Reinforcement (Support Area)

M_u = 13.5 kNm

Assume that D13-300 will be installed, then area of reinforcement for b = 1 meter of stairs is:

A_{rein} $= \frac{0.25\pi Ø^2}{\frac{1000}{s}}$

A_{rein} $= \frac{0.25(13)^2}{\frac{1000}{300}}$

A_{rein} = 442.44 mm^2

d $= h_{stairs} - C_c - \frac{Ø}{2}$

d = 125 mm – 20 mm – $\frac{13}{2}$ mm

d = 98.5 mm

a $= \frac{A_{Rein} \times fy}{\phi f\prime cb}$

a $= \frac{442.44 \times 420}{0.85(30)(1000)}$

a = 7.29 mm

$ØM_n$ $= 0.9(A_{rein})(fy)(d - \frac{a}{2})$

$ØM_n$ $= 0.9(442.44)(420)(98.5 - \frac{7.29}{2})$

$ØM_n$ = 15,864,033.75 Nmm = 15.86 kNm

Since $ØM_n > M_u$, it can be said that the reinforcement design is already be able to resist the moment.

To check the capacity from shear force:

V_u = 13.4 kNm

$ØV_c$ $= 0.75(0.17)\ (\lambda)\sqrt{fc'}bw\ d$

$ØV_c$ $= 0.75(0.17)\ (1)\sqrt{30'}(1000)\ (98.5)$

$ØV_c$ = 68787.107 N = 68.78 kN

Since $ØV_c > V_u$, it can be said that the reinforcement design is already safe from shear force.

Longitudinal Reinforcement (Field Area)

M_u = 10 kNm

Assume that D13-300 will be installed, then area of reinforcement for b = 1 meter of stairs is:

$A_{rein} = \frac{0.25\pi Ø^2}{\frac{1000}{s}}$

$A_{rein} = \frac{0.25(13)^2}{\frac{1000}{300}}$

$A_{rein} = 442.44\ mm^2$

$d = h_{stairs} - C_c - \frac{Ø}{2}$

$d = 125\ mm - 20\ mm - \frac{13}{2}\ mm$

$d = 98.5\ mm$

$a = \frac{A_{Rein} \times fy}{\phi f'cb}$

$a = \frac{442.44 \times 420}{0.85(30)(1000)}$

$a = 7.29\ mm$

$ØM_n = 0.9(A_{rein})(fy)(d - \frac{a}{2})$

$ØM_n = 0.9(442.44)(420)(98.5 - \frac{7.29}{2})$

$ØM_n$ = 15,864,033.75 Nmm = 15.86 kNm

Since $ØM_n > M_u$, it can be said that the reinforcement design is already be able to resist the load.

To check the capacity from shear force:

V_u = 13.4 kNm

$ØV_c$ $= 0.75(0.17)\ (\lambda)\sqrt{fc'}bw\ d$

$ØV_c$ $= 0.75(0.17)\ (1)\sqrt{30'}(1000)\ (98.5)$

$ØV_c$ = 68787.107 N = 68.78 kN

Since $ØV_c > V_u$, it can be said that the reinforcement design is already safe from shear force.

Transversal Reinforcement (Support Area)

M_u = 3.4 kNm

Assume that P10-300 will be installed, then area of reinforcement for b = 1 meter of stairs is:

A_{rein} $= \frac{0.25\pi Ø^2}{\frac{1000}{s}}$

A_{rein} $= \frac{0.25(10)^2}{\frac{1000}{300}}$

A_{rein} = 261.8 mm^2

d $= h_{stairs} - C_c - \frac{Ø}{2}$

d $= 125\text{ mm} - 20\text{ mm} - \frac{10}{2}\text{ mm}$

d = 100 mm

a $= \frac{A_{Rein} \times fy}{\phi f\prime cb}$

a $= \frac{261.8 \times 280}{0.85(30)(1000)}$

a = 2.87 mm

$ØM_n$ $= 0.9(A_{rein})(fy)(d - \frac{a}{2})$

$ØM_n$ $= 0.9(261.8)(280)(100 - \frac{2.87}{2})$

$ØM_n$ = 6,502,518.96 Nmm = 6.5 kNm

Since $ØM_n > M_u$, it can be said that the reinforcement design is already be able to resist the load.

To check the capacity from shear force:

V_u = 13.4 kNm

$ØV_c$ = $0.75(0.17)\ (\lambda)\sqrt{fc'}bw\ d$

$ØV_c$ = $0.75(0.17)\ (1)\sqrt{30'}(1000)\ (100)$

$ØV_c$ = 69,834.62608 N = 69.83 kN

Since $ØV_c > V_u$, it can be said that the reinforcement design is already safe from shear force.

<u>Transversal Reinforcement (Field Area)</u>

M_u = 1.1 kNm

Assume that P10-300 will be installed, then area of reinforcement for b = 1 meter of stairs is:

A_{rein} = $\frac{0.25\pi Ø^2}{\frac{1000}{s}}$

A_{rein} = $\frac{0.25(10)^2}{\frac{1000}{300}}$

A_{rein} = 261.8 mm^2

d = $h_{stairs} - C_c - \frac{Ø}{2}$

d = 125 mm – 20 mm – $\frac{10}{2}$ mm

d = 100 mm

a = $\frac{A_{Rein} \times fy}{\phi f\prime cb}$

a = $\frac{261.8 \times 280}{0.85(30)(1000)}$

a = 2.87 mm

$ØM_n$ = $0.9(A_{rein})(fy)(d - \frac{a}{2})$

$ØM_n$ = $0.9(261.8)(280)(100 - \frac{2.87}{2})$

$ØM_n$ = 6,502,518.96 Nmm = 6.5 kNm

Since $ØM_n > M_u$, it can be said that the reinforcement design is already be able to resist the load.

To check the capacity from shear force:

V_u = 13.4 kNm

$ØV_c$ = $0.75(0.17)\ (\lambda)\sqrt{fc'}bw\ d$

$ØV_c$ = $0.75(0.17)\ (1)\sqrt{30'}(1000)\ (100)$

$ØV_c$ = 69,834.62608 N = 69.83 kN

Since $ØV_c > V_u$, it can be said that the reinforcement design is already safe from shear force.

2.8 The Design of Beam

Based on SNI 2847:2019 Table 21.2.2 for planning reinforced concrete beams, it is necessary to determine the strength reduction factor of the structure experiencing bending and axial force can be seen in Figure 2.11.

Regangan tarik netto (ε_t)	Klasifikasi	ϕ			
		Jenis tulangan transversal			
		Spiral sesuai 25.7.3		Tulangan lainnya	
$\varepsilon_t \leq \varepsilon_{ty}$	Tekanan terkontrol	0,75	a)	0,65	b)
$\varepsilon_{ty} < \varepsilon_t < 0{,}005$	Transisi[1]	$0{,}75+0{,}15\dfrac{(\varepsilon_t-\varepsilon_{ty})}{(0{,}005-\varepsilon_{ty})}$	c)	$0{,}65+0{,}25\dfrac{(\varepsilon_t-\varepsilon_{ty})}{(0{,}005-\varepsilon_{ty})}$	d)
$\varepsilon_t \geq 0{,}005$	Tegangan terkontrol	0,90	e)	0,90	f)

[1] Untuk penampang transisi, diperbolehkan memakai nilai faktor kekuatan sama dengan penampang terkontrol tekan

Figure 2.22 Strength Reduction Factor for Moments, Axial Forces, Or a Combination of Moments and Axial Forces

During design of beam reinforcement, it should be noticed the support and field section of the beam. It will consider the negative and positive moment in field and support area, transversal reinforcement, and torsional reinforcement. Table 2.27 shows the internal forces output from MIDAS Gen for Beam B1.

Table 2.27 Internal Forces of Beam B1

B1 (350x600 mm)		
	M_{u-} (kNm)	M_{u+} (kNm)
Left	478.68	263.45
Mid	325.69	162.02
Right	478.68	263.45
	Support	Field
Vu (kN)	411.17	407.25
Tu	83.66	83.66

2.8.1 Longitudinal Reinforcement

Left End (Negative Moment)

Assume it will be designed with 2 layered reinforcements, with 25 mm of longitudinal reinforcement and 13 mm of transversal reinforcement.

$$d = h - C_c - (1.5\text{Ø}_{longitudinal}) - \text{Ø}_{transversal} - 25 \text{ mm}$$

$$d = 600 \text{ mm} - 40 \text{ mm} - 1.5\ (25) \text{ mm} - 13 \text{ mm} - 25 \text{ mm}$$

$$d = 484.5 \text{ mm}$$

$$\alpha = d - \sqrt{d^2 - \frac{2\times[Mu]}{0.85\ fc'\phi b}}$$

$$\alpha = 484.5 \text{ mm} - \sqrt{(484.5 \text{ mm})^2 - \frac{2\times[478.68 \times 10^6 \text{Nmm}]}{0.85\ (30)(0.9)(350 \text{ mm})}}$$

$$\alpha = 144.57 \text{ mm}$$

$$\beta_1 = 0.85 - \frac{0.05(fc-28)}{7}$$

(From SNI 2847:2019 Ps. 22.2.2.4.3)

$$\beta_1 = 0.85 - \frac{0.05(30-28)}{7}$$

$$\beta_1 = 0.84$$

$$c = \frac{a}{\beta 1}$$

$$c = \frac{144.57\ mm}{0.84}$$

$$c = 172.103 \text{ mm}$$

c_{max} = 0.375d

c_{max} = 0.375 (484.5 mm)

c_{max} = 181.69 mm

See that $c<c_{max}$, therefore the beam is already tension controlled (Φ=0.9).

$$A_s = \frac{Mu}{\Phi fy(d-\frac{a}{2})}$$

$$A_s = \frac{478.68}{0.9\ (420)\ (484.5 - \frac{144.57}{2})}$$

A_s = 3072.048 mm^2

$$n = \frac{(3072.048)}{0.25\pi(25)^2}$$

n = 6.25 **(use 7D25)**

$A_{s,used}$ = n(0.25) (π) ($ø^2$)

$A_{s,used}$ = 7 (0.25) (π) (25^2)

$A_{s,used}$ = 3436.117 mm^2

Minimum criteria for As,used:

$A_{s,max}$ = 0.025bd^2

$A_{s,max}$ = 0.025(350)$(484.5\ mm)^2$

$A_{s,max}$ = 4239.375 mm^2

$$A_{s,min\ 1} = \frac{\sqrt{fc'}bd}{4fy}$$

$$A_{s,min\ 1} = \frac{\sqrt{30}\ (350)\ (484.5)}{4(420)}$$

$A_{s,min\ 1}$ = 552.86 mm^2

$$A_{s,min\ 2} = \frac{1.4bd}{fy}$$

$$A_{s,min\ 2} = \frac{1.4\ (350)(484.5)}{(420)}$$

$A_{s,min\ 2}$ = 565.25 mm^2

Since $A_{s,used}$ exceeds both $A_{s,min\ 1}$ and $A_{s,min\ 2}$ but still under the value of $A_{s,max}$, therefore number of reinforcement is already controlled.

s $= \frac{b-2Cc-2\ ø_{transversal}-(nDø_{longitudinal})}{n-1}$

s $= \frac{350-2(40)-2\ (13)-(7(25))}{7-1}$

s = 11.5 mm

Since s is under 25 mm, the arrangement of reinforcement should be done in two layers, as it is assumed before. The arrangement will be 5 bars above and 2 bars below.

$S_{5,\ bars}$ $= \frac{b-2Cc-2\ ø_{transversal}-(nDø_{longitudinal})}{n-1}$

$S_{5,\ bars}$ $= \frac{350-2(40)-2\ (13)-(5(25))}{5-1}$

$S_{5,\ bars}$ = 29.75 mm (OK!)

$S_{2,\ bars}$ $= \frac{b-2Cc-2\ ø_{transversal}-(nDø_{longitudinal})}{n-1}$

$S_{2,\ bars}$ $= \frac{350-2(40)-2\ (13)-(2(25))}{2-1}$

$S_{2,\ bars}$ = 194 mm (OK!)

Nominal moment should be checked as follow.

M_n $= A_s \times fy \times \left(d - \frac{\alpha}{2}\right)$

M_n = 3436.7 mm^2 × 420 MPa × $\left(484.5 \text{ mm} - \frac{144.57 \text{ mm}}{2}\right)$

M_n = 594,898,123 Nmm

ϕMn = 0.9 (594,898,123 Nmm)

ϕMn = 535,408,311 Nmm

($\phi Mn > M_u$ = 478,680,000 Nmm)

Therefore, reinforcement is already designed well to control the moment.

Left End (Positive Moment)

$M_{u,used}$ = largest value of (263.45 kNm or $0.5M_{u(-)}$)

$M_{u,used}$ = 263.45 kNm

Assume that it will be designed in one layer.

d = $h - C_c - (0.5\text{Ø}_{longitudinal}) - \text{Ø}_{transversal}$

d = 600 mm – 40 mm – 0.5 (25) mm – 13 mm

d = 534.5 mm

α $= d - \sqrt{d^2 - \frac{2\times[Mu]}{0.85\,fc'\phi b}}$

α = 534.5 mm - $\sqrt{(534.5\text{ mm})^2 - \frac{2\times[263.45\times10^6\text{Nmm}]}{0.85\ (30)(0.9)(350\text{ mm})}}$

α = 65.357 mm

β_1 $= 0.85 - \frac{0.05(fc-28)}{7}$

(From SNI 2847:2019 Ps. 22.2.2.4.3)

β_1 $= 0.85 - \frac{0.05(30-28)}{7}$

β_1 = 0.84

c $= \frac{a}{\beta 1}$

c $= \frac{65.357\ mm}{0.84}$

c = 77.8012 mm

c_{max} = 0.375d

c_{max} = 0.375 (534.5 mm)

c_{max} = 200.4375 mm

See that $c<c_{max}$, therefore the beam is already tension controlled (Φ=0.9).

$$A_s = \frac{Mu}{\Phi fy(d-\frac{a}{2})}$$

$$A_s = \frac{263.45}{0.9\,(420)\,(534.5 - \frac{65.36}{2})}$$

$$A_s = 1388.857 \text{ mm}^2$$

$$n = \frac{(1388.8571\ )}{0.25\pi(25)^2}$$

n = 2.9 **(use 3D25)**

$$A_{s,used} = n(0.25)\,(\pi)\,(ø^2)$$

$$A_{s,used} = 3\,(0.25)\,(\pi)\,(25^2)$$

$$A_{s,used} = 1472.62 \text{ mm}^2$$

Minimum criteria for As,used:

$$A_{s,max} = 0.025bd^2$$

$$A_{s,max} = 0.025(350)(534.5 \text{ mm})^2$$

$$A_{s,max} = 4676.875 \text{ mm}^2$$

$$A_{s,min\ 1} = \frac{\sqrt{fc'}bd}{4fy}$$

$$A_{s,min\ 1} = \frac{\sqrt{30}\,(350)\,(534.5)}{4(420)}$$

$$A_{s,min\ 1} = 609.9188 \text{ mm}^2$$

$$A_{s,min\ 2} = \frac{1.4bd}{fy}$$

$$A_{s,min\ 2} = \frac{1.4\,(350)(534.5)}{(420)}$$

$$A_{s,min\ 2} = 623.58 \text{ mm}^2$$

Since $A_{s,used}$ exceeds both $A_{s,min\ 1}$ and $A_{s,min\ 2}$ but still under the value of $A_{s,max}$, therefore number of reinforcement is already controlled.

s $= \frac{b-2Cc-2\,\text{Ø}_{transversal}-(nD\text{Ø}_{longitudinal})}{n-1}$

s $= \frac{350-2(40)-2\,(13)-(3(25))}{3-1}$

s = 84.5 mm (OK!)

Nominal moment should be checked as follow.

M_n $= A_s \times fy \times \left(d - \frac{\alpha}{2}\right)$

M_n $= 1472.62\ mm^2 \times 420\ MPa \times \left(534.5\ mm - \frac{65.357\ mm}{2}\right)$

M_n = 310,376,823 Nmm

ϕM_n = 0.9 (310,376,823 Nmm)

ϕM_n = 279,339,141 Nmm

($\phi M_n > M_u$ = 263,450,000 Nmm)

Therefore, reinforcement is already designed well to control the moment.

Mid Span (Negative Moment)

$M_{u,used}$ = largest value of (325.69 kNm or $0.25M_{u(-)}$)

$M_{u,used}$ = 352.69 kNm

Assume that it will be designed in one layer.

d $= h - C_c - (0.5\text{Ø}_{longitudinal}) - \text{Ø}_{transversal}$

d = 600 mm – 40 mm – 0.5 (25) mm – 13 mm

d = 534.5 mm

α $= d - \sqrt{d^2 - \frac{2\times[Mu]}{0.85\,fc'\phi b}}$

α $= 534.5\ mm - \sqrt{(534.5\ mm)^2 - \frac{2\times[352.69\times10^6 Nmm]}{0.85\ (30)(0.9)(350\ mm)}}$

α = 82.175 mm

$\beta_1 = 0.85 - \frac{0.05(fc-28)}{7}$

(From SNI 2847:2019 Ps. 22.2.2.4.3)

$\beta_1 = 0.85 - \frac{0.05(30-28)}{7}$

$\beta_1 = 0.84$

$c = \frac{a}{\beta 1}$

$c = \frac{82.175\ mm}{0.84}$

$c = 97.828$ mm

$c_{max} = 0.375d$

$c_{max} = 0.375$ (534.5 mm)

$c_{max} = 200.4375$ mm

See that $c<c_{max}$, therefore the beam is already tension controlled (Φ=0.9).

$A_s = \frac{Mu}{\Phi fy(d-\frac{a}{2})}$

$A_s = \frac{352.69}{0.9\,(420)\,(534.5 - \frac{39.17}{2})}$

$A_s = 1746.235\ mm^2$

$n = \frac{(1746.235)}{0.25\pi(25)^2}$

n = 4 **(use 4D25)**

$A_{s,used} = n(0.25)\,(\pi)\,(ø^2)$

$A_{s,used} = 4\,(0.25)\,(\pi)\,(25^2)$

$A_{s,used} = 1963.5\ mm^2$

Minimum criteria for As,used:

$A_{s,max} = 0.025bd^2$

$A_{s,max}$ = 0.025(350)(534.5 mm)^2

$A_{s,max}$ = 4676.875 mm^2

$A_{s,min\ 1}$ $= \frac{\sqrt{fc'}bd}{4fy}$

$A_{s,min\ 1}$ $= \frac{\sqrt{30}\,(350)\,(534.5)}{4(420)}$

$A_{s,min\ 1}$ = 609.9188 mm^2

$A_{s,min\ 2}$ $= \frac{1.4bd}{fy}$

$A_{s,min\ 2}$ $= \frac{1.4\,(350)(534.5)}{(420)}$

$A_{s,min\ 2}$ = 623.58 mm^2

Since $A_{s,used}$ exceeds both $A_{s,min\ 1}$ and $A_{s,min\ 2}$ but still under the value of $A_{s,max}$, therefore number of reinforcement is already controlled.

s $= \frac{b-2Cc-2\,ø_{transversal}-(nDø_{longitudinal})}{n-1}$

s $= \frac{350-2(40)-2\,(13)-(4(25))}{4-1}$

s = 48 mm

Nominal moment should be checked as follow.

M_n $= A_s \times fy \times \left(d - \frac{\alpha}{2}\right)$

M_n = 1963.5 mm^2 × 420 MPa × $\left(534.5\text{ mm-}\frac{82.175\text{ mm}}{2}\right)$

M_n = 406,901,209 Nmm

ϕMn = 0.9 (406,901,209 Nmm)

ϕMn = 366,211,088 Nmm

(ϕMn > M_u = 325,690,000 Nmm)

Therefore, reinforcement is already designed well to control the moment.

Mid Span (Positive Moment)

$M_{u,used}$ = largest value of (162.02 kNm or $0.25M_{u(-)}$)

$M_{u,used}$ = 162.02 kNm

Assume that it will be designed in one layer.

d = $h - C_c - (0.5Ø_{longitudinal}) - Ø_{transversal}$

d = 600 mm – 40 mm – 0.5 (25) mm – 13 mm

d = 534.5 mm

$$\alpha = d - \sqrt{d^2 - \frac{2\times[Mu]}{0.85\ fc'\phi b}}$$

$$\alpha = 534.5\text{ mm} - \sqrt{(534.5\text{ mm})^2 - \frac{2\times[162.02\times10^6\text{Nmm}]}{0.85\ (30)(0.9)(350\text{ mm})}}$$

α = 39.172 mm

$$\beta_1 = 0.85 - \frac{0.05(fc-28)}{7}$$

(From SNI 2847:2019 Ps. 22.2.2.4.3)

$$\beta_1 = 0.85 - \frac{0.05(30-28)}{7}$$

β_1 = 0.84

$$c = \frac{a}{\beta 1}$$

$$c = \frac{39.172\ mm}{0.84}$$

c = 46.085 mm

c_{max} = 0.375d

c_{max} = 0.375 (534.5 mm)

c_{max} = 200.4375 mm

See that $c<c_{max}$, therefore the beam is already tension controlled (Φ=0.9).

$A_s = \frac{Mu}{\Phi fy(d - \frac{a}{2})}$

$A_s = \frac{162.02}{0.9\,(420)\,(534.5 - \frac{39.17}{2})}$

$A_s = 832.4198\ mm^2$

$n = \frac{(832.4198)}{0.25\pi(25)^2}$

$n = 2$ **(use 2D25)**

$A_{s,used} = n(0.25)\,(\pi)\,(ø^2)$

$A_{s,used} = 2\,(0.25)\,(\pi)\,(25^2)$

$A_{s,used} = 981.748\ mm^2$

Minimum criteria for As,used:

$A_{s,max} = 0.025bd^2$

$A_{s,max} = 0.025(350)(534.5\ mm)^2$

$A_{s,max} = 4676.875\ mm^2$

$A_{s,min\,1} = \frac{\sqrt{fc'}bd}{4fy}$

$A_{s,min\,1} = \frac{\sqrt{30}\,(350)\,(534.5)}{4(420)}$

$A_{s,min\,1} = 609.9188\ mm^2$

$A_{s,min\,2} = \frac{1.4bd}{fy}$

$A_{s,min\,2} = \frac{1.4\,(350)(534.5)}{(420)}$

$A_{s,min\,2} = 623.58\ mm^2$

Since $A_{s,used}$ exceeds both $A_{s,min\,1}$ and $A_{s,min\,2}$ but still under the value of $A_{s,max}$, therefore number of reinforcement is already controlled.

$s = \frac{b - 2Cc - 2\,ø_{transversal} - (nDø_{longitudinal})}{2-1}$

s $= \frac{350-2(40)-2\,(13)-(4(25))}{2-1}$

s = 194 mm

Nominal moment should be checked as follow.

$M_n = A_s \times fy \times \left(d - \frac{\alpha}{2}\right)$

$M_n = 981.748\ \text{mm}^2 \times 420\ \text{MPa} \times \left(534.5\ \text{mm} - \frac{39.172\ \text{mm}}{2}\right)$

M_n = 212,316,424 Nmm

ϕM_n = 0.9 (212,316,424 Nmm)

ϕM_n = 191,084,781 Nmm

($\phi M_n > M_u$ = 162,020,000 Nmm)

Therefore, reinforcement is already designed well to control the moment.

It needs to be noticed that reinforcements will be checked against the torsional moment (*see the final results on torsional reinforcement*).

2.8.2 Transversal reinforcement

During the calculation of transversal reinforcement, plasticity hinge will be assumed to be built in the joint of beams. Therefore, it will need to calculate the M_{pr}.

$A_{s,\ support\ (-)}$ = 3436.12 mm^2

$a_{support} = \frac{A_{s,field\ (-)} \times fy}{0.85 \times fc \times b}$

$a_{support} = \frac{3436.12 \times 420}{0.85 \times 30 \times 350}$

$a_{support}$ = 161.7 mm

$A_{s,\ support\ (-)}$ = 2454.37 mm^2

$a_{support} = \frac{A_{s,field\ (-)} \times fy}{0.85 \times fc \times b}$

$a_{support} = \frac{2454.37 \times 420}{0.85 \times 30 \times 350}$

$a_{support} = 115.5$ mm

$a_{pr}^{-} = 1.25\ (a_{support\ (-)})$

$a_{pr}^{-} = 1.25\ (161.7)$

$a_{pr}^{-} = 202.125$ mm

$a_{pr}^{+} = 1.25\ (a_{support\ (+)})$

$a_{pr}^{+} = 1.25\ (115.5)$

$a_{pr}^{+} = 144.374$ mm

$M_{pr}^{-} = 1.25 \times A_{s,support(-)} \times fy \times \left(d - \frac{a_{pr}^{-}}{2}\right)$

$M_{pr}^{-} = 1.25 \times 3436.117 \times 420 \times \left(487.5 - \frac{202.125}{2}\right)$

$M_{pr}^{-} = 691{,}706{,}878$ Nmm

$M_{pr}^{+} = 1.25 \times A_{s,support(+)} \times fy \times \left(d - \frac{a_{pr}^{+}}{2}\right)$

$M_{pr}^{+} = 1.25 \times 2454.37 \times 420 \times \left(534.5 - \frac{144.374}{2}\right)$

$M_{pr}^{+} = 531{,}282{,}959$ Nmm

Table 2.28 shows the distribution of the dead load from slab near the beam.

Table 2.28 Load Distribution

Component	Load		
Slab thickness (125 mm)	=	3	kN
Sand (5 cm; 16 kN/m)	=	0.8	kN
Mortar (2 cm; 21 kN/m)	=	0.42	kN
Floor finishing (1 cm; 24 kN/m)	=	0.24	kN
Plafond	=	0.18	kN
Partition	=	1	kN
MEP	=	0.25	kN

DL = 5.89 kN

Beam's Load = 11.78 kN

Distance of each beam = 4 m

$V_{e,1}$ $= \frac{M_{pr}{}^{+} + M_{pr}{}^{-}}{L}$

$V_{e,1}$ $= \frac{691{,}706{,}878 + 531{,}282{,}959}{8000}$

$V_{e,1}$ = 152,873 N = 152.873 kN

Calculation of the shear force due to gravitational load can be seen in Table 2.29.

Table 2.29 Shear Force Distribution

1. Dead load			(DL)
	Load of Beam	11.78	kN
	Slab	35.34	kN
	Self-weight	15.96	kN
	VDL	63.08	kN
2. Live Load			(LL)
	Loads on beam	3.59	kN
	VLL	10.77	kN

V_g = 1.2 DL + 1.6 LL = 86.466 kN

V_e = $V_{e,1} + V_g$

V_e = 152.873 kN + 86.466 kN

V_e = 239.34 kN

Because V_e from calculation is smaller than the output from MIDAS Gen, $V_{e,used}$ that will be considered is the output from MIDAS Gen, $V_{e,used}$ = 411.17 kN.

<u>Support area</u>

Assume 2D13 will be installed.

V_c $= 0.17 \times \lambda \times \sqrt{fc} \times b \times d$

V_c $= 0.17 \times 1 \times \sqrt{30} \times 350\ mm \times 484.5\ mm$

V_c = 157.89 kN

$V_s = \frac{V_{e,used}}{\phi} - V_c$

$V_s = \frac{411.17}{0.75} - 157.89\ kN$

V_s = 390.33 kN

$V_{s,max} = \frac{2}{3}\sqrt{fc} \times b \times d$

$V_{s,max} = \frac{2}{3}\sqrt{30} \times 350\ mm \times 484.5\ mm$

$V_{s,max}$ = 613.008 kN (OK!)

Calculation of spacing

s $= \frac{n \times 0.25 \times \pi \times D^2 \times Fyt \times d}{V_s}$

s $= \frac{2 \times 0.25 \times \pi \times 13^2 \times 420 \times 484.5}{390.33}$

s = 138.394 mm

$s_{max,1}$ = 0.25d

$s_{max,1}$ = 0.25(484.5 mm)

$s_{max,1}$ = 121.125 mm

$s_{max,2}$ = 150 mm

$s_{max,3}$ = 6D

$s_{max,3}$ = 6(25)

$s_{max,3}$ = 150 mm

s_{used} = 100 mm (use 2D13-100)

Reinforcement capacity check

$V_{s,used} = \frac{n \times 0.25 \times \pi \times D^2 \times Fyt \times d}{s_{used}}$

$V_{s,used} = \frac{2 \times 0.25 \times \pi \times 13^2 \times 420 \times 484.5}{100}$

$V_{s,used}$ = 540.194 kN

V_n $= V_{s,used} + V_c$

V_n $= 540.194$ kN $+ 157.89$ kN

V_n $= 698.089$ kN

$\phi Vn/Vu$ $= \frac{0.75 \times 698.089}{411.17}$

$\phi Vn/Vu$ $= 1.273$

Since the capacity is more than one, then reinforcement has been installed properly.

<u>Field area</u>

Assume 2D13 will be installed.

Maximum shear stress at the plastic hinge joint needs to be calculated first.

V_{max} $= \frac{V_{support} \times 0.5 \times (L-2h)}{0.5 \times L}$

V_{max} $= \frac{411.17 \times 0.5 \times (8-2(0.6))}{0.5 \times 8}$

V_{max} $= 287.819$ kN

Since V_{max} is smaller than MIDAS Gen output for field area, then MIDAS Gen output will be used = 407.25 kN

V_c $= 0.17 \times \lambda \times \sqrt{fc} \times b \times d$

V_c $= 0.17 \times 1 \times \sqrt{30} \times 350\ mm \times 484.5\ mm$

V_c $= 157.89$ kN

V_s $= \frac{V_{e,used}}{\phi} - V_c$

V_s $= \frac{407.25}{0.75} - 157.89\ kN$

V_s $= 385.11$ kN

$V_{s,max}$ $= \frac{2}{3}\sqrt{fc} \times b \times d$

$V_{s,max}$ $= \frac{2}{3}\sqrt{30} \times 350\ mm \times 484.5\ mm$

$V_{s,max}$ = 613.008 kN (OK!)

Calculation of spacing

s $= \frac{n \times 0.25 \times \pi \times D^2 \times Fyt \times d}{V_s}$

s $= \frac{2 \times 0.25 \times \pi \times 13^2 \times 420 \times 484.5}{385.11}$

s = 140.272 mm

s_{max} = 0.5d

s_{max} = 0.5(484.5)

s_{max} = 242.25 mm

s_{used} = 100 mm (use 2D13-100)

Reinforcement capacity check

$V_{s,used}$ $= \frac{n \times 0.25 \times \pi \times D^2 \times Fyt \times d}{s_{used}}$

$V_{s,used}$ $= \frac{2 \times 0.25 \times \pi \times 13^2 \times 420 \times 484.5}{100}$

$V_{s,used}$ = 540.194 kN

V_n = $V_{s,used}$ + V_c

V_n = 540.194 kN + 157.89 kN

V_n = 698.089 kN

ϕVn/Vu $= \frac{0.75 \times 698.089}{407.25}$

ϕVn/Vu = 1.285

Since the capacity is more than one, then reinforcement has been installed properly.

2.8.3 Torsional reinforcement

From structural analysis by MIDAS Gen, it is obtained that the torsional moment occurred into beam is:

Tu = 61.04 kNm

Beam will be design with torsional reinforcement with diameter (d_{bt} = 13 mm).

Check whether torsional effect can be neglected.

A_{cp} $= b \times h$

= 350 x 600

= 210,000 mm^2

P_{cp} $= 2 \times (b + h)$

= 2 × (350+600)

= 1,900 m

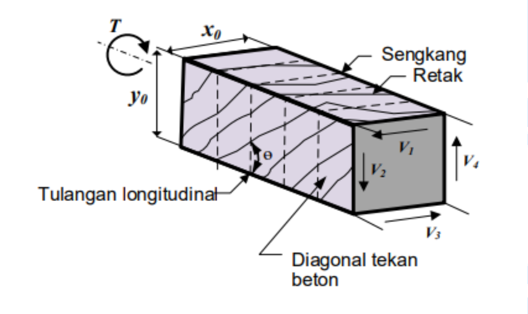


Figure 2.23 Space Truss Analogy

According to SNI 2847:2019, for normal concrete, Torsional can be neglected if:

T_{cr} $= 0.33 \times 1 \times \sqrt{fc'} \times \frac{{A_{cp}}^2}{P_{cp}}$

T_{cr} $= 0.33 \times 1 \times \sqrt{30} \times \frac{210{,}000^2}{1{,}900}$

T_{cr} = 41,952,665 Nmm

$0.25T_{cr}$ = 10,488,166.29 Nmm

Observing that T_u = 83,66 kNm ≈ 83,660,000 Nmm still exceeds the value of $0.25T_{cr}$ = 10,488,166.29 Nmm, then in this case, torsional reinforcement should be installed. Before designing the reinforcement, sectional properties of a beam should be check whether dimension of beam is enough to hold the torsion.

<u>Sectional Properties</u>

x_0 $= b - (2 \times cc) - Dstirrups$

$= 350 - (2 \times 40) - 13$

= 257 mm

y_0 $= h - (2 \times cc) - Dstirrups$

$= 600 - (2 \times 40) - 13$

= 507 mm

A_{0h} $= x_0 \times y_0$

= 257 × 507

= 130,299 mm^2

Bruto Area (A_0) that surrounded by shear force path (SNI 2847:2019 ps 22.7.6.1.1)

= 0.85 A_0

= 0.85 (130,299 mm^2)

= 104,239.2 mm^2

Perimeter due to center line of torsional reinforcement

P_h $= 2 \times (x_0 + y_0)$

$= 2 \times (257 + 507)$

= 1528 mm

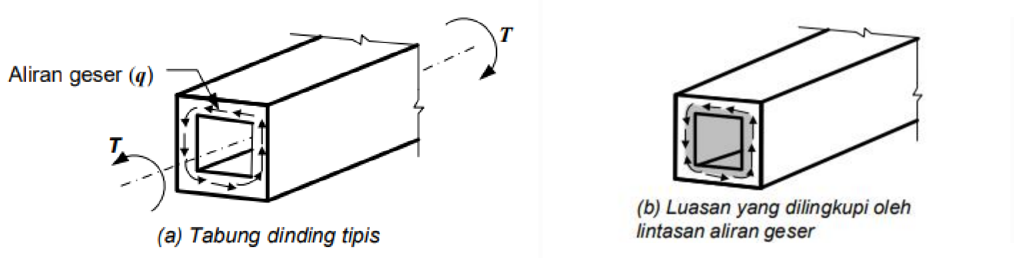


Figure 2.24 Analogy for shell-thin beam (a) and Bruto area (A_0) that surrounded by shear force path

Dimensional check

Tu	= 83,66 kNm ≈ 83,660,000 Nmm
φTcr	= 0.75 × Tu
	= 0.75 (41,952,665 Nmm)
	= 31,464,499 Nmm
Tu, used	= 31,464,499 Nmm
Vu, field	= 411,170 N
Vc	$= 0.17 \times \lambda \times \sqrt{fc'} \times b \times d$
	$= 0.17 \times 1 \times \sqrt{30} \times 350 \times 484.5$
	= 157,896.0896 N

Dimension control (SNI 2847:2019 ps 22.7.7.1)

$$\sqrt{\left(\frac{Vu,field}{h \times d}\right)^2 + \left(\frac{Tu,used}{1.7 \times A0h^2}\right)^2} = \sqrt{\left(\frac{146850}{700 \times 639}\right)^2 + \left(\frac{45780000}{1.7 \times 185428^2}\right)^2}$$

$$= 2.941 < 3.4095$$

From dimension control result, it can be concluded that the dimension of beam is enough to hold torsion moment.

Another parameter:

Yield strength of torsional reinforcement (f_y/f_{yt}) = 1 (non-prestressed concrete).

θ = 45 (SNI 2847:2019 ps. 22.7.6.1.2 for non-prestressed concrete).

Calculate the torsion for transversal.

n (support) = 2

n (field) = 2

s (support) = 100 mm

s (field) = 100 mm

According to SNI 2847:2019 ps. 9.7.6.3.3, spacing for transversal torsion reinforcement can't exceed:

s, max 1 $= \frac{Ph}{8}$

$= \frac{1528}{8}$

= 191 mm

s, max 2 = 300 mm

$\frac{Av+t}{s}(used, field) = \frac{n \times 0.25 \times \pi \times d^2}{s}$

$= \frac{2 \times 0.25 \times \pi \times 13^2}{100}$

= 2.6546 mm^2/mm

$\frac{Av+t}{s}(used, support) = \frac{n \times 0.25 \times \pi \times d^2}{s}$

$= \frac{2 \times 0.25 \times \pi \times 13^2}{100}$

= 2.6546 mm^2/mm

$\frac{At}{s} = \frac{Tu}{2 \times A_0 \times f_{yt}} \times \cot(45^0)$

$\frac{At}{s} = \frac{31,464,499}{2 \times 130,299 \times 420} \times \cot(45^0)$

= 0.3593 mm^2/mm (used 1 foot of stirrup)

Calculate the Area of Stirrup:

$A_{v/s}$ (support) $= \frac{\left(\frac{Vu, used}{0.75}\right) - Vc}{f_{yt} d}$

$$= \frac{\left(\frac{411170 \text{ N}}{0.75}\right) - 157{,}896.0896 \text{ N}}{420 \text{ MPa} \times 484.5 \text{ mm}}$$

= 1.9182 mm^2/mm

$A_{v/s}$ (field) $= \frac{\left(\frac{\text{Vu, used}}{0.75}\right)\text{-Vc}}{f_{yt}\text{d}}$

$$= \frac{\left(\frac{407250 \text{ N}}{0.75}\right) - 157{,}896.0896 \text{ N}}{420 \text{ MPa} \times 484.5 \text{ mm}}$$

= 1.8925 mm^2/mm

$\frac{Av+t}{s}(need, support)$ $= A_{v/s}$ (support) + 2× $\frac{\text{At}}{\text{s}}$

= 1.9182 mm^2/mm + 2(0.3593 mm^2/mm)

= 2.6369 mm^2/mm

$\frac{Av+t}{s}(need, field)$ $= A_{v/s}$ (field) + 2× $\frac{\text{At}}{\text{s}}$

= 1.8925 mm^2/mm + 2(0.3593 mm^2/mm)

= 2.6112 mm^2/mm

According to SNI 2847:2019 ps. 9.6.4.2, minimum of transversal reinforcement, if it's needed to be installed, should be more than:

$\frac{Av+t}{s}(min, 1)$ $= 0.062 \times \sqrt{fc'} \times \frac{b}{fy}$

$= 0.062 \times \sqrt{30} \times \frac{350}{420}$

= 0.2828

$\frac{Av+t}{s}(min, 1)$ $= 0.35 \times \frac{b}{fyt}$

$= 0.35 \times \frac{400}{420}$

= 0.3

Observing the result, (Av+t)/s, used for field = 2.6546 mm^2/mm and (Av+t)/s, used for support = 2.6546 mm^2/mm exceeds the minimum transveral reinforcement reinforcement (0.2829 and 0.3) and Av+t/s, need (2.6369 mm^2/mm for support and 2.6112 mm^2/mm for field). Therefore, all requirements for transversal torsion reinforcement has been fulfilled.

Longitudinal Torsion Reinforcement

For longitudinal torsion reinforcement, it will be designed that diameter of the reinforcement is 13 mm.

Additional longitudinal reinforcement needed to hold torsion:

$$Al = \frac{At}{s} \times Ph \times \frac{fy}{fyt} \times \cot^2(45^0)$$

$$= 0.3593 \times 1528 \times 1 \times \cot^2(45^0)$$

$$= 549.08 \text{ mm}^2$$

$$Al, min = \left(0.42 \times \sqrt{fc'} \times \frac{Acp}{fy}\right) - \left(\frac{At}{s} \times Ph \times \frac{fy}{fyt}\right)$$

$$= \left(0.42 \times \sqrt{30} \times \frac{210000}{420}\right) - (0.3539 \times 1528 \times 1)$$

$$= 601.14 \text{ mm}^2$$

$$Al, used = 601.14 \text{ mm}^2$$

In longitudinal reinforcement design for beam (see Chapter 2.8.1), it is obtained that reinforcement that will be installed is as follow.

Left end (Negative Moment) : 7D25

Left end (Positive Moment) : 3D25

Mid span (Negative Moment) : 4D25

Mid span (Positive Moment) : 2D25

During torsional reinforcement design, especially for longitudinal torsional reinforcement, sometimes it is needed to add more reinforcements from the result above to help holding the torsional moment. After checking with the torsional longitudinal requirements (see explanations below), longitudinal reinforcements that will be installed are as follow:

Left end (Negative Moment) : 7D25

Left end (Positive Moment) : 5D25

Mid span (Negative Moment) : 4D25

Mid span (Positive Moment) : 3D25

As, need support (upper) $= n \times 0.25 \times \pi \times d^2$

$= 7\ (0.25)\ (\pi)\ (25^2)$

$= 3072.0488\ mm^2$

As, need support (bottom) $= n \times 0.25 \times \pi \times d^2$

$= 5\ (0.25)\ (\pi)\ (25^2)$

$= 1388.8571\ mm^2$

As, need field (upper) $= n \times 0.25 \times \pi \times d^2$

$= 4\ (0.25)\ (\pi)\ (25^2)$

$= 1746.2355\ mm^2$

As, need field (bottom) $= n \times 0.25 \times \pi \times d^2$

$= 3\ (0.25)\ (\pi)\ (25^2)$

$= 832.4198\ mm^2$

As + Al need

As + Al need, support =Al, used+As, need support(u)

+As, need support(b)

$= 601.14\ mm^2 + 3072.0488\ mm^2 + 1388.8571\ mm^2$

$= 5062.0455\ mm^2$

As + Al need,field =Al, used +As, need field(u)

+As, need field (b)

$= 601.14\ mm^2 + 1746.2355\ mm^2 + 832.4198\ mm^2$

$= 3179.7950\ mm^2$

Calculation of As+Al used:

n, longitudinal reinforcement (upper, support) = 7

n, longitudinal reinforcement (lower, support) = 5

n, longitudinal reinforcement (middle, support) = 2

n, longitudinal reinforcement (vertical, support) = 2 + $n_{middle}/2$

= 3

Horizontal space $= \frac{b-2Cc-2ds-dlongitudinal}{\min(n,upper\ ;n,lower)-1}$

$= \frac{350-2(40)-2(13)-25}{5-1}$ mm

= 54.75 mm

Vertical space $= \frac{h-2Cc-2ds-dlongitudinal}{n,vertical-1}$

$= \frac{600-2(40)-2(13)-25}{3-1}$ mm

= 234.5 mm

n, longitudinal reinforcement (upper, field) = 4

n, longitudinal reinforcement (lower, field) = 3

n, longitudinal reinforcement (middle, field) = 2

n, longitudinal reinforcement (vertical, field) = 2 + $n_{middle}/2$

= 3

Horizontal space $= \frac{b-2Cc-2ds-dlongitudinal}{\min(n,upper\ ;n,lower)-1}$

$= \frac{350-2(40)-2(13)-25}{3-1}$ mm

= 109.50 mm

Vertical space $= \frac{h-2Cc-2ds-dlongitudinal}{n,vertical-1}$

$= \frac{600-2(40)-2(13)-25}{3-1}$ mm

= 234.5 mm

Horizontal and vertical spacing result show that spacing of the torsional reinforcement is still less than 300 mm. Therefore, the spacing has been suitable with SNI 2847:2019 ps. 11.5.6.2.

As+Al, used for support $= [(7+5)(0.25)(\pi)(25^2)]$

$+[(2)(0.25)(\pi)(13^2)]$

$= 6155.950 \text{ mm}^2$

As+Al, used for field $= [(4+3)(0.25)(\pi)(25^2)]$

$+[(2)(0.25)(\pi)(13^2)]$

$= 3701.58 \text{ mm}^2$

Based on the calculation before, As+Al, need for support is obtained to be = 5062.0455 mm^2 and As+Al, need for field is obtained to be = 3701.58 mm^2.

Since As+Al, used for support is 6155.950 mm^2 and As+Al, used for field is 3179.795 mm^2, it can be concluded that longitudinal reinforcement has been installed as it is required, since the installed reinforcement exceeds the required reinforcement.

2.9 The Design of Column

Column is a vertical element of a structure that receives a combination of bending and axial load. Column will transfer any loads from structure itself (self-weight, live load, self-imposed dead load, or other loads calculated during structural analysis) from floor slab, beam, and column to the lower-column, until it arrives the foundation to transfer loads to the soil. Therefore, total loads received by a column is total of load from upper floor and load from the level of the column itself. From those statement, it can be concluded that 1st floor column will receive bigger loads than upper-level of column. Therefore, internal forces of a column will choose maximum load from 1st floor. Table 2.30 Shows axial forces and bending moment resulted from structural analysis done by MIDAS Gen for K1(600X600)

Table 2.30 Axial Forces-Bending Moment Resulted for K1 (600X600)

Condition	P (kN)	M2 (kNm)	M3 (kNm)
P max	369.100	501.780	474.610
P min	3378.230	-305.700	-412.500
M2 Max	2001.430	651.710	-555.940

M2 Min	1220.830	-652.980	-570.890
M3 Max	1306.030	-483.330	669.000
M3 Min	988.030	-338.910	-615.150

2.9.1 Longitudinal Reinforcement

Data

L = 4000 mm

b = 600 mm

h = 600 mm

$d_{longitudinal}$ = 25 mm

$d_{stirrup}$ = 13 mm

c_c = 40 mm

fc' = 30 MPa

f_y = 420 MPa

f_{yt} = 420 MPa

h_{beam} = 600 mm

L_n = 3400 mm

Check the Axial forces.

According to SNI 2847:2019, ρ is limited from 0.01 and can't be exceed than 0.06.

Number of reinforcements is designed as follows.

n = 28

$$\rho = n \times 0.25 \times \pi \times \frac{{d_{longitudinal}}^2}{b \times h}$$

$$\rho = n \times 0.25 \times \pi \times \frac{25^2}{600 \times 600}$$

ρ = 3.82% (OK)

Table 2.24 shows the result of longitudinal reinforcement analysis helped with SPColumn program, while Figure 2.31 shows the interaction diagram resulted from the program.

Table 2.31 SP-Column Analysis Result

No	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕM_{nx} kNm	ϕM_{ny} kNm	$\phi M_n/M_u$	NA Depth Mm	d_t Depth mm	ε_t	ϕ
1	369,1	-305,7	-412,5	-586,78	-791,77	1,919	334	750	0,0037	0,791
2	3378,23	501,78	474,61	595,6	563,35	1,187	489	758	0,0017	0,65
3	2001,43	651,71	-555,94	683,07	-582,69	1,048	419	758	0,0024	0,677
4	1220,83	-652,98	-570,89	-712,86	-623,25	1,092	379	758	0,003	0,727
5	1306,03	-483,33	669	-558,83	773,51	1,156	377	750	0,003	0,724
6	988,03	-338,91	-615,15	-481,1	-873,23	1,42	351	734	0,0033	0,752

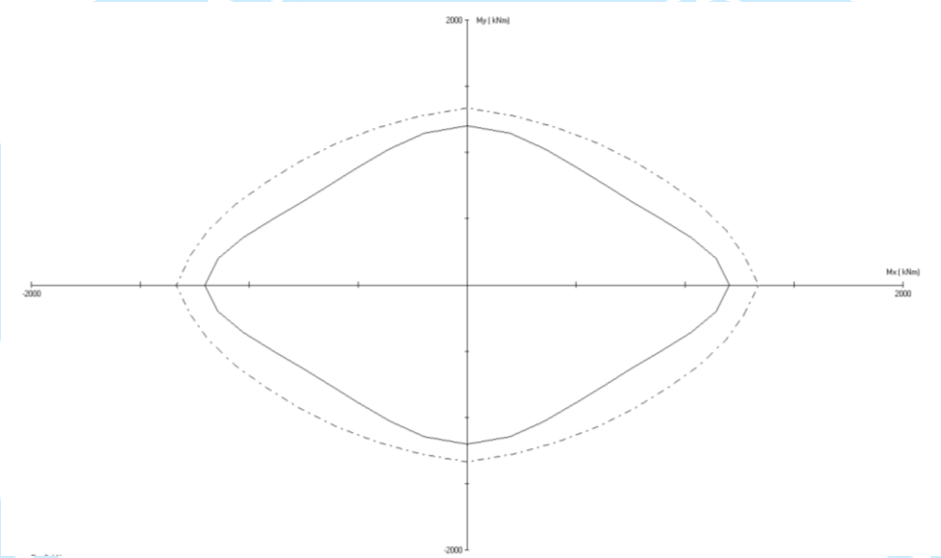


Figure 2.25 Interaction Diagram of a column

<u>Check the Strong Column-Weak Beam</u>

To determine the nominal moment calculated for strong column-weak beam, the program should be reanalyzed with multiplication factor of 1.25 for reinforcing yield strength (f_y). As a result, Table 2.32 and Table 2.22 shows the nominal moment resulted from the SPColumn program.

Table 2.32 SP-Column Analysis Result (Reinforced Yeild Strength Factor)

No	P_u	M_{ux}	M_{uy}	ϕM_{nx}	ϕM_{ny}	$\phi M_n/M_u$	NA Depth	d_t Depth	ε_t	ϕ
	kN	kNm	kNm	kNm	kNm		Mm	mm		

1	369,1	-305,7	-412,5	-602	- 812,26	1,969	345	750	0,00353	0,745
2	3378,2	501,78	474,61	604,2	571,49	1,204	487	758	0,00167	0,65
3	2001,4	651,71	-555,94	663,65	- 566,13	1,018	423	758	0,00237	0,65
4	1220,8	-652,98	-570,89	-693	- 605,86	1,061	387	758	0,00287	0,676
5	1306	-483,33	669	-541,6	749,71	1,121	385	750	0,00284	0,672
6	988,03	-338,91	-615,15	-479	- 869,46	1,413	359	733	0,00313	0,703

Table 2.33 SP-Column Analysis Result (Mnx and Mny)

No	**P_u** kN	**M_{ux}** kNm	**M_{uy}** kNm	**ϕM_{nx}** kNm	**ϕM_{ny}** kNm	**ϕ**	Mn_x	Mn_y
1	369,1	-305,7	-412,5	-586,78	-791,77	0,791	741,82	1000,97
2	3378,23	501,78	474,61	595,6	563,35	0,65	916,31	866,69
3	2001,43	651,71	-555,94	683,07	-582,69	0,677	1008,97	860,69
4	1220,83	-652,98	-570,89	-712,86	-623,25	0,727	980,55	857,29
5	1306,03	-483,33	669	-558,83	773,51	0,724	771,86	1068,38
6	988,03	-338,91	-615,15	-481,1	-873,23	0,752	639,76	1161,21

From Table 2.33, it can be seen all nominal moments calculated for a column. According to NEHRP 16-917-40, nominal moment for strong column-weak beam, it can be taken nominal moment from minimum and maximum of axial forces. Therefore, Mn_c for the calculation can be taken 741.92 kNm.

Taking out nominal moment resulted from beam (see Chapter 2.8), then data below can be taken.

Mnc = 741.820 kNm

Mn (-) field Beam = 166.153 kNm

Mn (+) field Beam = 159.101 kNm

According to SNI 2847:2019 ps.18.7.3.2, for strong column-weak beam, these requirements should be fulfilled.

$2Mn_c \geq 1.2\ (Mn^- + Mn^+)$

$2(741.820\ kNm) \geq 1.2(594.898\ kNm + 517.295\ kNm)$

$1483.64\ kNm \geq 1334.63\ kNm$ (OK!)

Therefore, strong column-weak beam is fulfilled.

2.9.2 Transversal Reinforcement

Length of Plasticity Hinge Zone

L_{01} = 600 mm (equals to h)

L_{02} $= \frac{Ln}{6} = \frac{3400}{6} = 566.667$ mm

L_{03} = 450 mm (SNI 2847:2019 ps.18.7.5.1.)

L_0 taken = 600 mm

Transversal Reinforcement of Support Zone (Plastic Hinge)

For initial assumption, design the reinforcement as follow.

n_1 = 3

n_2 = 3

s = 100 mm

$X_{i,\,max}$ = 300 mm

$A_{sh,1}$ $= n_1 \times 0.25 \times \pi \times ds^2$

$= 3 \times 0.25 \times \pi \times 13^2$

= 398.197 mm²

$A_{sh,2}$ $= n_2 \times 0.25 \times \pi \times ds^2$

$= 3 \times 0.25 \times \pi \times 13^2$

= 398.197 mm²

$A_{sh/s,1}$ $= \frac{Ash,1}{s}$

$= \frac{398.197}{100}$

= 3.982 mm²/mm

$A_{sh/s,2}$ $= \frac{Ash,2}{s}$

$= \frac{398.197}{100}$

= 3.982 mm²/mm

Confinement (Plastic Hinge)

b_c $= b - (2 \times cc)$

$= 600 - 2(40)$

$= 520$ mm

h_c $= h - (2 \times cc)$

$= 600 - 2(40)$

$= 520$ mm

A_g $= b \times h$

$= 600 \times 600$

$= 360000$ mm²

A_{ch} $= b_c \times h_c$

$= 520 \times 520$

$= 270400$ mm²

Weak Axis Side Calculation (SNI 2847:2019 ps. 18.7.5.4.):

$A_{sh/s}$ min,1 $= 0.3 \times \left(\frac{b_c \times fc'}{fyt}\right) \times \left(\frac{A_g}{A_{ch}} - 1\right)$

$= 0.3 \times \left(\frac{520 \times 30}{420}\right) \times \left(\frac{360000}{270400} - 1\right)$

$= 3.692$ mm²

$A_{sh/s}$ min,2 $= 0.09 \times b_c \times \frac{fc'}{fyt}$

$= 0.09 \times 520 \times \frac{30}{420}$

$= 3.43$ mm²

With those minimum requirements, $A_{sh/s,1}$ = 3.982 mm²/mm still exceeds both $A_{sh/s}$ min,1 = 3.692 mm² and $A_{sh/s}$ min,2 = 3.43 mm². Therefore, the reinforcement's section is fulfilled the requirement.

Strong Axis Side Calculation (SNI 2847:2019 ps. 18.7.5.4.):

$A_{sh/s}$ min,1 $= 0.3 \times \left(\frac{h_c \times fc'}{fyt}\right) \times \left(\frac{A_g}{A_{ch}} - 1\right)$

$= 0.3 \times \left(\frac{520 \times 30}{420}\right) \times \left(\frac{360000}{270400} - 1\right)$

$= 3.692$ mm²

$A_{sh/s\ min,2}$ $= 0.09 \times h_c \times \frac{fc'}{fyt}$

$= 0.09 \times 520 \times \frac{30}{420}$

$= 3.43$ mm²

With those minimum requirements, $A_{sh/s},2$ = 3.982 mm²/mm still exceeds both $A_{sh/s}$ min,1 = 3.692 mm² and $A_{sh/s}$ min,2 = 3.43 mm². Therefore, the reinforcement's section is fulfilled the requirement.

Spacing Check (SNI 2847:2019 ps. 18.7.5.3.):

$s_{max,1}$ $= \frac{b}{4}$

$= \frac{600}{4}$

= 150 mm

$s_{max,2}$ $= 6 \times db$

$= 6 \times 25$

= 150 mm

$s_{max,3}$ = Maximum value of [100 mm; minimum value of (150 mm; 100 + $\frac{350 - X_{i,max}}{3}$)]

= Maximum value of [100 mm; minimum value of (150 mm; 100 + $\frac{350 - 300}{3}$)]

= Maximum value of [100 mm; minimum value of (150 mm; 116.67 mm)]

= Maximum value of [100 mm; 116.67 mm)]

= 116.67 mm

Since spacing installed = 100 mm is still under $s_{max,1}$, $s_{max,2}$, and $s_{max,3}$, then it can be said that the spacing requirements is fulfilled.

Shear Strength of Plastic Hinge Zone

M_{pr} of a column can be taken from Table ….. Probable moment of a column can use the largest value of nominal moment.

M_{pr} = 1161.210 kNm

$V_{u,1} = 2 \times \frac{Mpr}{Ln}$

$= 2 \times \frac{1161.210 \times 10^6\ Nmm}{3400\ mm}$

$= 683{,}065$ N

From structural analysis by MIDAS Gen, it can be obtained that:

$V_{u,2}$ for weak axis = 305800 N

$V_{u,2}$ for strong axis = 263333 N

Calculation of the shear of weak axis:

Shear force that will be designed (V_u) is the largest value from $V_{u.1}$ and $V_{u,2}$. During the calculation, it also needs smallest compression forces (N_u) from structural analysis by MIDAS Gen, as it equals to 369.1 kN

Vu = 683,065 N

Φ = 0,75 (SNI 2847:2019 Table 21.2.1.)

$$V_c = 0.17 \times \frac{(1+Nu) \times 10^3}{(14 \times b \times h)} \times \sqrt{fc'} \times h \times \left(b - cc - ds - \frac{dlongitudinal}{2}\right)$$

$$= 0.17 \times \frac{(1+369.1) \times 10^3}{(14 \times 600 \times 600)} \times \sqrt{30} \times 600 \times \left(600 - 40 - 13 - \frac{25}{2}\right)$$

= 320,482 N

$$V_{s,need} = \frac{Vu}{\Phi} - \mathrm{Vc}$$

$$= \frac{683{,}065\ \mathrm{N}}{0{,}75} - 320{,}482\ \mathrm{N}$$

= 590,272 N

$$A_{s/s\ need} = \frac{Vs,need}{fyt \times \left(b - cc - ds - \frac{db}{2}\right)}$$

$$= \frac{590{,}272\ \mathrm{N}}{420 \times \left(600 - 40 - 13 - \frac{25}{2}\right)}$$

= 2.6294 mm²/mm

$A_{s/s\ Min\ 1}$ $= 0.062 \times \sqrt{fc'} \times \frac{b}{fyt}$

$= 0.062 \times \sqrt{30} \times \frac{600}{420}$

$= 0.4851$ mm²/mm

$A_{s/s\ Min\ 2}$ $= 0.35 \times \frac{b}{fyt}$

$= 0.35 \times \frac{600}{420}$

$= 0.5$ mm²/mm

With those minimum requirements, $A_{sh/s}$,1 = 3.982 mm²/mm still exceeds both $A_{s/s\ need}$ = 2.6294 mm²/mm, $A_{s/s}$ min,1 = 0.4851mm²/mm, and $A_{s/s}$ min,2 = 0.4851 mm²/mm. Therefore, the reinforcement's area per spacing is fulfilled the requirement.

Calculation of the shear of strong axis:

Shear force that will be designed (V_u) is the largest value from $V_{u.1}$ and $V_{u,2}$. During the calculation, it also needs smallest compression forces (N_u) from structural analysis by MIDAS Gen, as it equals to 369.1 kN.

Vu = 683,065 N

Φ = 0,75 (SNI 2847:2019 Table 21.2.1.)

V_c $= 0.17 \times \frac{(1+Nu)\times 10^3}{(14\times b\times h)} \times \sqrt{fc'} \times h \times \left(b - cc - ds - \frac{dlongitudinal}{2}\right)$

$= 0.17 \times \frac{(1+369.1)\times 10^3}{(14\times 600\times 600)} \times \sqrt{30} \times 600 \times \left(600 - 40 - 13 - \frac{25}{2}\right)$

$= 320,482$ N

$V_{s,need}$ $= \frac{Vu}{\Phi} - \mathrm{Vc}$

$= \frac{683{,}065\ \mathrm{N}}{0{,}75} - 320{,}482\ \mathrm{N}$

$= 590,272$ N

$A_{s/s\ need}$ $= \frac{Vs,need}{fyt \times \left(b - cc - ds - \frac{db}{2}\right)}$

$= \frac{590,272\ \text{N}}{420 \times \left(600 - 40 - 13 - \frac{25}{2}\right)}$

$= 2.6294\ \text{mm}^2/\text{mm}$

$A_{s/s\ Min\ 1}$ $= 0.062 \times \sqrt{fc'} \times \frac{b}{fyt}$

$= 0.062 \times \sqrt{30} \times \frac{600}{420}$

$= 0.4851\ \text{mm}^2/\text{mm}$

$A_{s/s\ Min\ 2}$ $= 0.35 \times \frac{b}{fyt}$

$= 0.35 \times \frac{600}{420}$

$= 0.5\ \text{mm}^2/\text{mm}$

With those minimum requirements, $A_{sh/s},2$ = 3.982 mm²/mm still exceeds both $A_{s/s\ need}$ = 2.6294 mm²/mm, $A_{s/s}$ min,1 = 0.4851mm²/mm, and $A_{s/s}$ min,2 = 0.4851 mm²/mm. Therefore, the reinforcement's area per spacing is fulfilled the requirement.

<u>Transversal Reinforcement Outside of Support Zone (Plastic Hinge)</u>

n_1 = 2

n_2 = 2

s = 150 mm

$A_{v,\ weak\ axis}$ $= n_1 \times \pi \times 0.25 \times ds^2$

$= 2 \times \pi \times 0.25 \times 13^2$

= 265.465 N

$A_{v,\ strong\ axis}$ $= n_2 \times \pi \times 0.25 \times ds^2$

$= 2 \times \pi \times 0.25 \times 13^2$

= 265.465 N

<u>Confinement Reinforcement Outside Support Zone (Plastic Hinge)</u>

$S_{max,1}$ $= 6 \times db$

$= 6 \times 25$

= 150 mm

$S_{max,2}$ = 150 mm

Since spacing installed = 150 mm is same as $s_{max,1}$, and $s_{max,2}$, = 150 mm, then it can be said that the spacing requirements is fulfilled.

<u>Shear Strength Outside of Plastic Hinge Zone</u>

Calculation of the shear of weak axis:

Shear force that will be designed (V_u) will directly use the result from MIDAS Gen structural analysis.

Vu = 305,800 N

Φ = 0,75

$$V_c = 0.17 \times \frac{(1+Nu)\times 10^3}{(14\times b\times h)} \times \sqrt{fc'} \times b \times \left(h - cc - ds - \frac{db}{2}\right)$$

$$= 0.17 \times \frac{(1+369.1)\times 10^3}{(14\times 600\times 600)} \times \sqrt{30} \times 600 \times \left(600 - 40 - 13 - \frac{25}{2}\right)$$

= 320,482 N

$V_{s,need}$ = Maximum value of $[\frac{Vu}{\Phi} - \mathrm{Vc};0]$

= Maximum value of $[\frac{305{,}800\ \mathrm{N}}{0{,}75} - 320{,}482\ \mathrm{N};0]$

= Maximum value of [87252 N;0]

= 87,252 N

$$A_{v/s\ need} = \frac{Vs,need}{fyt\times\left(b-cc-ds-\frac{db}{2}\right)}$$

$$= \frac{87{,}252\ \mathrm{N}}{420 \times \left(600 - 40 - 13 - \frac{25}{2}\right)}$$

= 0.3887 mm²/mm

$$A_{s/s\ Min\ 1} = 0.062 \times \sqrt{fc'} \times \frac{b}{fyt}$$

$$= 0.062 \times \sqrt{30} \times \frac{600}{420}$$

= 0.4851 mm²/mm

$$A_{s/s\ Min\ 2} = 0.35 \times \frac{b}{fyt}$$

$= 0.35 \times \frac{600}{420}$

$= 0.5$ mm²/mm

$A_{s/s}$ $= \frac{A_{v,weak\ axis}}{s}$

A_v/s $= \frac{265.465\ mm^2}{150\ mm}$

$= 1.769$ mm²/mm

With those minimum requirements, A_v/s = 1.769 mm²/mm still exceeds both $A_{v/s}$,need = 0.3887 mm²/mm, $A_{s/s}$ min,1 = 0.4851 mm²/mm, and $A_{s/s}$ min,2 = 0.4851 mm²/mm. Therefore, the reinforcement's area per spacing is fulfilled the requirement.

Calculation of the shear of strong axis:

Shear force that will be designed (V_u) will directly use the result from MIDAS Gen structural analysis.

Vu = 263,333 N

Φ = 0,75

V_c $= 0.17 \times \frac{(1+Nu)\times 10^3}{(14\times b\times h)} \times \sqrt{fc'} \times b \times \left(h - cc - ds - \frac{db}{2}\right)$

$= 0.17 \times \frac{(1+369.1)\times 10^3}{(14\times 600\times 600)} \times \sqrt{30} \times 600 \times \left(600 - 40 - 13 - \frac{25}{2}\right)$

= 320,482 N

$V_{s,need}$ = Maximum value of $[\frac{Vu}{\Phi} - \mathrm{Vc};0]$

= Maximum value of $[\frac{263{,}333\ \mathrm{N}}{0{,}75} - 320{,}482\ \mathrm{N};0]$

= Maximum value of [30,269 N;0]

= 30,269 N

$A_{v/s\ need}$ $= \frac{Vs,need}{fyt\times\left(b-cc-ds-\frac{db}{2}\right)}$

$= \frac{30{,}629\ \mathrm{N}}{420 \times \left(600 - 40 - 13 - \frac{25}{2}\right)}$

$= 0.1364\ mm^2/mm$

$A_{s/s\ Min\ 1}$ $= 0.062 \times \sqrt{fc'} \times \frac{b}{fyt}$

$= 0.062 \times \sqrt{30} \times \frac{600}{420}$

$= 0.4851\ mm^2/mm$

$A_{s/s\ Min\ 2}$ $= 0.35 \times \frac{b}{fyt}$

$= 0.35 \times \frac{600}{420}$

$= 0.5\ mm^2/mm$

$A_{s/s}$ $= \frac{A_{v,strong\ axis}}{s}$

A_v/s $= \frac{265.465\ mm^2}{150\ mm}$

$= 1.769\ mm^2/mm$

With those minimum requirements, $A_v/s = 1.769\ mm^2/mm$ still exceeds both $A_{v/s}$,need = 0.1364 mm²/mm, $A_{s/s}$ min,1 = 0.4851 mm²/mm, and $A_{s/s}$ min,2 = 0.4851 mm²/mm. Therefore, the reinforcement's area per spacing is fulfilled the requirement.

2.10 Beam-Column Joint

Joint dimension check

According to SNI 2847:2019 ps. 18.8.2.3, if longitudinal reinforcement of a beam will be continued into beam-column joint, dimension of a parallel column with those beam reinforcement can't be less than $20d_b$ (normal-weight concrete).

$20d_b$ = 500 mm < dimension of column = 600 mm

According to SNI 2847:2019 ps. 18.8.4.3, sectional area of a joint, A_j, is calculated by height of joint times width of joint, with height of joint must be same as width of a column, and width of joint must be same as width of a column, unless there are beams that frame into wider columns.\

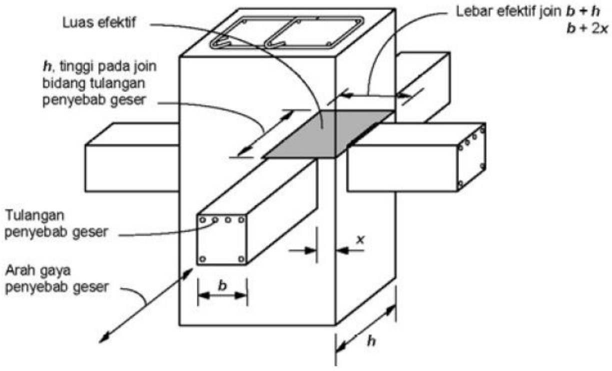


Figure 2.26 Effective Area for Beam-Column Joint

To decide the height of joint (h):

h will be taken as the height of column = 600 mm

h = 600 mm > 0.5 height of beam

h = 600 mm > 0.5 (600) = 300 mm → OK

Effective width will be taken by the smallest value from:

b + h = 350 + 600 = 950 mm

b + 2x = 350 + 2(125) = 600 mm (will be taken)

$$x = \frac{h_{column} - b_{beam}}{2} = \frac{600-350}{2} = 125\ mm$$

A_j = 600(600) = 360,000 mm^2

<u>Shear Force at Joint</u>

From beam design, M_{pr}^- and M_{pr}^+ were already calculated as follow.

M_{pr}^- = 691,706,878 Nmm

M_{pr}^- = 691.707 kNm

M_{pr}^+ = 531,282,959 Nmm

M_{pr}^+ = 510.283 kNm

M_{pr}^- and M_{pr}^+ will also be used to calculate the shear force occurred at the joint.

$$M_e = \frac{{M_{pr}}^- + {M_{pr}}^+}{2}$$

$$M_e = \frac{691.707 + 510.283}{2}$$

M_e = 600.995 kNm

Then, shear force that designed at column (V_e) with effective length, l_n, can be calculated as follow.

$$V_e = \frac{M_e + M_e}{l_n}$$

$$V_e = \frac{2(600.995)}{4 - 0.6}$$

V_e = 353.526 kNm

During the calculation, it will be assumed that sidesway that occur is going to the right. Therefore, tension force of reinforcement and compression force of concrete can be calculated both from the left and right side.

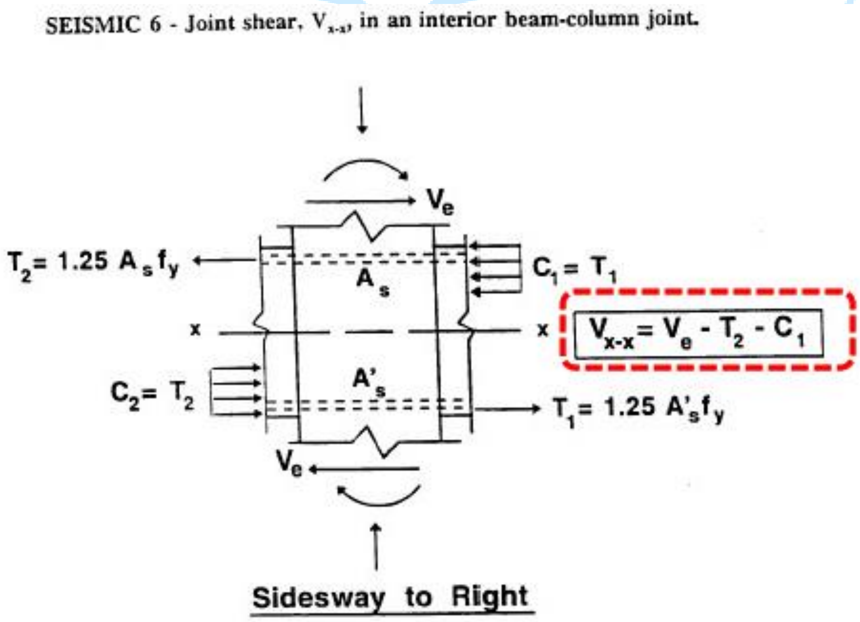


Figure 2.27 Sidesway to Right Diagram Calculation

Forces of the longitudinal reinforcement of beam in the face of joint should be calculated by assuming the stress of the flexible tensile reinforcement is $1.25f_y$.

Tension force of reinforcement and compression force of concrete from the right side can be determined as follow.

Area of reinforcement: $A_{s\_5D25} = 2454.369\ mm^2$

$T_1 = 1{,}25 A_s f_y = 1{,}25 \times 2454.369 \times 420$

$= 1288543.725\ N = 1288.544$ kN

$C_1 = T_1 = 1288.544$ kN

Tension force of reinforcement and compression force of concrete from the left side can be determined as follow.

Area of reinforcement: $A_{s\_7D25} = 3436.117\ mm^2$

$T_2 = 1{,}25 A_s f_y = 1{,}25 \times 3436.117 \times 420$

$= 1803961.407\ N = 1803.961$ kN

$C_2 = T_2 = 1803.961$ kN

Therefore, shear force at joint (V_j) can be determined as follow.

$V_j = V_e - T_2 - C_1$

$V_j = 353.526$ kNm – 1803.961 kN – 1288.544 kN

$V_j = -2735.979$ kNm

Nominal shear strength (V_n) will be determined based on SNI 2847:2019 Table 18.8.4.1. Since the joint will be constrained by beam in all four sides, V_n can be calculated as follow.

$V_n = 1{,}7\lambda\sqrt{f_c{}'}A_j$

$V_n = 1{,}7\sqrt{30} \times 360000$

$= 3352062.052$ N $= 3352.062$ kN

Based on SNI 2847:2019 ps. 21.2.4.3, for beam-column joint, strength reduction factor can be taken as $\phi = 0.85$.

ϕV_n = 0.85(3352.062)

= 2849.2527 kNm

<u>Control of shear force</u>

$\phi V_n > |V_j|$

2849.2527 kNm > 2735.979 kNm (fulfilled the requirement)