CHAPTER II
UPPER STRUCTURE PLANNING

2.1 Preliminary Design

The planning work of the Assalafiyyah Islamic Boarding School building in the City of Yogyakarta, especially for structural work components using steel material for the main structure (slab, beams, and columns). The lower structure of the Assalifayyah Islamic Boarding School building uses reinforced concrete sloof beams and pile foundations. This building is planned to be divided into two parts with dilatation to avoid torsion in the building. Assallafiyyah Islamic Boarding School, Sleman Regency, Yogyakarta, consist of 2 main buildings which have 3 story level each and a 4-meter height each story. This building consists of several structural components which is beams, columns, and slab. This building has main structural columns with a size of 500×500 mm. In addition, this building has 14 types of main beams with a size of 400×200 mm (BI 1 Dorm), a size of 350×200 mm (BI 2 Dorm, BI 2 Edu, and BI 3 Edu), a size of 300×200 mm (BI 3 Dorm and BI 6 Edu), size 250×200 mm (BI 4 Dorm, BI 4 Edu, BI 7 Edu, and BI 8 Edu), size 200×200 mm (BI 5 Dorm, BI 5 Edu and BI 9 Edu). There are 8 types of support beam in this building with a size of 250×200 mm (BA 1 Dorm, BA 3 Dorm, BA 1 Edu and BA 5 Edu), size 350×200 mm (BA 2 Dorm, BA 2 Edu and BA 3 Edu) and size 400×250 mm (BA 4 Edu).

2.1.1 Planning Regulations and Standards

Some of the planning rules and standards used in this work are as follows:

1. Minimum Load for Planning of Buildings and Other Structures
   (SNI 1727:2013)
2. Earthquake Resistance Planning Standards for Building Structures
   (SNI 1726:2019)
3. Procedures for Planning Steel Structures for Buildings (SNI 1729:2015)
4. Procedure for Calculation of Concrete Structures for Buildings
   (SNI 2847:2019)

2.1.2 Structural Material Specification

The specifications of the materials used in this work are as follows:

1. Steel Profile
   - Profil steel we use is BJ 37 with yield stress, fy = 240 MPa and ultimate voltage, fu = 370 MPa
   - Modulus of elasticity of steel, Es = 200.000 MPa
2. Concrete
   - The compressive strength of concrete at the age of 28 days fc’ = 25 MPa (bottom structure)
   - Modulus of elasticity of concrete Ec = 4700 √ fc’ = 23500 MPa
3. Reinforciement steel
   - Reinforcement steel with D > 12 mm, deformed steel with yield stress, fy = 420 MPa is used
   - Reinforcement steel with D ≤ 12 mm, plain reinforcing steel is used with yield stress, fy = 235 MPa
   - Steel's modulus of elasticity, Es = 200.000 Mpa

2.2 Interpretation of Soil Data and Site Class Purchases

2.2.1 Define Site Classification (SA-SF)

The characteristics of the project site, especially those related to geotechnical aspects, must be identified properly in the planning process through site investigation activities. The project site investigation activities can be in the form of soil investigations in the field and in the laboratory. Furthermore, the results of the investigation of the

Site Class	$\overline{Vs}$ (m/s)	$\overline{N}$ or $\overline{Nch}$	$\overline{Su}$ (kPa)
SA (batuan keras)	>1500	N/A	N/A
SB (batuan)	750 sampai 1500	N/A	N/A
SC (tanah keras, sangat padat dan batuan lunak)	350 sampai 750	>50	≥100
SD (tanah sedang)	175 sampai 350	15 sampai 50	50 sampai 100
SE (tanah lunak)	<175	<15	<50
	Atau setiap profil tanah yang mengandung lebih dari 3 m tanah dengan karakteristik sebagai berikut : 1. Indeks plastisitas, PI > 20 2. Kadar air, w ≥ 40% 3. Kuat geser niralir $\overline{Su}$ < 25 kPa		
SF (tanah khusus, yang membutuhkan investugasu geoteknik spesifik dan analisis respon spesifik)	Setiap profil lapisaan tanah yang memiliki salah satu atau lebih dari karakteristik berikut: • Rawan dan berpotensi gagal atau runtuh akbiat beban gempa seperti mudah likuifaksi, lempung sangat sensitive, tanah tersementasi lemah • Lempung sangat organic dan/ atau gambut (ketebalan H > 3 m) • Lempung berplastisitas sangat tinggi (ketebalan H > 7,5 m dengan indeks plastisitas, PI > 75) • Lapisan Lempung lunak/ setengah teguh dengan ketebalan H > 35 m dengan $\overline{Su}$ < 50 kPa		

Table 2. 1 Site Classification

project site will be used as a basis for determining site classification. In SNI 1726:2019 the site classification is divided into 6 types, namely SA (hard rock), SB (rock), SC (hard soil), SD (medium soil), SE (soft soil), and SF.

Based on the results of the N-SPT test conducted in the field, the project location is included in the SD location classification (medium soil). Complete N-SPT data can be seen in the soil investigation report.

2.2.2 Determine the site coefficients (Fa and Fv)

To determine the spectral response of the MCER earthquake acceleration mapped on the ground surface, an amplification factor is needed for a period of 0.2 seconds (Fa) and 1 second (Fv). The amplification factor is determined based on the location class and ground acceleration parameters. The amplification factor in the 0.2 second (Fa) period is determined by the location class and the MCER earthquake acceleration spectral response parameter is mapped for the 0.2 second (Ss) period. While the amplification factor in a period of 1 second (Fv) is determined by the site class and spectral response parameters The MCER earthquake acceleration is mapped for a period of 1 second (S1).

Determination of site coefficients (Fa and Fv) based on Tables 2.2 and 2.3

Site Class	Parameter respons spektral percepatan gempa maksimum yang dipertimbangkan risiko-tertarget (MCE$_R$) terpetakan pada periode pendek, T = 0,2 detik, S_1					
	$S_1 \leq 0{,}25$	$S_1 = 0{,}5$	$S_1 = 0{,}75$	$S_1 = 1{,}0$	$S_1 = 1{,}25$	$S_1 \geq 1{,}5$
SA	0,8	0,8	0,8	0,8	0,8	0,8
SB	0,9	0,9	0,9	0,9	0,9	0,9
SC	1,3	1,3	1,2	1,2	1,2	1,2
SD	1,6	1,4	1,2	1,1	1,0	1,0
SE	2,4	1,7	1,3	1,1	0,9	0,8
SF	$SS^{(a)}$					

Table 2. 2 Site Coefficient, Fa (SNI 1726:2019)

Notes: (a) SS = Site requiring specific geotechnical investigation and site-specific response analysis, see 0

Site Class	Parameter respons spektral percepatan gempa maksimum yang dipertimbangkan risiko-tertarget (MCE$_R$) terpetakan pada periode 1 detik, S_1					
	$S_1 \leq 0{,}25$	$S_1 = 0{,}5$	$S_1 = 0{,}75$	$S_1 = 1{,}0$	$S_1 = 1{,}25$	$S_1 \geq 1{,}5$
SA	0,8	0,8	0,8	0,8	0,8	0,8
SB	0,8	0,8	0,8	0,8	0,8	0,8
SC	1,5	1,5	1,5	1,5	1,5	1,4
SD	2,4	2,2	2,0	1,9	1,8	1,7
SE	4,2	3,3	2,8	2,4	2,2	2,0
SF	$SS^{(a)}$					

Table 2. 3 Site Coefficient, Fv (SNI 1726:2019)

Note: (a) SS = Site requiring specific geotechnical investigation and site-specific response analysis, see 0

Based on Table 2.2 and Table 2.3, for the SD site class (medium soil) the values of Fa and Fv are 1.07524 and 1.8125, respectively. Furthermore, the values of Fa and Fv are used to determine the parameters of the acceleration response spectrum in the short period (SMS) and 1 second period (SM1) which can be calculated using the following equation:

SMS = Fa x Ss = 1.1418g

SM1 = Fv x S1 = 0.8836

2.2.3 Calculating design acceleration parameters (SDS dan SD1)

In the previous step, the SMS and SM1 values have been obtained. Furthermore, based on the SMS and SM1 values, the design spectral acceleration parameters for the short 0.2 second period (SDS) and 1 second period (SD1) need to be determined to construct the spectral response curve. SDS and SD1 values are calculated using the following equation:

$$SDS = 2/3 \times SMS = 0.761\ g$$

$$SD1 = 2/3 \times SM1 = 0.589\ g$$

2.2.4 Constructing the design spectrum response curve

Based on the response spectral parameters calculated in the previous stage, the response spectral design curve can be structured as follows (see Figure 2.4 and Figure 2.5):

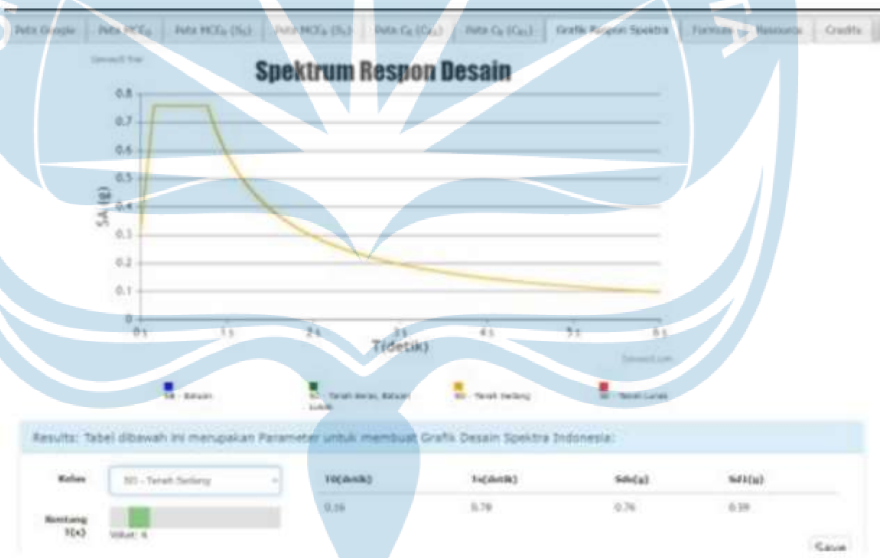


Figure 2. 1 Response Spectrum Design Curve

T	Description	Sa	Description
0,00	0	0,304	0,4 Sds
0,16	To	0,761	Sds
0,78	Ts	0,761	Sda
0,78	Ts+0	0,755	Sa = SD1/T
0,88	Ts+0,1	0,669	Sa = SD1/T
0,98	Ts+0,2	0,601	Sa = SD1/T
1,08	Ts+0,3	0,545	Sa = SD1/T
1,18	Ts+0,4	0,499	Sa = SD1/T
1,28	Ts+0,5	0,460	Sa = SD1/T
1,38	Ts+0,6	0,427	Sa = SD1/T
1,48	Ts+0,7	0,398	Sa = SD1/T
1,58	Ts+0,8	0,373	Sa = SD1/T
1,68	Ts+0,9	0,351	Sa = SD1/T
1,78	Ts+1	0,331	Sa = SD1/T
1,88	Ts+1,1	0,313	Sa = SD1/T
1,98	Ts+1,2	0,297	Sa = SD1/T
2,08	Ts+1,3	0,283	Sa = SD1/T
2,18	Ts+1,4	0,270	Sa = SD1/T
2,28	Ts+1,5	0,258	Sa = SD1/T
2,38	Ts+1,6	0,247	Sa = SD1/T
2,48	Ts+1,7	0,238	Sa = SD1/T
2,58	Ts+1,8	0,228	Sa = SD1/T
2,68	Ts+1,9	0,220	Sa = SD1/T
2,78	Ts+2	0,212	Sa = SD1/T
2,88	Ts+2,1	0,205	Sa = SD1/T
2,98	Ts+2,2	0,198	Sa = SD1/T
3,08	Ts+2,3	0,191	Sa = SD1/T
3,18	Ts+2,4	0,185	Sa = SD1/T
3,28	Ts+2,5	0,180	Sa = SD1/T

T	Desciption	Sa	Description
3,38	Ts+2,6	0,174	Sa = SD1/T
3,48	Ts+2,7	0,169	Sa = SD1/T
3,58	Ts+2,8	0,165	Sa = SD1/T
3,68	Ts+2,9	0,160	Sa = SD1/T
3,78	Ts+3	0,156	Sa = SD1/T
3,88	Ts+3,1	0,152	Sa = SD1/T
3,98	Ts+3,2	0,148	Sa = SD1/T
4,08	Ts+3,3	0,144	Sa = SD1/T
4,18	Ts+3,4	0,141	Sa = SD1/T
4,28	Ts+3,5	0,138	Sa = SD1/T
4,38	Ts+3,6	0,134	Sa = SD1/T
4,48	Ts+3,7	0,131	Sa = SD1/T
4,58	Ts+3,8	0,129	Sa = SD1/T
4,68	Ts+3,9	0,126	Sa = SD1/T
4,78	Ts+4	0,123	Sa = SD1/T
4,88	Ts+4,1	0,121	Sa = SD1/T
4,98	Ts+4,2	0,118	Sa = SD1/T
5,08	Ts+4,3	0,116	Sa = SD1/T
5,18	Ts+4,4	0,114	Sa = SD1/T
5,28	Ts+4,5	0,112	Sa = SD1/T
5,38	Ts+4,6	0,109	Sa = SD1/T
5,48	Ts+4,7	0,107	Sa = SD1/T
5,58	Ts+4,8	0,106	Sa = SD1/T
5,68	Ts+4,9	0,104	Sa = SD1/T
5,78	Ts+5	0,102	Sa = SD1/T
5,88	Ts+5,1	0,100	Sa = SD1/T
5,98	Ts+5,2	0,098	Sa = SD1/T
6,00	Ts+5,22	0,098	Sa = SD1/T
6,08	Ts+5,3	0,097	Sa = SD1/T

Table 2. 4 Period Values and Response Spectrum Acceleration

2.2.5 Define seismic design category (KDS: A - F)

The designed structure must be determined to be included in the seismic design category (KDS) in accordance with Article 6.5 of SNI 1726:2019. In Figure 2.6 and Figure 2.7 are presented seismic design categories based on the relationship between SDS and SD1 with KDS.

Nilai S_{DS}	Kategori risiko	
	I atau II atau III	IV
S_{D1} < 0,067	A	A
0,067 ≤ S_{D1} < 0,133	B	C
0,133 ≤ S_{D1} < 0,20	C	D
0,20 ≤ S_{D1}	D	D

Table 2.5 Seismic Design Categories based on values SDS (SNI 1726:2019)

Nilai S_{D1}	Kategori risiko	
	I atau II atau III	IV
S_{DS} < 0,167	A	A
0,167 ≤ S_{D1} < 0,33	B	C
0,33 ≤ S_{D1} < 0,50	C	D
0,50 ≤ S_{D1}	D	D

Table 2. 6 Seismic Design Categories based on Values SD1 (SNI 1726:2019)

In this work, based on Table 2.6 and Table 2.7, the seismic design categories (KDS) D.

2.2.6 Define system and structure parameters (R, Cd, Ωo)

The seismic force resisting structural system is permitted to be set differently on each of the orthogonal axes of the structure. Parameter R, Cd, Ωo for each type of seismic force resisting structural system is presented in Table 2.7.

Tabel 12 – Faktor R, C_d, dan Ω_0 untuk sistem pemikul gaya seismik

Sistem pemikul gaya seismik	Koefisien modifikasi respons, R[a]	Faktor kuat lebih sistem, Ω_0[b]	Faktor pembesaran defleksi, C_d[c]	Batasan sistem struktur dan batasan tinggi struktur, h_n (m)[d]				
				Kategori desain seismik				
				B	C	D[e]	E[e]	F[f]
A. Sistem dinding penumpu								
1. Dinding geser beton bertulang khusus[a,h]	5	2½	5	TB	TB	48	48	30
2. Dinding geser beton bertulang biasa[a]	4	2½	4	TB	TB	TI	TI	TI
3. Dinding geser beton polos didetail[a]	2	2½	2	TB	TI	TI	TI	TI
4. Dinding geser beton polos biasa[a]	1½	2½	1½	TB	TI	TI	TI	TI
5. Dinding geser pracetak menengah[a]	4	2½	4	TB	TB	12[i]	12[i]	12[i]
6. Dinding geser pracetak biasa[a]	3	2½	3	TB	TI	TI	TI	TI
7. Dinding geser batu bata bertulang khusus	5	2½	3½	TB	TB	48	48	30
8. Dinding geser batu bata bertulang menengah	3½	2½	2¼	TB	TB	TI	TI	TI
9. Dinding geser batu bata bertulang biasa	2	2½	1¾	TB	48	TI	TI	TI
10. Dinding geser batu bata polos didetail	2	2½	1¾	TB	TI	TI	TI	TI
11. Dinding geser batu bata polos biasa	1½	2½	1¼	TB	TI	TI	TI	TI
12. Dinding geser batu bata prategang	1½	2½	1¾	TB	TI	TI	TI	TI
13. Dinding geser batu bata ringan (AAC) bertulang biasa	2	2½	2	TB	10	TI	TI	TI
14. Dinding geser batu bata ringan (AAC) polos biasa	1½	2½	1½	TB	TI	TI	TI	TI
15. Dinding rangka ringan (kayu) dilapisi dengan panel struktur kayu yang ditujukan untuk tahanan geser, atau dengan lembaran baja	6½	3	4	TB	TB	20	20	20
16. Dinding rangka ringan (baja canai dingin) yang dilapisi dengan panel struktur kayu yang ditujukan untuk tahanan geser, atau dengan lembaran baja	6½	3	4	TB	TB	20	20	20
17. Dinding rangka ringan dengan panel geser dari semua material lainnya	2	2½	2	TB	TB	10	TI	TI
18. Sistem dinding rangka ringan (baja canai dingin) menggunakan bresing strip datar	4	2	3½	TB	TB	20	20	20
B. Sistem rangka bangunan								
1. Rangka baja dengan bresing eksentris	8	2	4	TB	TB	48	48	30
2. Rangka baja dengan bresing konsentris khusus	6	2	5	TB	TB	48	48	30
3. Rangka baja dengan bresing konsentris biasa	3¼	2	3¼	TB	TB	10[j]	10[j]	TI[j]
4. Dinding geser beton bertulang khusus[a,h]	6	2½	5	TB	TB	48	48	30
5. Dinding geser beton bertulang biasa[a]	5	2½	4½	TB	TB	TI	TI	TI
6. Dinding geser beton polos detail[a]	2	2½	2	TB	TI	TI	TI	TI
7. Dinding geser beton polos biasa[a]	1½	2½	1½	TB	TI	TI	TI	TI
8. Dinding geser pracetak menengah[a]	5	2½	4½	TB	TB	12[i]	12[i]	12[i]
9. Dinding geser pracetak biasa[a]	4	2½	4	TB	TI	TI	TI	TI
10. Rangka baja dan beton komposit dengan bresing eksentris	8	2	4	TB	TB	48	48	30
11. Rangka baja dan beton komposit dengan bresing konsentris khusus	5	2	4½	TB	TB	48	48	30
12. Rangka baja dan beton komposit dengan bresing biasa	3	2	3	TB	TB	TI	TI	TI
13. Dinding geser pelat baja dan beton komposit	6½	2½	5½	TB	TB	48	48	30
14. Dinding geser baja dan beton komposit khusus	6	2½	5	TB	TB	48	48	30
15. Dinding geser baja dan beton komposit biasa	5	2½	4½	TB	TB	TI	TI	TI
16. Dinding geser batu bata bertulang khusus	5½	2½	4	TB	TB	48	48	30
17. Dinding geser batu bata bertulang menengah	4	2½	4	TB	TB	TI	TI	TI
18. Dinding geser batu bata bertulang biasa	2	2½	2	TB	48	TI	TI	TI

Table 2. 7 R, Cd, Ωo for earthquake resisting system

Tabel 12 – Faktor R, C_d, dan Ω_0 untuk sistem pemikul gaya seismik (lanjutan)

Sistem pemikul gaya seismik	Koefisien modifikasi respons, R^a	Faktor kuat lebih sistem, Ω_0^b	Faktor pembesaran defleksi, C_d^c	Batasan sistem struktur dan batasan tinggi struktur, h_n (m)[d] Kategori desain seismik B	C	D^e	E^e	F^f
19. Dinding geser batu bata polos didetail	2	2½	2	TB	TI	TI	TI	TI
20. Dinding geser batu bata polos biasa	1½	2½	1¼	TB	TI	TI	TI	TI
21. Dinding geser batu bata prategang	1½	2½	1¾	TB	TI	TI	TI	TI
22. Dinding rangka ringan (kayu) yang dilapisi dengan panel struktur kayu yang dimaksudkan untuk tahanan geser	7	2½	4½	TB	TB	22	22	22
23. Dinding rangka ringan (baja canai dingin) yang dilapisi dengan panel struktur kayu yang dimaksudkan untuk tahanan geser, atau dengan lembaran baja	7	2½	4½	TB	TB	22	22	22
24. Dinding rangka ringan dengan panel geser dari semua material lainnya	2½	2½	2½	TB	TB	10	TB	TB
25. Rangka baja dengan bresing terkekang terhadap tekuk	8	2½	5	TB	TB	48	48	30
26. Dinding geser pelat baja khusus	7	2	6	TB	TB	48	48	30
C. Sistem rangka pemikul momen								
1. Rangka baja pemikul momen khusus	8	3	5½	TB	TB	TB	TB	TB
2. Rangka batang baja pemikul momen khusus	7	3	5½	TB	TB	48	30	TI
3. Rangka baja pemikul momen menengah	4½	3	4	TB	TB	10[k]	TI[k]	TI[k]
4. Rangka baja pemikul momen biasa	3½	3	3	TB	TB	TI[i]	TI[i]	TI[i]
5. Rangka beton bertulang pemikul momen khusus[m]	8	3	5½	TB	TB	TB	TB	TB
6. Rangka beton bertulang pemikul momen menengah	5	3	4½	TB	TB	TI	TI	TI
7. Rangka beton bertulang pemikul momen biasa	3	3	2½	TB	TI	TI	TI	TI
8. Rangka baja dan beton komposit pemikul momen khusus	8	3	5½	TB	TB	TB	TB	TB
9. Rangka baja dan beton komposit pemikul momen menengah	5	3	4½	TB	TB	TI	TI	TI
10. Rangka baja dan beton komposit terkekang parsial pemikul momen	6	3	5½	48	48	30	TI	TI
11. Rangka baja dan beton komposit pemikul momen biasa	3	3	2½	TB	TI	TI	TI	TI
12. Rangka baja canai dingin pemikul momen khusus dengan pembautan[n]	3½	3[o]	3½	10	10	10	10	10
D. Sistem ganda dengan rangka pemikul momen khusus yang mampu menahan paling sedikit 25 % gaya seismik yang ditetapkan								
1. Rangka baja dengan bresing eksentris	8	2½	4	TB	TB	TB	TB	TB
2. Rangka baja dengan bresing konsentris khusus	7	2½	5½	TB	TB	TB	TB	TB
3. Dinding geser beton bertulang khusus[g,h]	7	2½	5½	TB	TB	TB	TB	TB
4. Dinding geser beton bertulang biasa[g]	6	2½	5	TB	TB	TI	TI	TI
5. Rangka baja dan beton komposit dengan bresing eksentris	8	2½	4	TB	TB	TB	TB	TB
6. Rangka baja dan beton komposit dengan bresing konsentris khusus	6	2½	5	TB	TB	TB	TB	TB
7. Dinding geser pelat baja dan beton komposit	7½	2½	6	TB	TB	TB	TB	TB
8. Dinding geser baja dan beton komposit khusus	7	2½	6	TB	TB	TB	TB	TB
9. Dinding geser baja dan beton komposit biasa	6	2½	5	TB	TB	TI	TI	TI
10. Dinding geser batu bata bertulang khusus	5½	3	5	TB	TB	TB	TB	TB
11. Dinding geser batu bata bertulang menengah	4	3	3½	TB	TB	TI	TI	TI
12. Rangka baja dengan bresing terkekang terhadap tekuk	8	2½	5	TB	TB	TB	TB	TB
13. Dinding geser pelat baja khusus	8	2½	6½	TB	TB	TB	TB	TB

Figure 2. 8 R, Cd, Ωo for earthquake resisting system table (continuous)

Tabel 12 – Faktor R, C_d, dan Ω_0 untuk sistem pemikul gaya seismik (lanjutan)

Sistem pemikul gaya seismik	Koefisien modifikasi respons, R^a	Faktor kuat lebih sistem, Ω_0^b	Faktor pembesaran defleksi, C_d^c	Batasan sistem struktur dan batasan tinggi struktur, h_n (m)[d] Kategori desain seismik B	C	D[e]	E[e]	F[f]
E. Sistem ganda dengan rangka pemikul momen menengah mampu menahan paling sedikit 25 % gaya seismik yang ditetapkan								
1. Rangka baja dengan bresing konsentris khusus[f]	6	2½	5	TB	TB	10	TI	TI
2. Dinding geser beton bertulang khusus[g,h]	6½	2½	5	TB	TB	48	30	30
3. Dinding geser batu bata bertulang biasa	3	3	2½	TB	48	TI	TI	TI
4. Dinding geser batu bata bertulang menengah	3½	3	3	TB	TB	TI	TI	TI
5. Rangka baja dan beton komposit dengan bresing konsentris khusus	5½	2½	4½	TB	TB	48	30	TI
6. Rangka baja dan beton komposit dengan bresing biasa	3½	2½	3	TB	TB	TI	TI	TI
7. Dinding geser baja dan beton komposit biasa	5	3	4½	TB	TB	TI	TI	TI
8. Dinding geser beton bertulang biasa[g]	5½	2½	4½	TB	TB	TI	TI	TI
F. Sistem interaktif dinding geser-rangka dengan rangka pemikul momen beton bertulang biasa dan dinding geser beton bertulang biasa[g]	4½	2½	4	TB	TI	TI	TI	TI
G. Sistem kolom kantilever didetail untuk memenuhi persyaratan untuk :								
1. Sistem kolom baja dengan kantilever khusus	2½	1¼	2½	10	10	10	10	10
2. Sistem kolom baja dengan kantilever biasa	1¼	1¼	1¼	10	10	TI[j]	TI[j]	TI[j]
3. Rangka beton bertulang pemikul momen khusus[m]	2½	1¼	2½	10	10	10	10	10
4. Rangka beton bertulang pemikul momen menengah	1½	1¼	1½	10	10	TI	TI	TI
5. Rangka beton bertulang pemikul momen biasa	1	1¼	1	10	TI	TI	TI	TI
6. Rangka kayu	1½	1½	1½	10	10	10	TI	TI
H. Sistem baja tidak didetail secara khusus untuk ketahanan seismik, tidak termasuk sistem kolom kantilever	3	3	3	TB	TB	TI	TI	TI

CATATAN

[a] Koefisien modifikasi respons, R, untuk penggunaan pada keseluruhan standar. Nilai R mereduksi gaya ke level kekuatan bukan pada level tegangan izin.

[b] Jika nilai pada tabel faktor kuat lebih, Ω_0, lebih besar atau sama dengan 2,5, maka Ω_0 diizinkan untuk direduksi setengah untuk struktur dengan diafragma fleksibel.

[c] Faktor pembesaran simpangan lateral, C_d, untuk penggunaan dalam 0, 0, dan 0

[d] TB = Tidak Dibatasi dan TI = Tidak Diizinkan.

[e] Lihat 7.2.5.4 untuk penjelasan sistem pemikul gaya seismik yang dibatasi sampai bangunan dengan ketinggian 72 m atau kurang.

[f] Lihat 7.2.5.4 untuk sistem pemikul gaya seismik yang dibatasi sampai bangunan dengan ketinggian 48 m atau kurang.

[g] Dinding geser didefinisikan sebagai dinding struktural.

[h] Definisi "Dinding Struktural Khusus", termasuk konstruksi pracetak dan cor di tempat.

[i] Penambahan ketinggian sampai 13,7 m diizinkan untuk fasilitas gudang penyimpanan satu tingkat.

[j] Rangka baja dengan bresing konsentrik biasa diizinkan pada bangunan satu tingkat sampai ketinggian 18 m di mana beban mati atap tidak melebihi 0,96 kN/m² dan pada struktur griya tawang (*penthouse*).

[k] Lihat 0 untuk struktur yang dikenai kategori desain seismik D, E, atau F.

[l] Lihat 0 untuk struktur yang dikenai kategori desain seismik D, E, atau F.

[m] Definisi "Rangka Momen Khusus", termasuk konstruksi pracetak dan cor di tempat.

Figure 2. 9 R, Cd, Ωo for earthquake resisting system table (continuous)

The seismic force resisting structure system used in the residential structure is SRPMK so that the structural parameters are obtained as follows: R = 8, Cd = 5.5, dan Ωo = 3

2.2.7 Evaluating structural systems for structural irregularities

In the design process, the structure must be classified as a regular or irregular structure by referring to SNI 1726:2019. Structural irregularities are divided into horizontal and vertical irregularities. Furthermore, the types and explanations of horizontal and vertical irregularities are presented in more detail in table 2.11

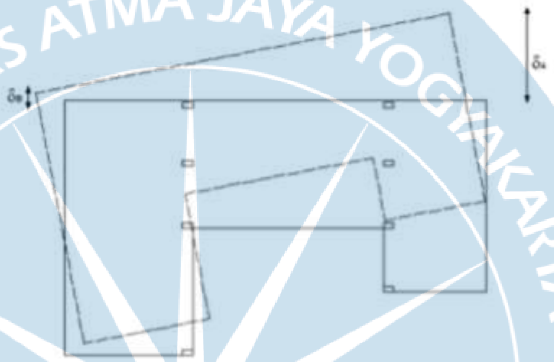
	Tipe dan penjelasan ketidakberaturan	Pasal referensi	Penerapan kategori desain seismik
1a.	Ketidakberaturan torsi didefinisikan ad ajika simpangan antar tingkat maksimum, yang dihitung termasuk torsi tak terduga dengan A_1 = 1,0, disalah satu ujung struktur melintang terhadap suatu sumbu adalah lebih dari 1,2 kali simpangan antar tingkat rata – rata di kedua ujung struktur. Persyaratan ketidakberaturan torsi dalam pasal-pasal teferensi berlaku hanya untuk struktur di mana diafragmanya kaku atau setengah kaku.	0 0 0 0 Tabel 16 0	D, E, dan F B, C, D, E, dan F C, D, E, dan F C, D, E, dan F D, E, dan F B, C, D, E, dan F
1b.	Ketidakberaturan torsi berlebihan didefiniskan ada jika simpangan antar tingkat maksimum yang dihitung termasuk akibat torsi tak terduga dengan A_1 = 1,0, disalah satu ujung struktur melintang terhadap suatu sumbu adalah lebih dari 1,4 kali simpangan antar tingkat rata-rata di kedua ujung struktur. Persyaratan ketidakberaturan torsi berlebihan dalam pasal-pasal referensi berlaku hanya untuk struktur di mana diafragmanya kaku atau setengah kaku.	0 0 0 0 0 0 Tabel 16 0	E dan F D B, C, dan D C dan D C dan D D B, C, dan D
2.	Ketidakberaturan sudut dalam didefinsikan ada jika kedua dimensi proyek denah struktur dari lokasi sudut dalam lebih besar dari 15% dimensi denah struktur dalam arah yang ditinjau.	0 Tabel 16	D,E, dan F D,E, dan F
3.	Ketidakberaturan diskontinuitas diafragma didefinisikan ada jika terdapat suatu diafragma yang memiliki diskontinuitas atau variasi kekakuan mendadak, termasuk yang mempunyai daerah terpotong atau terbuka lebih besar dari 50% daerah diafragma bruto yang tertutup, atau perubahan kekakuan diafragma efektif lebih dari 50% dari suatu tingkat ke tingkat selanjutnya.	0 Tabel 16	D,E, dan F D,E, dan F
4.	Ketidakberaturan akibat pergeseran tegak lurus terhadap bidang didefinisikan ad ajika terdapa diskontinuitas dalam lintasan tahanan gaya lateral, seperti pergeseran tegak lurus terhadap bidang pada setidaknya satu elemen vertical pemikul gaya lateral.	0 0 0 Tabel 16 0	B,C,D,E, dan F D,E, dan F B,C,D,E dan F D,E, dan F B,C,D,E, dan F
5.	Ketidakberaturan sistem non parallel didefinisikan ada jika elemen vertical pemikul gaya lateral tidak parallel terhadap sumbu-sumbu orthogonal utama sistem pemikul gaya seismik.	0 0 Tabel 16 0	C,D,E, dan F B,C,D,E, dan F D,E, dan F B,C,D,E, dan F

Table 2. 10 Types and descriptions of structural horizontal irregularities table

The following is the result of calculating and checking the horizontal

irregularity of the structure:

- Torsional irregularity, defined to exist if the maximum story drift (calculated torque including unexpected torque) at an end of the structure transverse to the axis is more than 1.2 times the average story drift at both ends of the structure (see Figure 2.2). The torsional irregularity requirements in the reference articles apply only to structures whose diaphragms are rigid (rigid) or semi-rigid (semi-rigid).



$$\delta_{avg} = \frac{\delta_A + \delta_B}{2} \qquad A_x = \left[\frac{\delta_{max}}{1.2(\delta_{avg})}\right]^2$$

Figure 2.2 Illustration of checking irregularities of type 1a and 1b (SNI 1726:2019)

Based on checking for torsional irregularities, the result is that the maximum floor deviation in the X and Y directions is less than 1.2 times the average floor deviation so that there is no type 1a horizontal irregularity in the structure under review.

- Excessive torsional irregularity is defined to exist if the maximum grade deviation (calculated torque including unexpected torque) at the ends of the structure transverse to the axis is more than 1.4 times the average grade deviation at both ends of the structure (see Figure 2.12). The excessive torsional irregularity requirement in the reference clause applies only to structures where the diaphragm is rigid or semi-rigid. Based on checking for excessive torsional irregularities, it was found that the maximum floor deviation in the X and Y directions is less than 1.4 times the average floor deviation so that there is no type 1b horizontal irregularity in the structure under consideration.

- Inner angle irregularity, defined to exist if the two structural plan projections from the inner corner are greater than 15% of the structural plan dimensions in the specified direction (see Figure 2.3)

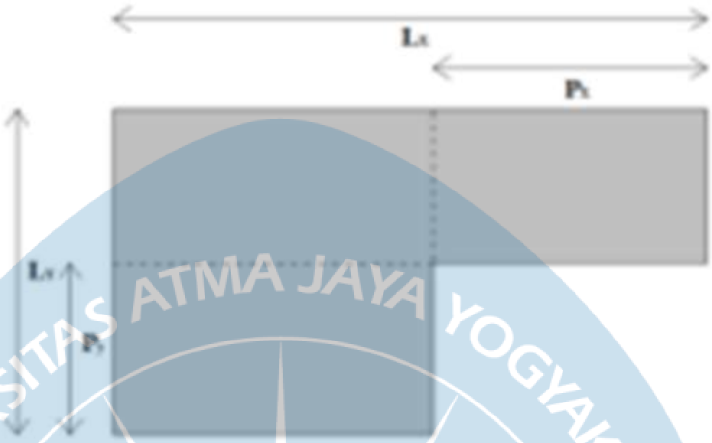


Figure 2. 3 Illustration of checking type 2 horizontal irregularity (SNI 1726:2019)

Based on checking the interior angle irregularities, it was found that the two structural plan projections from the interior angle were less than 15% of the structural plan dimensions in the specified direction so that there was no type 2 horizontal irregularity in the structure under consideration.

- Diaphragmatic discontinuity irregularities, defined to exist if there is a diaphragm with a sudden discontinuity or variation in stiffness, including one having a cut or open area greater than 50% of the gross diaphragm area surrounding it, or a change in effective diaphragm stiffness of more than 50% from one story to the next (see Figure 2.4)

Figure 2. 4 Illustration of checking type 3 horizontal irregularity (SNI 1726:2019)

Based on checking the interior angle irregularities, it was found that there was no type 3 horizontal irregularity in the structure under review

- Transverse displacement irregularities with respect to the plane, defined to exist if there is a discontinuity in the path of lateral resistance, such as transverse shear with respect to the plane of a vertical element (see Figure 2.5)

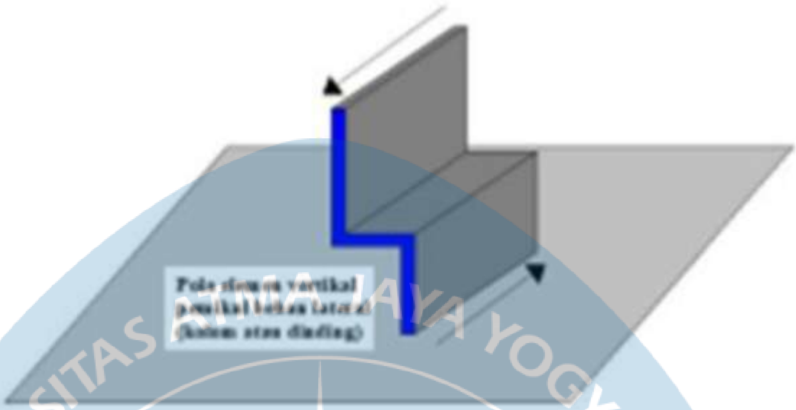


Figure 2. 5 Illustration of checking type 4 horizontal irregularity (SNI 1726:2019)

Based on checking the interior angle irregularities, it was found that there was no type 4 horizontal irregularity in the structure under review

- non-parallel system irregularity, defined to exist if the vertical lateral resisting elements are not parallel or symmetrical to the main orthogonal axes of the seismic resisting system (see Figure 2.6)

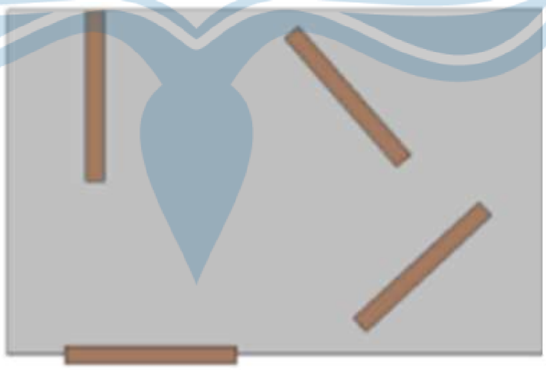


Table 2. 6 Illustration of checking type 5 horizontal irregularity (SNI 1726:2019)

Based on checking the interior angle irregularities, it was found that there was no type 5 horizontal irregularity in the structure under review.

	Tipe dan penjelasan ketidakberaturan	Pasal referensi	Penerapan kategori desain seismik
1a.	Ketidakberaturan Kekuatan Tingkat Lunak didefinisikan ada jika terdapat suatu tingkat yang kekakuan lateralnya kurang dari 70% kekakuan lateral tingkat diatasnya atau kurang dari 80% kekakuan rata-rata tiga tingkat diatasnya.	Tabel 16	D, E, dan F
1b.	Ketidakberaturan Kekakuan Tingkat Lunak Berlebihan didefinisikan ad ajika terdapat suatu tingkat yang kekakuan lateralnya kurang dari 60% kekakuan lateral tingkat diatasnya atau kurang dari 70% kekakuan rata-rata tiga tingkat diatasnya.	0 Tabel 16	E dan F D, E, dan F
2.	Ketidakberaturan Berat (Massa) didefinisikan ad ajika massa efektif di sebarang tingkat lebih dari 150% massa efektif tingkat di dekatnya. Atap yang lebih ringan dari lantai di bawahnya tidak perlu ditinjau.	Tabel 16	D, E, dan F
3.	Ketidakberaturan Geometri Vertikal didefinisikan ad ajika dimensi horizontal sistem pemikul gaya seismic di sebarang tingkat lebih dari 130% dimensi horizontal sistem pemikul gaya seismic tingkat didekatnya.	Tabel 16	D, E, dan F
4.	Ketidakberaturan Akibat Diskontinuitas Bidang pada Elemen Vertikal Pemikul Gaya Lateral didefinisikan ada jiak pergeseran arah bidang elemen pemikul gaya lateral lebih besar dari panjang elemen itu atau terdapat reduksi kekakuan elemen pemikul di tingkat dii bawahnya.	0 0 Tabel 16	B, C, D, E, dan F D, E, dan F D, E, dan F
5a.	Ketidakberaturan Tingkat Lemah Akibat Diskontinuitas pada Kekuatan Lateral Tingkat didefinisikan ad ajika kekuatan lateral suatu tingkat kurang dari 80% kekuatan lateral tingkat diatasnya. Kekuatan lateral tingkat adalah kekuatan total semua elemen pemikul seismic yang berbagi geser tingkat pada arah yang ditinjau.	0 Tabel 16	E dan F D, E, dan F
5b.	Ketidakberaturan Tingkat Lemah Berlebihan Akibat Diskontinuitas pada Kekuatan Lateral Tingkat didefinsikan ad ajika kekuatan lateral suatu tingkat kurang dari 65% kekuatan lateral tingkat diatasnya. Kekuatan lateral tingkat adalah kekuatan total semua elemen pemikul seismic yang berbagi geser tingkat pada arah yang ditinjau.	0 0 Tabel 16	D,E, dan F B dan C D, E, dan F

Table 2. 11 Types and descriptions of structural vertical irregularities table (SNI 1726:2019)

Following are the results of calculations and checking of structural vertical irregularities:

- Soft story stiffness irregularity is defined to exist if there is a story where the lateral stiffness is less than 70% of the lateral stiffness of the story above or less than 80% of the average stiffness of 3 stories above. Above it (see Figure 2.7). Based on checking the soft level stiffness irregularity, it was found that there was no type 1a vertical irregularity in the structure under review.

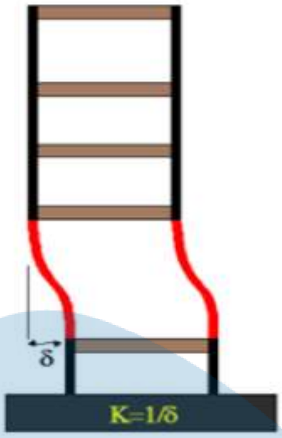


Figure 2. 7 Illustration of checking vertical irregularities of type 1a and 1b (SNI 1726:2019)

- Excessive soft story stiffness irregularity, defined to exist if there is a story where the lateral stiffness is less than 60% of the lateral stiffness of the story above or less than 70% of the average stiffness. 3 levels above it (see Figure 2.18). Based on checking the excessive soft level stiffness irregularity, it was found that there was no type 1b vertical irregularity in the structure under review.
- Heavy (mass) irregularity is defined to exist if the effective of all levels is more than 150% of the effective level of the nearby. A roof that is lighter than the floor below does not need to be considered (see Figure 2.8). Based on checking the weight (mass) irregularity, it was found that there was no type 2 vertical irregularity in the structure under review.

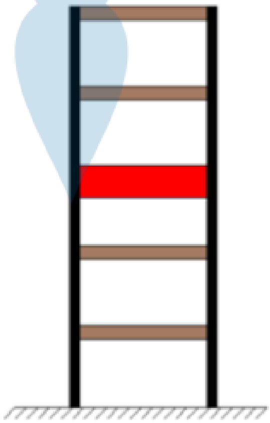


Figure 2. 8 Illustration of checking type 2 vertical irregularity (SNI 1726:2019)

- Vertical geometric irregularity, defined to exist if the horizontal dimension of the seismic retaining system at all levels is more than 130% of the horizontal dimension of the seismic restraint system of the adjacent story (see Figure 2.9).

Based on checking the vertical geometric irregularities, it was found that there was no type 3 vertical irregularity in the structure under review.

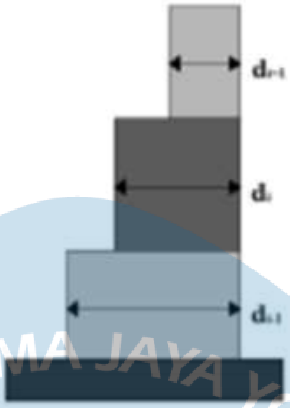


Figure 2. 9 Illustration of checking type 3 vertical irregularity (Source:SNI 1726:2019)

- The plane discontinuity in the irregularity of the vertical lateral resisting element is defined to exist if the plane displacement of the lateral resisting element is greater than the length of the element or there is a reduction in the stiffness of the retaining element in the story below (see Figure 2.10). Based on checking the irregularity of the discontinuity of the plane direction in the irregularity of the vertical lateral force resisting element, it was found that there was no type 4 vertical irregularity in the structure under review.

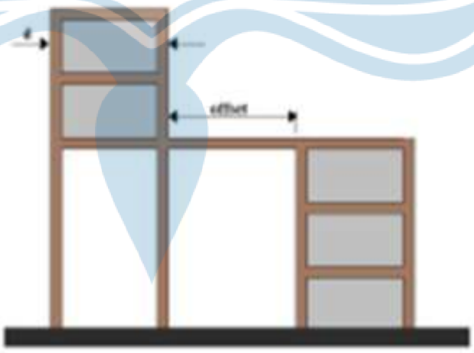


Figure 2. 10 Illustration of checking type 4 vertical irregularity (SNI 1726:2019)

- A discontinuity in the story lateral strength irregularity is defined to exist if the story lateral strength is less than 80% of the story above it. The story lateral strength is the total lateral strength of all seismic retaining elements sharing the story shear for the direction under consideration (see Figure 2.11). Based on checking the irregularities of the discontinuities in the lateral strength irregularities of the story, it was found that there was no vertical irregularity of type 5a in the structure under review.

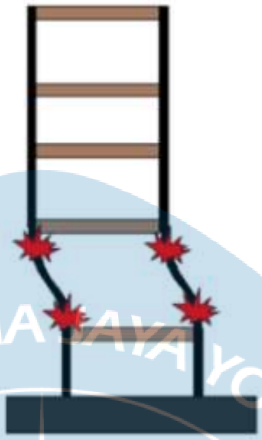


Figure 2. 11 Illustration of checking vertical irregularities types 5a and 5b (SNI 1726:2019)

- A discontinuity in the excessive story lateral strength irregularity, defined to exist if the story lateral strength is less than 65% of the story above it. The story lateral strength is the total strength of all seismic retaining elements sharing the story shear for the direction under consideration (see Figure 2.22). Based on checking the irregularity of the discontinuities in the excessive lateral strength irregularities, it was found that there was no type 5b vertical irregularity in the structure under review.

2.2.8 Determining the flexibility of the diaphragm

For structures having horizontal structural irregularities, the diaphragm should be modeled as semi-rigid. In this work, the residential structure does not have horizontal structural irregularities so that the diaphragm is modeled as a rigid diaphragm.

2.2.9 Determine the redundancy factor (ρ)

The redundancy factor (ρ) must be applied to the seismic force resisting structural system in each of the two orthogonal directions for all structures in accordance with Article 7.3.4 of SNI 1726:2019. The value of can be taken as equal to 1.0 if each story resisting more than 35% of the base shear force in the direction under consideration must comply with the requirements of table 2.12

Elemen pemikul gaya lateral	Persyaratan
Rangka dengan bresing	Penghilang suatu bresing individu, atau sambungan yang tehubung, tidak akan mengakibatkan reduksi kekuatan tingkat lebih dari 33, dan tidak akan menghasilkan system dengan ketidakberaturan torsi yang berlebihan (ketidakberaturan struktur horizontal Tipe 1b).
Rangka pemikul momen	Kehilangan tahananan momen di sambungan balok-kolom di kedua ujung suatu balok tunggal tidak akan mengakibatkan reduksi kekuatan tingkat lebih dari 33%, dan tidak akan menghasilkan sistem dengan ketidakberaturan torsi yang berlebihan (ketidak beraturan struktur horizontal 1b).
Dinding geser atau pilar dinding dengan rasio tinggi terhadap panjang lebih besar dari 1.0	Penghilang suatu dinding geser atau pilar dinding dengan rasio tinggi terhadap panjang lebih besar dari 1.0 di seberang tingkat, atau sambungan kolektor yang terhubung, tidak akan mengakibatkan reduksi kekuatan tingkat lebih dari 33%, dan tidak akan menghasilkan sistem dengan ketidakberaturan torsi yang berlebihan (ketidak beraturan struktur horizontal 1b).
Kolom kantilever	Kehilangan tahanan momen di sambungan dasar pada sebarang kolom kantilever tunggal tidak akan mengakibatkan reduksi kekuatan tingkat lebih dari 33%, dan tidak akan menghasilkan sistem dengan ketidakberaturan torsi yang berlebihan (ketidak beraturan struktur horizontal 1b).
Lainnya	Tidak ada persyatan

Table 2. 12 Requirements for individual stories resisting more than 35% of the base shear force (SNI 1726:2019)

Another condition that allows to be taken as equal to 1.0 is if the structure has a regular plan at all levels provided the seismic force resisting system consists of at least two seismic force resisting perimeter spans framing on each side of the structure in each of the orthogonal directions. in each grade resisting more than 35% of the base shear force. The number of spans for shear walls shall be calculated as the length of the shear wall divided by the story height or twice the length of the shear wall divided by the story height for light frame construction. If these conditions are not met then, must be taken equal to 1.3. In this work, the redundancy factor used is 1.3

2.2.10 Selecting the lateral force/earthquake analysis procedure (ELF, RS, TH)

Earthquake loads regulated in SNI 1726:2019 can be carried out through 3 types of analytical procedures, namely the analysis of equivalent lateral forces, analysis of the response spectrum of the body (response spectra), and historical procedures.Seismic response (time history). The seismic load analysis procedure that is permitted to be used is influenced by the seismic design category and structural characteristics as presented in table 2.13.

Based on table 2.13, in this work it is permitted to use the analysis of the variance response spectrum as the seismic load analysis procedure.

Kategori desain seismik	Karakteristik struktur	Analisis gaya lateral ekivalen pasal 0	Analisis spektrum response ragam pasal 0	Prosedur respons Riwayat waktu seismik pasal 0
B, C	Semua struktur	I	I	I
D, E, F	Bangunan dengan kategori risiko I atau II yang tidak melabihi 2 tingkat diatas dasar	I	I	I
	Struktur tanpa ketidak beraturan structural dan ketinggian tidak melebihi 48,8 m	I	I	I
	Struktur tanpa ketidak beraturan structural dengan ketinggian melebihi 48,8 m dan *T* < 3,5 *T*	I	I	I
	Struktur dengan ketinggian tidak melebihi 48,8 m dan hanya memiliki ketidak beraturan horizontal tipe 2,3,4 atau 5 atau ketidak beraturan vertikal tipe 4, 5a atau 5b	I	I	I
	Semua struktur lainnya	TI	I	I

Table 2. 13 Analytical procedures that may be used table (SNI 1726:2019)

Calculating earthquake loads using the equivalent lateral force (ELF) procedure The equivalent lateral force (ELF) analysis procedure is based on the first response mode. This analytical procedure applies only to regular structures with T < 3.5 Ts (where Ts = SD1/SDS), the stiffness of adjacent stories does not differ by more than 30%, the strengths of adjacent stories do not differ by more than 20%, and masses at adjacent levels do not differ by more than 50%. If this is not met, dynamic analysis procedures should be used, i.e. response spectrum analysis of variance or time history procedures. In general, the magnitude of the earthquake force generated by the ELF analysis procedure is a function of the effective earthquake weight (Wt) and the earthquake response coefficient (Cs). Furthermore, earthquake forces are distributed to each level of the building structure to be designed. Earthquake forces generated from the ELF analysis procedure need to be taken into account because if using dynamic analysis procedures, the resulting earthquake forces need to be compared with the earthquake forces generated by the ELF analysis procedure. The steps for calculating earthquake forces using the ELF analysis procedure are presented as follows:

1. Determining the Period of the Natural Fundamental Structure (T)

The natural fundamental period of the structure will determine the value of the seismic response coefficient (Cs) which will also determine the value of the seismic base shear force (VELF). If a more accurate structural period (Tc) is not available, the structural period used can be taken as Ta. However, if a more accurate structural period (Tc) can be obtained (through structural modeling) then the structural period used should be determined by following the following conditions (see also Figure 2.12):

So T =

- If Tc > Cu.Ta Cu.Ta
- If Ta < Tc < Cu.Ta So T = Tc
- If Tc < Ta So T = Ta

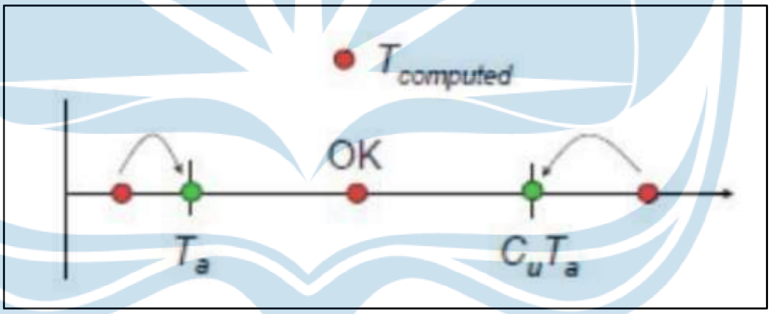


Figure 2. 12 Determination of the period of the structure used (FEMA 481)

The approximation fundamental period (Ta) is determined on the basis of equation: Ta = Ct . h x. Where hn is the height of the structure (in m), while the coefficients Ct and x are determined based on

Tipe struktur	C_i	x
Sistem rangka pemikul momen di mana rangka memikul 100% gaya seismic uang disyaratkan dan tidak dilingkupi atau dihubungkan dengan komponen yang lebih kaku dan akan mencegah rangka dari defleksi jika dikenai gaya seismik: • Rangka baja pemikul momen • Rangka beton pemikul momen	 0,0724 0,0466	 0,8 0,9
Rangka baja dengan bresing eksentris	0,0731	0,75
Rangka baja dengan bresing terkekang terhadap tekuk	0,0731	0,75
Semua sistem struktur lainnya	0,0488	0,75

Table 2. 14 Parameter values for the approach period Ct and x table

The coefficient values for the upper limit of the calculated structure period (Cu) are set according to Table 2.15

Parameter percepatan response spectral desain pada 1 detik, S_{d1}	Koefisien C_s
≥ 0,4	1,4
0,3	1,4
0,2	1,5
0,15	1,6
≤ 0,1	1,7

Table 2. 15 Coefficients for the upper bound in the calculated period

In this work, the type of structure used is a moment-bearing steel frame so that the values of Ct = 0.0724 and x = 0.8 are obtained. Furthermore, based on the value of SD1 = 0.589 g, the coefficient of Cu = 1.4 is obtained. So that the value of Ta = 0.926 seconds and Cu.Ta = 1.297 seconds is obtained. The period value of the structure modeling results, Tc = 0.713 seconds (Ta < Tc < Cu.Ta) so that the period of the structure used in the earthquake load analysis with the ELF procedure is 0.926 seconds.

2. Determining the Seismic Response Coefficient (Cs)

The seismic response coefficient (Cs) is determined based on the following equation: Cs = SDS / (R / Ie) The Cs value calculated according to the above equation need not exceed the Cs value calculated by the following equation: Cs = SD1 / (T x (R / Ie)

However, the value of Cs must not be less than Cs which is calculated by the following equation:Cs = 0,044 SDS Ie ≥ 0,01

In this work, the results of the calculation of the seismic response

coefficient (Cs) are as follows:

Cs = SDS / (R / Ie) 0.142

Cs = SD1 / (T x (R / Ie) 0.110

Cs = 0,044 SDS Ie ≥ 0,01 0.050

3. Determining Effective Seismic Weight (W)

The effective seismic weight of the structure (W) shall include all dead loads and other loads included in the following list:

● In areas used for storage: a minimum of 25% live floor load (floor live load in public garages and open parking structures, and storage loads not exceeding 5% of the effective seismic weight on a floor, need not be included)

● If provisions for partitions are required in the floor load design: taken as the largest of the actual partition weight or minimum floor area weight of 0.48 kN/m2

● Total operating weight of permanent equipment

● Landscaping weight and other loads on roof gardens and other similar areas

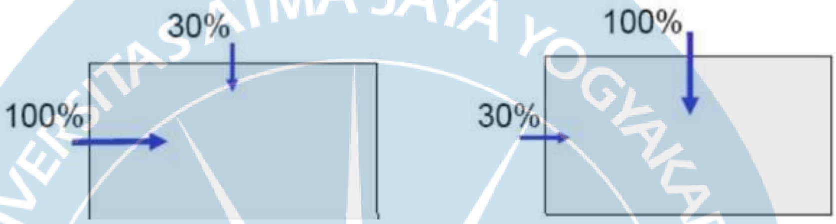


Figure 2. 13 Orthogonal Loads (FEMA 431 B)

Calculate and add orthogonal loads (if required) The addition of orthogonal loads is done by providing an additional load of 30% of the main lateral load, perpendicular to the direction of the main load under consideration (see Figure 2.13). Orthogonal loads need to be added and applied to structures with seismic design categories C, D, E, and F. In this work, the structures are included in seismic design category D so that additional orthogonal loads need to be done. This addition is accommodated in the combined design load.

Calculate and add torque loads (if required) The building structure for all seismic design categories (KDS) must consider the design torque and the unexpected torque. Unexpected torque is applied to the structural model by providing an eccentricity of 5% in the X and Y axes, respectively (see Figure 2.14).

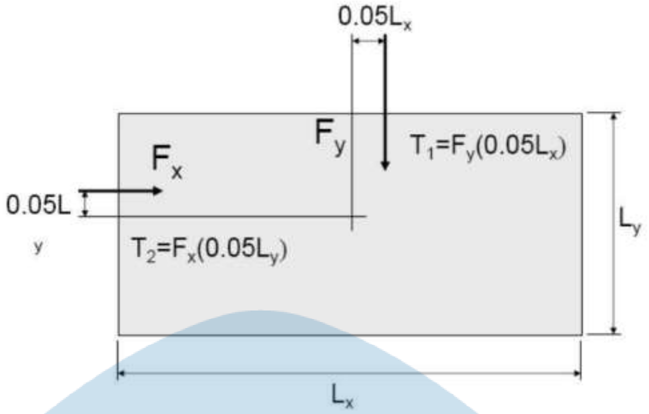


Figure 2. 14 Unexpected Torque

If the building structure is included in seismic design categories C, D, E, and F and has torsional irregularities 1a and 1b, the unexpected torsional enlargement must be considered (see Figure 2.19). The unexpected torque magnification is calculated using the following equation:

ex = eox + (0.05 B Ax)

ey = eoy + (0.05 L Ay)

means,

eox dan eoy are the congenital eccentricity, while 0.05 B Ax and 0.05 L Ay are the unexpected eccentricities.

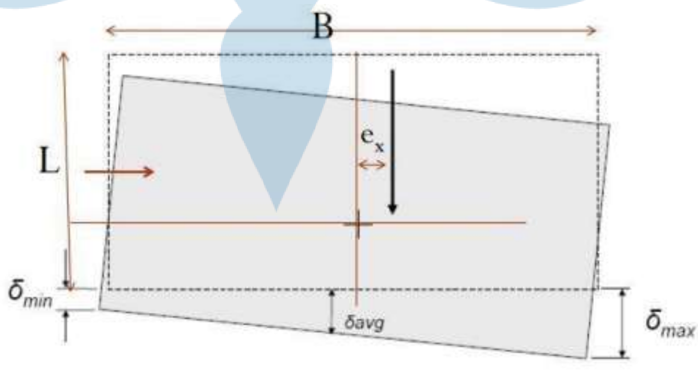


Figure 2. 15 Unexpected torque magnification

In this work, the building structure is included in the seismic design category (KDS) D but there is no torsion irregularity 1a and 1b so there is no need to consider unexpected torque enlargement (it is enough to consider unexpected torque by giving 5% eccentricity each in the X-axis direction and Y.)

2.2.11 Load Combination Plan

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

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

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

1. (1.2+0.2SDS)DL + 1.0LL ± 0.3ρEx ± 1.0ρEy
2. (1.2+0.2SDS)DL + 1.0LL ± 1.0ρEx ± 0.3ρEy
3. (0.9-0.2SDS)DL ± 0.3ρEx ± 1.0ρEy
4. (0.9-0.2SDS)DL ± 1.0ρEx ± 0.3ρEy

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

1. DL
2. DL + LL
3. DL + (Lr or R)
4. DL + 0.75LL + 0.75(Lr or R)
5. DL + (0.6W or 0.7E)

6. DL + 0.75(0.6W or 0.7E) + 0.75LL + 0.75(Lr or R)

7. 0.6DL + 0.6W

8. 0.6DL + 0.7E

With means,

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

LL = Live load

Lr = Live load on the roof structure

R = Rain load W = Wind load

Ex = earthquake load direction x Ey = earthquake load direction y

SDS = Design spectral acceleration parameter for a short period of 0.2 seconds

The ultimate load combinations used in this work are presented in table 2.16.

Kombinasi Beban	DL	LL	Ex	Ey
COMB1	1,40	-	-	-
COMB2	1,20	1,60	-	-
COMB3	1,296	1,00	1,30	0,39
COMB4	1,296	1,00	1,30	-0,39
COMB5	1,296	1,00	-1,30	0,39
COMB6	1,296	1,00	-1,30	-0,39
COMB7	1,296	1,00	0,39	1,30
COMB8	1,296	1,00	-0,39	1,30
COMB9	1,296	1,00	0,39	-1,30
COMB10	1,296	1,00	-0,39	-1,30
COMB11	0,8	-	1,30	0,39
COMB12	0,8	-	1,30	-0,39
COMB13	0,8	-	-1,30	0,39
COMB14	0,8	-	-1,30	-0,39
COMB15	0,8	-	0,39	1,30
COMB16	0,8	-	-0,39	1,30

Table 2. 16 Ultimate load combination

2.3 Structural System Determination

2.3.1 Structural System

The structure of the Assalafiyyah Islamic Boarding School is designed using a special moment-bearing frame system (SRPMK) with columns and beams as a moment reinforcement.

2.3.2 Structural Model

The structural design process is carried out based on the internal forces that occur in the structural elements due to the working ultimate load. This internal force can be obtained through structural modeling. Structural modeling is carried out in a three-dimensional model (3D model). In structural modeling, truss elements are used to idealize beams and columns. While the floor slab is idealized as a deck.

2.3.3 Use Limit Performance

Service limit performance is evaluated based on a combination of service loads. One of the service limit parameters that will be evaluated is the deviation between floors due to the influence of the design earthquake for each of the orthogonal axes of the structure. The deviation between floors that occurs must be smaller than the allowable deviation between floors with the aim of limiting the occurrence of excessive steel melting and concrete cracking, preventing non-structural damage, and preventing excessive deviations that cause discomfort for building occupants

2.3.4 Ultimate Limit Performance

At the ultimate limit performance, the ultimate load combination is used to analyze the internal forces that occur in the structural elements. These internal forces are then used in the design process of structural elements such as slabs, beams, columns, foundations, etc. The ultimate limit performance will determine the safety of the structure in supporting the ultimate design load acting on the structure.

2.4 Structure Loading Planning

2.4.1 Gravity Load

Gravity load is determined based on SNI 1727:2013 Minimum Load for Design of Buildings and Other Structures. The gravity load in the structural design of a residential house includes the structure's own weight/dead load (DL), additional dead load (ADL), and live load (LL). These expenses are explained as follows:

a. Self Weight Structure (DL)

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

b. Additional Dead Load (ADL)

Additional dead load is an additional load due to the use of non-structural components (architectural and MEP) that are attached to and burden the main structure of the building. The additional dead load is explained as follows:

Additional Dead Load On Floor Slab

- Slab load = (120 / 1000) x 24 kN/m^3= 2.88 kN/m^2
- Sand (thickness 4 cm) = 0.04 x 17 kN/m^3= 0.68 kN/m^2
- Spesi (thickness 2 cm) = 0.02 x 20 kN/m^3= 0.40 kN/m^2
- Ceramic/cover (thickness 1 = 0.01 x 24 kN/m^3= 0.24 kN/m^2
- Partition = 1 kN/m^2
- Ceiling, MEP Installation, etc. = 0.25 kN/m^2

Total Additional Dead Load = 5.45 kN/m^2

Total Additional Dead Load (Software Input)

without Slab load = 2.57 kN/m^2

c. Live Load (LL)

Live load is the load that occurs as a result of the use of the building structure. The living burden can come from people/goods that can move from place to place. The dormitory and educational building is included in the category of public space according to SNI 1727:2013 so that the living load is determined as follows:

Live Load = 4.79 kN/m^2

2.4.2 Earthquake Load

Earthquake loads are determined based on SNI 1726:2019 Earthquake Resistance Planning Standards for Building Structures. The steps for calculating the design earthquake load are presented as follows:

a. Determine the building risk category (I-IV)

The building risk category is determined based on the operational function/type of utilization of a building. In SNI 1726:2019, building risk categories are divided into 4 types, namely risk categories I, II, III, and IV (see table 2.17). In this work, the structure of the Assalafiyyah Islamic boarding school is included in the category of public buildings so that it is designated as building risk category IV.

Tabel 3 – Kategori risiko bangunan gedung dan nongedung untuk beban gempa

Jenis pemanfaatan	Kategori risiko
Gedung dan nongedung yang memiliki risiko rendah terhadap jiwa manusia pada saat terjadi kegagalan, termasuk, tapi tidak dibatasi untuk, antara lain: - Fasilitas pertanian, perkebunan, perternakan, dan perikanan - Fasilitas sementara - Gudang penyimpanan - Rumah jaga dan struktur kecil lainnya	I
Semua gedung dan struktur lain, kecuali yang termasuk dalam kategori risiko I,III,IV, termasuk, tapi tidak dibatasi untuk: - Perumahan - Rumah toko dan rumah kantor - Pasar - Gedung perkantoran - Gedung apartemen/ rumah susun - Pusat perbelanjaan/ mall - Bangunan industri - Fasilitas manufaktur - Pabrik	II
Gedung dan nongedung yang memiliki risiko tinggi terhadap jiwa manusia pada saat terjadi kegagalan, termasuk, tapi tidak dibatasi untuk: - Bioskop - Gedung pertemuan - Stadion - Fasilitas kesehatan yang tidak memiliki unit bedah dan unit gawat darurat - Fasilitas penitipan anak - Penjara - Bangunan untuk orang jompo Gedung dan nongedung, tidak termasuk kedalam kategori risiko IV, yang memiliki potensi untuk menyebabkan dampak ekonomi yang besar dan/atau gangguan massal terhadap kehidupan masyarakat sehari-hari bila terjadi kegagalan, termasuk, tapi tidak dibatasi untuk: - Pusat pembangkit listrik biasa - Fasilitas penanganan air - Fasilitas penanganan limbah - Pusat telekomunikasi Gedung dan nongedung yang tidak termasuk dalam kategori risiko IV, (termasuk, tetapi tidak dibatasi untuk fasilitas manufaktur, proses, penanganan, penyimpanan, penggunaan atau tempat pembuangan bahan bakar berbahaya, bahan kimia berbahaya, limbah berbahaya, atau bahan yang mudah meledak) yang mengandung bahan beracun atau peledak di mana jumlah kandungan bahannya melebihi nilai batas yang disyaratkan oleh instansi yang berwenang dan cukup menimbulkan bahaya bagi masyarakat jika terjadi kebocoran.	III

Table 2. 17 Building and non-building risk categories (SNI 1762:2019)

Jenis pemanfaatan	Kategori risiko
Gedung dan nongedung yang dikategorikan sebagai fasilitas yang penting, termasuk, tetapi tidak dibatasi untuk: - Bangunan-bangunan monumental - Gedung sekolah dan fasilitas pendidikan - Rumah ibadah - Rumah sakit dan fasilitas kesehatan lainnya yang memiliki fasilitas bedah dan unit gawat darurat - Fasilitas pemadam kebakaran, ambulans, dan kantor polisi, serta garasi kendaraan darurat - Tempat perlindungan terhadap gempa bumi, tsunami, angin badai, dan tempat perlindungan darurat lainnya - Fasilitas kesiapan darurat, komunikasi, pusat operasi dan fasilitas lainnya untuk tanggap darurat - Pusat pembangkit energi dan fasilitas publik lainnya yang dibutuhkan pada saat keadaan darurat - Struktur tambahan (termasuk menara telekomunikasi, tangki penyimpanan bahan bakar, menara pendingin, struktur stasiun listrik, tangki air pemadam kebakaran atau struktur rumah atau struktur pendukung air atau material atau peralatan pemadam kebakaran) yang disyaratkan untuk beroperasi pada saat keadaan darurat Gedung dan nongedung yang dibutuhkan untuk mempertahankan fungsi struktur bangunan lain yang masuk ke dalam kategori risiko IV.	IV

Table 2. 18 Building and non-building risk categories (SNI 1762:2019) (continues)

b. Determine the earthquake priority factor (Ie)

The earthquake priority factor is determined based on the building risk category. In Table 2.19, the earthquake priority factor (Ie) is presented in accordance with SNI 1726:2019. In this work, the residential structure is included in the building risk category II so that the earthquake priority factor (Ie) is set at 1.50

Kategori risiko	Faktor keutamaan gempa, I_e
I atau II	1,0
III	1,25
IV	1,50

Table 2. 19 Earthquake priority factor

c. Determine the ground acceleration parameters (Ss and S1)

Soil acceleration parameters (Ss and S1) are influenced by soil properties at the project site. The values of Ss and S1 are used to determine the spectral response to the acceleration of the MCER earthquake at ground level, where Ss and S1 are respectively the parameters of the spectral response to the acceleration of the MCER earthquake which are mapped for a short period and a period of 1.0 second. In Table 2.6 and 2.7, respectively, the values of Ss and S1 are presented for the maximum considered risk-targeted earthquake (MCER) in bedrock. In this work, the location of the building is in the city of Yogyakarta, so the values of Ss = 1. 0619g and S1 = 0.4875g are used. It based on figure 2.16 and 2.17.

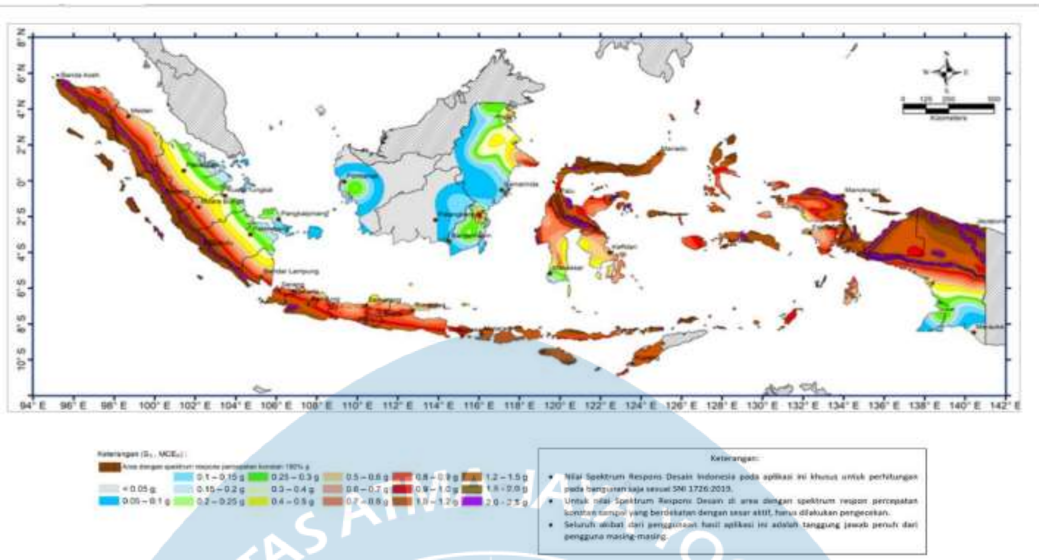


Figure 2. 16 Ss, maximum considered risk-targeted (MCER) earthquake on bedrock for a short period (0.2 seconds) (Source: SNI 1726:2019)

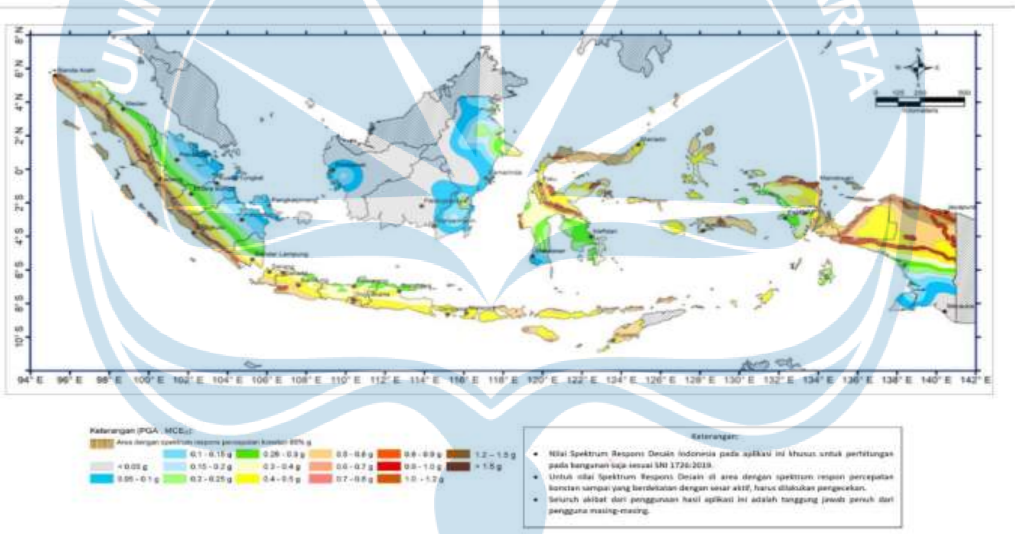


Figure 2. 17 S1 maximum considered target risk earthquake (MCER) on bedrock for a period of 1 second (Source: SNI 1726:2019)

2.5 Structure Modeling

Structural modeling is carried out to determine the internal forces that occur in structural elements and structural behavior due to workloads. The results of the structural modeling are used as the basis for designing the required cross-sectional dimensions of the structural elements.

2.5.1 Structural System

The structure of the Assalafiyyah Islamic Boarding School is designed using a special moment-bearing frame system (SRPMK) with columns and beams as a moment reinforcement.

2.5.2 Structural Model

The structural design process is carried out based on the internal forces that occur in the structural elements due to the working ultimate load. This internal force can be obtained through structural modeling. Structural modeling is carried out in a three-dimensional model (3D model). In structural modeling, truss elements are used to idealize beams and columns. While the floor slab is idealized as a deck.

2.5.2.1 Use Limit Performance

Service limit performance is evaluated based on a combination of service loads. One of the service limit parameters that will be evaluated is the deviation between floors due to the influence of the design earthquake for each of the orthogonal axes of the structure. The deviation between floors that occurs must be smaller than the allowable deviation between floors with the aim of limiting the occurrence of excessive steel melting and concrete cracking, preventing non-structural damage, and preventing excessive deviations that cause discomfort for building occupants

2.5.2.2 Ultimate Limit Performance

At the ultimate limit performance, the ultimate load combination is used to analyze the internal forces that occur in the structural elements. These internal forces are then used in the design process of structural elements such as slabs, beams, columns, foundations, etc. The ultimate limit performance will determine the safety of the structure in supporting the ultimate design load acting on the structure.

Structural Material Specification

The specifications of the materials used in the structural design are presented as follows:

1. Profile steel

▪ The profile steel used is BJ 37 with yield stress, fy = 240 MPa and ultimate

stress, fu = 370 MPa

▪ Steel's modulus of elasticity, Es = 200.000 MPa

2. Concrete

▪ The compressive strength of concrete at the age of 28 days, fc' = 25 MPa (bottom structure)

▪ Modulus of elasticity of concrete, Ec = 4700 √ fc' = 23500 MPa

3. Reinforcing steel

▪ Reinforcement steel with D > 12 mm, used deformed steel with yield stress , fy

= 420 MPa

▪ Reinforcement steel with D ≤ 12 mm, plain reinforcing steel with yield

stress is used, fy = 235 MPa

▪ Steel's modulus of elasticity, Es = 200.000 MPa

2.5.2.3 Structure Modeling Using ETABS application

The structural model is carried out with several idealizations. For example, floor slabs are idealized as shell elements, while beams and columns are idealized as truss elements. The structural modeling carried out is able to accommodate the effects of steel damage during an earthquake, namely by reducing the moment of inertia of the cross-section of the structural elements.

The moment of inertia of the plate is reduced to 25% of the initial moment of inertia. In beam structural elements, the moment of inertia is reduced to 35% of the initial moment of inertia. In addition, the torque is also reduced by 25% to balance the reduction value against the inertia of the structural elements. Whereas in the column, the moment of inertia is reduced to 70% of the initial moment of inertia. The structure of the restaurant is designed using a structural system in the form of a special moment resisting frame structure. The structure is modeled in 3D models (3D Models) using software assistance (see Figure 2.25).

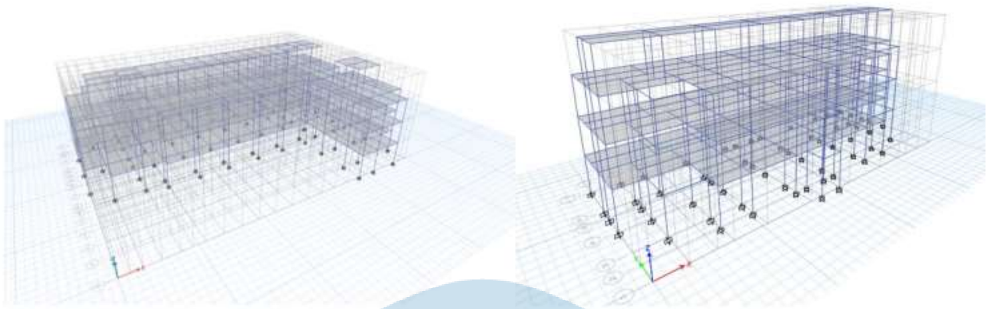


Figure 2. 18 Dormitory and Educational building structure model Building A (left) and Building B (right).

- **Definition of Material**

The materials used in the structural analysis are as follows:

Table 2.20 Material Specification

Material	fy	fu	fc'
Steel	240 MPa	370 MPa	-
Concrete (K300)	-	-	25 MPa
Steel Reinforcement	420 MPa	-	-
Plain Steel Reinforcement	240 MPa	-	-

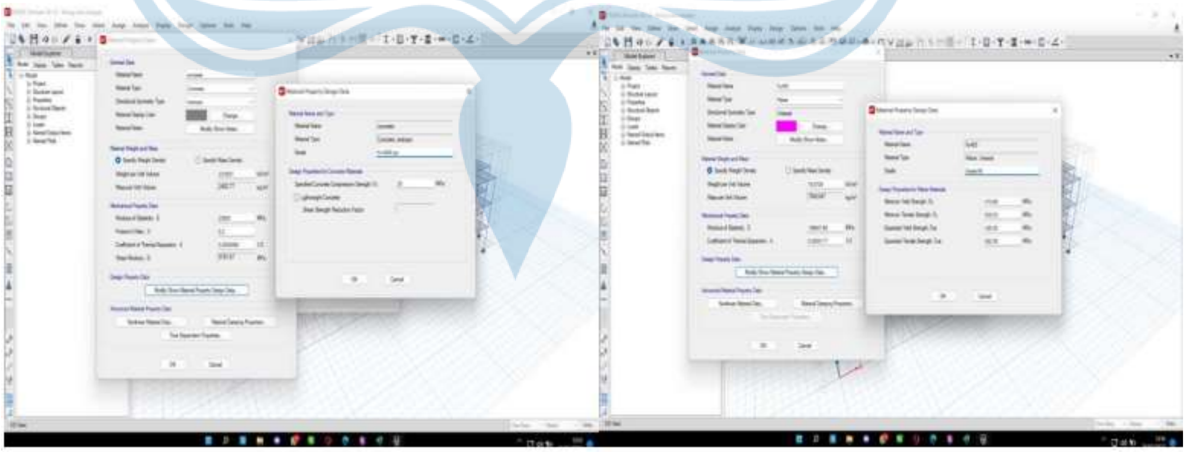


Figure 2. 19 Material Properties

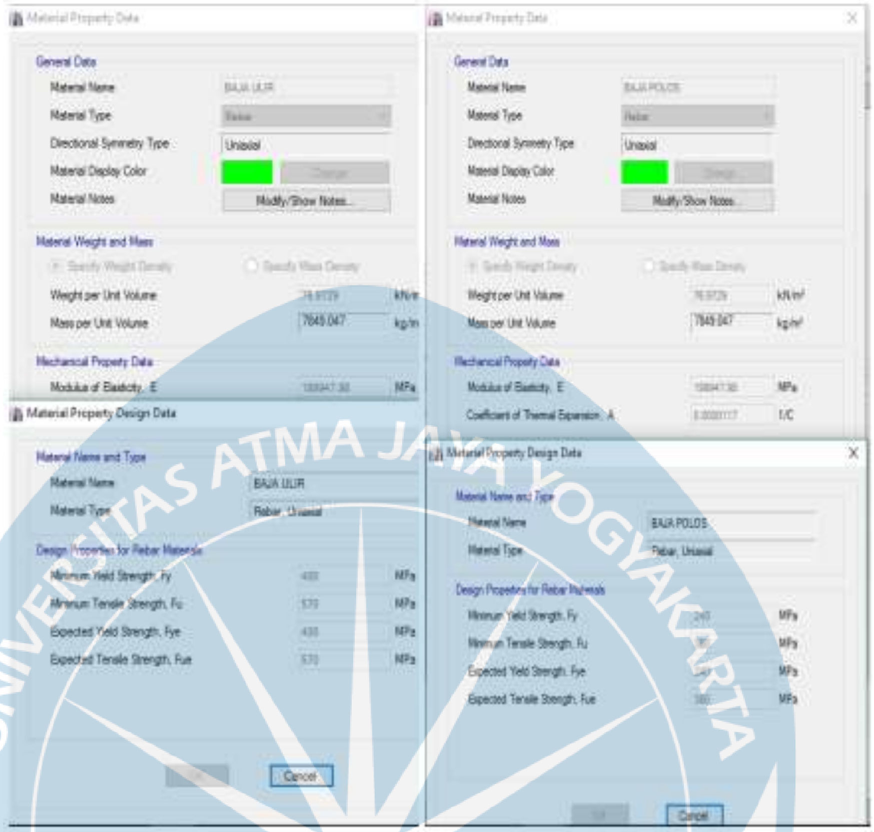


Figure 2. 20 Material Properties

- **Definition of Beam and Column Profile**

The beam and column cross sections are defined as follows

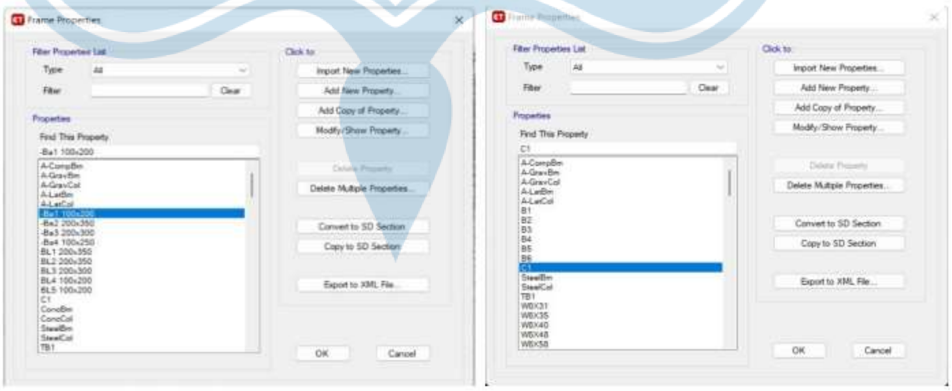


Figure 2. 21 Section Properties

Example of Column Section Properties (H 300)

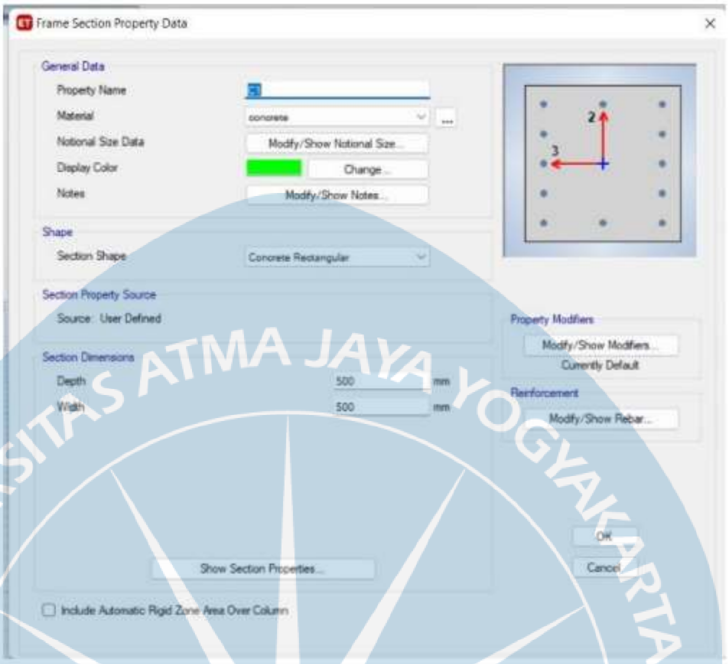


Figure 2. 22 Section Properties Column K2

- **3D modeling of structures**

After the material and section properties are complete, the next step is to create a 3D model. The model accommodates all sizes of beams and columns, along with the reinforcement planned to be installed as shown in figure 2.29.

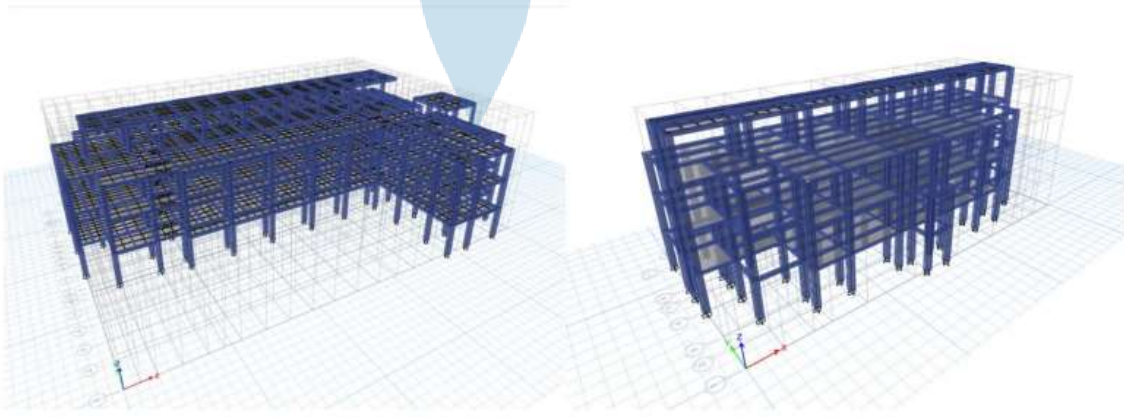


Figure 2. 23 3D Model of Building A (left) and Building B (right)

- **Giving Workload**

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

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

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

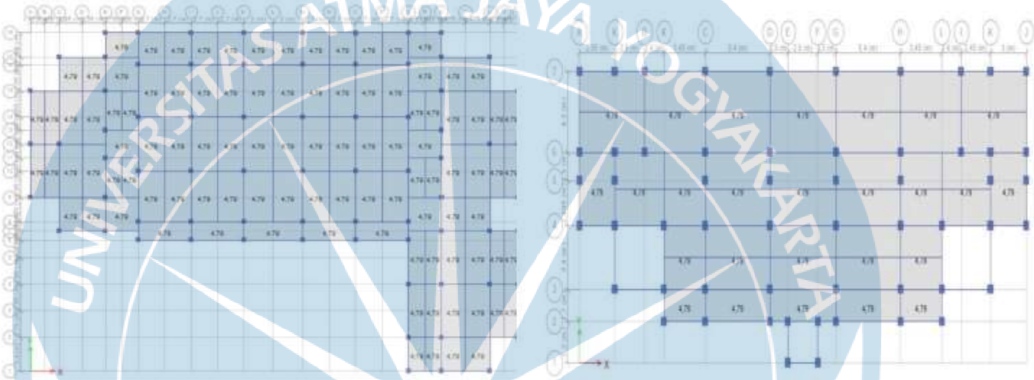


Figure 2. 24 Loading of Floor Plates in 3D Model

- **Giving Earthquake Burden**

Earthquake load is modeled in the program with the response spectrum function. Calculations and quantities can be seen in the Input data - response spectrum section. After obtaining the spectrum response graph, the graph is then input into the program, as shown in Figure 2.32.

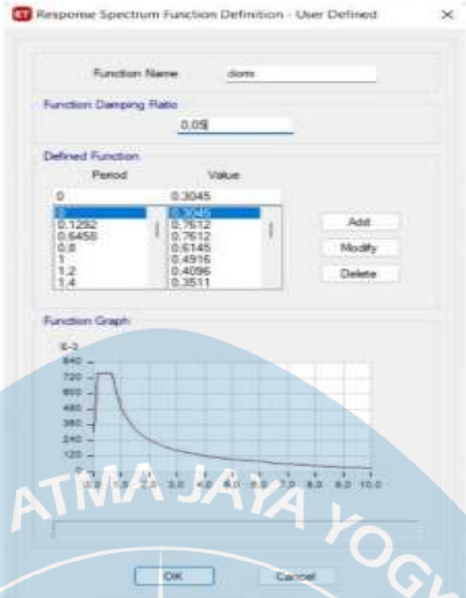


Figure 2. 25 Earthquake Load Input

After the earthquake load has entered, a combination of loading is carried out which allows several extreme loads to work together.

- **Providing Combinations and Loading Factors**

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

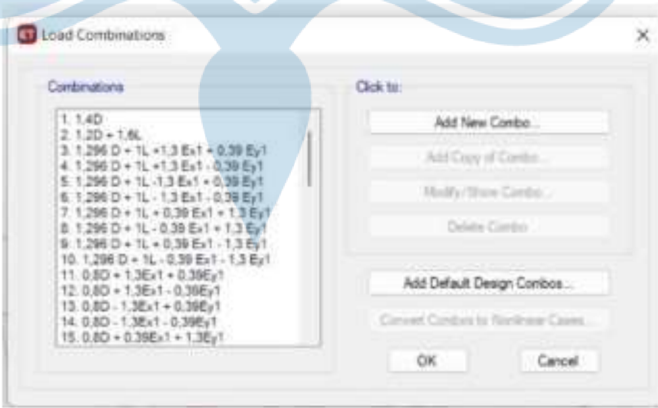


Figure 2. 26 Load Combination Input

2.6 Interpretation of Modeling Output

After all the forces are installed, several treatments on the structure are carried out such as giving a mass source and a diaphragm, after which the program is run. The result of running the program is in the form of internal forces acting on the beams and columns of the structure. This force is the key in analyzing the strength of the structure itself. The force obtained in the running results can be seen in Figure 2.34

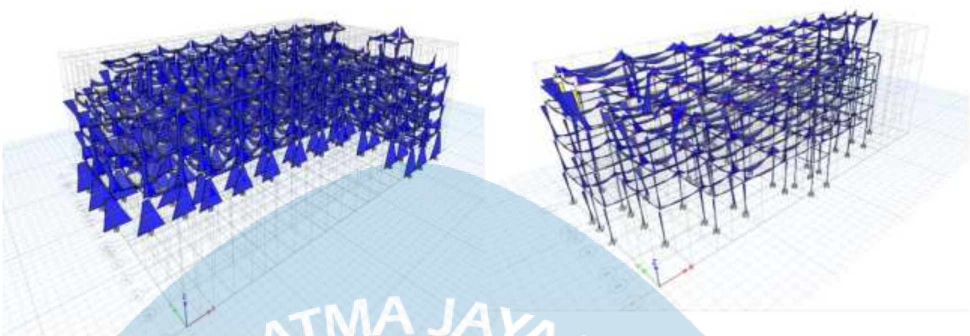


Figure 2. 27 Working Force, Running Results

2.6.1 Inner Style Results

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

2.6.2 Structure Behavior Check

2.6.2.1 Structural Fundamental Period Check

In this work, the type of structure used is a moment-bearing steel frame so that the value of . is obtained Ct = 0.0724 dan x = 0.8. Next based on the value SD1 = 0.589 g get coefficient Cu = 1.4. So that the value of Ta = 0.926 seconds dan Cu.Ta = 1.297 seconds. Structural modeling result period value, Tc = 0.713 seconds (Ta < Tc < Cu.Ta). So the period of the structure used is T = 0.926 seconds.

2.6.2.2 Checking Modal Participation Mass Ratio

Based on the results of structural modeling, the capital participation mass ratio is presented in Table 2.17. The number of modes (modes) required to determine the natural vibrational variation for the structure must be sufficient to obtain a combined mass participation of at least 90% of the actual mass of each orthogonal horizontal direction of the response considered by the model, in accordance with Article 7.9.1 of SNI 1726:2019.Based on the results of structural modeling, it is found that in both directions involving 582 modes of vibration, it is sufficient to produce more than 90% of the actual mass in both X and Y directions (see Table 2.35).

(Building A)

Case	Mode	Sum UX	Sum UY
Modal	1	0.7732	0.0002
Modal	2	0.7739	0.7722
Modal	3	0.783	0.7886
Modal	4	0.8811	0.7886
Modal	5	0.8811	0.8807
Modal	6	0.8813	0.8814
Modal	7	0.883	0.8816
Modal	8	0.8843	0.8816
Modal	9	0.8843	0.8816
Modal	10	0.956	0.8837
Modal	11	0.9621	0.9284
Modal	12	0.9625	0.9642
Modal	13	0.9626	0.9642
Modal	14	0.9626	0.9642
Modal	15	0.9627	0.9643
Modal	16	0.9992	0.9643
Modal	17	0.9992	0.9993
Modal	18	0.9998	0.9998

Table 2.21 Capital Participation Mass Ratio

(Building B)

Case	Mode	Sum UX	Sum UY
Modal	1	0,2636	0,0003
Modal	2	0,8388	0,0032
Modal	3	0,8401	0,8379
Modal	4	0,8428	0,8386
Modal	5	0,8630	0,9016
Modal	6	0,9290	0,9196
Modal	7	0,9290	0,9652
Modal	8	0,9329	0,9656
Modal	9	0,9812	0,9656
Modal	10	0,9812	0,9812
Modal	11	0,9856	0,9813
Modal	12	0,9857	0,9988
Modal	13	0,9997	0,9988
Modal	14	0,9997	0,9997
Modal	15	0,9997	0,9999
Modal	16	0,9997	0,9999
Modal	17	0,9997	1
Modal	18	0,9999	1

Table 2.22 Capital Participation Mass Ratio (continue)

2.6.2.3 Basic Sliding Style Check (Base Shear)

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

2.6.2.4 Eccentricity Check

The building structure for all seismic design categories (KDS) must consider the design torque and the unexpected torque. Unexpected torque is applied to the structural model by providing 5% eccentricity in the X and Y axes respectively. In this work, the building structure belongs to the seismic design category D so that the design torque and unexpected torque must be considered. Furthermore, if the building structure belongs to the seismic design category C, D, E, and F and has torsional irregularities 1a and 1b, the unexpected torsional enlargement must be considered. In this work, there are no torsional irregularities 1a and 1b in the building structure under consideration so that the unexpected torsional enlargement can be neglected.

2.6.3. Inter-Story Deviation

Story response of the 3-storey building with SRPMK concrete structure type, with design force deflection (δxe) which can be seen in the *Story Response table* (ETABS) as shown on figure 2.49.

EX1

TABLE: Story Response

Story	Elevation	Location	X-Dir	Y-Dir
	m		mm	mm
Story4	16	Top	27,219575	1,7796537
Story3	12	Top	25,48542	1,8778659
Story2	8	Top	22,255321	1,7448131
Story1	4	Top	17,18271	1,5176257
Base	0	Top	0	0

EY1

TABLE: Story Response

Story	Elevation	Location	X-Dir	Y-Dir
	m		mm	mm
Story4	16	Top	3,7321475	39,74215
Story3	12	Top	5,1070949	38,931862
Story2	8	Top	4,7798705	36,964988
Story1	4	Top	4,1996872	33,817119
Base	0	Top	0	0

Table 2.23 Force Delfection

The allowable deviation between levels / story can be seen in figure 2.50.

Struktur	**Kategori**		
	I atau II	III	IV
Struktur, selain dari struktur dinding geser batu bata, 4 tingkat atau kurang dengan dinding interior, partisi, langit-langit dan sistem dinding eksterior yang telah didesain untuk mengakomodasi simpangan antar tingkat.	$0{,}025h_{sx}^{c}$	$0{,}020h_{sx}$	$0{,}015h_{sx}$
Struktur dinding geser kantilever batu bata	$0{,}010h_{sx}$	$0{,}010h_{sx}$	$0{,}010h_{sx}$
Struktur dinding geser batu bata lainnya	$0{,}007h_{sx}$	$0{,}007h_{sx}$	$0{,}007h_{sx}$
Semua struktur lainnya	$0{,}020h_{sx}$	$0{,}015h_{sx}$	$0{,}010h_{sx}$

Table 2.24 Table of SNI Savings Between Permit Levels

Calculations on the deviation between story can use the equation below:

$$\delta_x = \frac{C_d\delta_{Xe}}{I_e} = \frac{5.5\delta_{Xe}}{1.5} = 5.5\delta_{XE} \qquad (2.1)$$

$$\frac{\Delta_a}{\rho} = \frac{0.015h}{1.3} = 0.0115h \qquad (2.2)$$

EX

Story	hx	h	γxe	Δ	Δi	ΔAllowable	Desc.
	mm	mm	mm	mm	mm	mm	
Roof	16000	4000	149,708	9,50068	0,60529	46,1538462	OK
3rd Story	12000	4000	140,17	8,89539	1,12743	46,1538462	OK
2nd Story	8000	4000	122,404	7,76796	1,77054	46,1538462	OK
1st Story	4000	4000	94,5049	5,99743	5,99743	46,1538462	OK
Base	-	-	-	-	-	-	-

EY

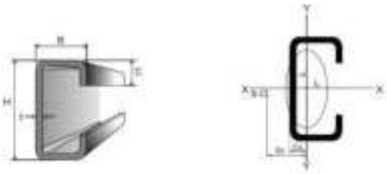
Story	hx	h	γxe	Δ	Δi	ΔAllowable	Desc.
	mm	mm	mm	mm	mm	mm	
Roof	16000	4000	218,582	13,8715	0,28282	46,1538462	OK
3rd Story	12000	4000	214,125	13,5887	0,68651	46,1538462	OK
2nd Story	8000	4000	203,307	12,9022	1,09873	46,1538462	OK
1st Story	4000	4000	185,994	11,8035	11,8035	46,1538462	OK
Base	-	-	-	-	-	-	-

Table 2.25 Result Recapitulation of Inter-Story Deviation

Based on the calculation above, it can be concluded that the deviation that occurs in this 3-storey structure does not exceed the permit deviation.

2.7 Roof Structure Design

2.7.1 Dormitory Roof



METRIC SIZE

DIMENSION	THICKNESS	SECTION AREA	WEIGHT UNIT	GEOMETRICAL MOMENT OF INERTIA		MODULUS OF SECTION		RADIUS OF GYRATION		CENTER OF GRAVITY	SHEAR CENTER	TORSION CONSTANT	WARPING CONSTANT
H x B x C	t	A	kg/m	Ix	Iy	Zx	Zy	rx	ry	Cy	Xo	J	Cw
mm	mm	cm^2		cm^4	cm^4	cm^3	cm^3	cm	cm	cm	cm	cm^4	cm^6
C 100 x 50 x 20	2.0	4.54	3.56	71	17	14.3	5.4	3.97	1.93	1.67	4.48	605	444
	2.2	5.17	4.06	81	19	16.1	6.0	3.95	1.92	1.66	4.46	912	496
	2.5	5.59	4.39	87	20	17.2	6.5	3.94	1.90	1.86	4.45	1164	528
	2.8	6.20	4.87	96	22	19.1	7.1	3.92	1.89	1.86	4.42	1621	574
	3.0	6.61	5.19	101	23	20.2	7.4	3.91	1.88	1.86	4.41	1962	603
	3.2	7.01	5.50	106	24	21.3	7.8	3.90	1.87	1.86	4.40	2362	630
C 125 x 50 x 20	2.0	5.04	3.95	120	18	19.3	5.5	4.89	1.91	1.69	4.15	672	675
	2.3	5.75	4.51	136	21	21.8	6.2	4.87	1.89	1.69	4.12	1013	756
	2.5	6.21	4.88	147	22	23.5	6.6	4.86	1.88	1.69	4.11	1295	806
	2.8	6.90	5.42	162	24	25.9	7.2	4.84	1.88	1.69	4.08	1804	877
	3.0	7.36	5.78	172	25	27.5	7.6	4.83	1.85	1.68	4.07	2207	922
	3.2	7.81	6.13	181	27	29.0	8.0	4.82	1.84	1.68	4.05	2660	965
C 150 x 50 x 20	2.0	5.54	4.35	185	18	24.7	5.6	5.79	1.87	1.55	3.86	738	971
	2.3	6.32	4.96	210	22	28.0	6.3	5.77	1.88	1.55	3.84	1115	1088
	2.5	6.84	5.37	228	23	30.2	6.8	5.75	1.85	1.55	3.82	1425	1162
	2.8	7.60	5.97	250	26	33.3	7.4	5.73	1.83	1.54	3.80	1987	1267
	3.0	8.11	6.37	265	27	35.4	7.8	5.72	1.82	1.54	3.78	2432	1334
	3.2	8.61	6.76	280	28	37.4	8.2	5.71	1.81	1.54	3.77	2938	1398
C 150 x 65 x 20	2.0	6.14	4.82	218	36	29.1	8.3	5.98	2.43	2.12	5.19	818	1784
	2.3	7.01	5.50	248	41	33.0	9.4	5.94	2.42	2.12	5.16	1236	2006
	2.5	7.59	5.96	267	44	35.6	10.0	5.93	2.41	2.12	5.15	1581	2148
	2.8	8.44	6.63	295	48	39.4	11.0	5.91	2.39	2.12	5.13	2207	2352
	3.0	9.01	7.07	314	51	41.8	11.6	5.90	2.38	2.11	5.11	2702	2482
	3.2	9.57	7.51	332	54	44.2	12.2	5.89	2.37	2.11	5.09	3268	2608
C 200 x 75 x 20	2.0	7.54	5.92	467	56	46.7	10.6	7.87	2.73	2.20	5.48	1003	4571
	2.3	8.62	6.77	531	64	53.1	12.0	7.85	2.72	2.20	5.47	1520	5189
	2.5	9.34	7.33	573	68	57.3	12.9	7.84	2.71	2.20	5.45	1946	5537
	2.8	10.40	8.17	636	75	63.6	14.2	7.82	2.69	2.20	5.42	2719	6085
	3.0	11.11	8.72	676	80	67.6	15.0	7.80	2.68	2.19	5.41	3332	6437
	3.2	11.81	9.27	716	84	71.6	15.8	7.79	2.67	2.19	5.39	4030	6779

Table 2. 26 Profile Canal-C

- **Specification**
    - C Channel Profile : C 200 x 75 x 20
    - Thickness : 3.0 mm
    - Section Area : 11.11 cm^2
    - Unit Weight : 8.72 kg/ m
    - Ix : 6760000 mm^4
    - Iy : 800000 mm^4
    - Zx (W3) : 67600 mm^3
    - Zy (W2) : 15000 mm^3
    - Bitumen roof mass : 10 kg
    - Ceiling mass : 20 kg
    - Truss Spacing : 1200 mm = 1.2 meter
    - Purlin Spacing : 1.74 meter
    - α : 50˚

- Ø : 0.9
- E : 200000
- Truss Weight : 79.849 kg

2.7.1.1 Gording Design Plan

1. Gording Load

The calculation of the curtain load includes self-weight, roof weight, and ceiling weight so that the dead load (D) of the gording moment will be obtained. Own weight is taken as 8.72 kg/m. The calculation of the weight of the roof is calculated using the formula:

Roof weight $= \frac{Purlin\ Spacing}{Cos\ a}\ x\ Bitumen\ Roof\ Weight$

$= \frac{1.74}{Cos\ 50}$ x 0.13

= 0.3519 kN/m

The calculation of ceiling weight is calculated using the formula:

Ceiling's weight $= Purlin\ Spacing\ \times\ ceiling\ weight$

= 1.74 x 0.2

= 0.348 kN/m

The calculation of Dead Load (D) is calculated using the formula:

Dead Load $= Weight\ of\ Gording\ +\ Roof\ +\ Ceiling$

= 0.0855 + 0.3519 + 0.384

= 0.785 kN/m

The live load (P) is taken 1.0 kN.

2. Gording Moment Plan

The calculation of the gording moment plan includes the gording loads in the 2nd and 3rd axes directions. The gording load in the 2nd axis direction of this building uses the formula:

$M_{2,D}$ $= \frac{1}{8}\ q \sin \propto \left(\frac{L}{2}\right)^2$

$= \frac{1}{8} x\ 0.785 \sin 40 \left(\frac{1.2}{3}\right)^2$

= 0.0101 kN/m

$M_{2,L}$ $= \frac{1}{4}\ P \sin \propto \left(\frac{L}{2}\right)$

$= \frac{1}{4}\ 1 \sin 40 \propto \left(\frac{1.2}{2}\right)$

Gording Moment Plan

$M_{3,D}$ $= \frac{1}{8} \times q \times \cos a \times L2$

$= 1/8 \text{ x } 0.785 \text{ x COS } 50 \text{ x } 1.2^2$

= 0.0909 knm

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

= 1/4 x 1 x COS 50 x 1.2

= 0.193 knm

$M_{2,D}$ $= \frac{1}{8} \times q \times \sin a \times \frac{L}{32}$

$= 1/8 \text{ x } 0.785 \text{ x sin } 40 \text{ x } 1.2/3^2$

= 0.0101 knm

$M_{2,L}$ $= \frac{1}{4} \times P \times \sin a \times L/3$

= 1/4 x 1 x sin 40 x 1.2/ 3

= 0.0643

$M_{3,U}$ $= 1.4\ M3, D$

= 1.4 x 0.0909

= 0.127 knm

$M_{3,U}$ $= 1.2\ M3, D + 1.6\ M3, L$

= 1.2 x 0.0909 + 1.6 x 0.193

= 0.418 knm

Choose the big one that is 0.418 knm

$M_{2,U}$ $= 1.4\ M2, D$

= 1.4 x 0.0101

= 0.014

$M_{2,U}$ $= 1.2\ M2, D + 1.6\ M2, L$

= 1.2 x 0.0101 + 1.6 x 0.0643

= 0.115

Choose the big one that is 0.115 knm

Stress

$fb = \frac{\text{M3,U}}{\text{ØW3}} + \frac{\text{M2,U}}{\text{ØWw}} \leq Fy\ with\ value$ Ø = 0.9

= 0.418 / (0.9 x 67600) + 0.115 / (0.9 x 15000)

= 0.0000153794 x 1000000

= 15.37939577 (because 111.439 ≤ 240 MPa, the C profile stress is safe)

Gording Deflection Check

$$\delta\ 2 = \frac{5}{384}\ X\ \frac{q\cos\ \alpha\ L^4}{EI} + \frac{1}{48} + \frac{P\cos\alpha\ L^3}{EI}$$

δ 2 = 5/384 x (0.785 x cos 50^o x 1200^4 / 200000 x 6760000) +

(1/48) x (1 x cos 50^o) x (1200^3) / 20000 x 6760000

= 0.0101

$$\delta\ 3 = \frac{5}{384}\ X\ \frac{q\sin\ \alpha}{EI} + (\frac{1}{3})^4 + \frac{1}{8}\frac{P\sin\alpha}{EI} + (\frac{L}{3})^3$$

δ 3 = 5/384 x (0.785 x sin 50^o)/ (200000 x 800000) x (1200/3)^4 + (1/48 x 1 x sin 50) / (200000 x 800000) x (1200/3)^3

= 0.0013

δ = $\sqrt{\delta\ 2^2 + \delta\ 3^2} \leq \frac{1}{240}L$

δ = √0.0101^2 + 0,0013^2

= 0.0102 ≤ 5.000

because the gording deflection is 2.0618 ≤ 12.5 then the gording deflection is safe.

2.7.1.2 Sagrod Design Plan

Number of gording (n) under nok = 4

$F_{t,D}$ = n($\frac{L}{3} \times\ q \times \sin\alpha$)

= 4(1.2/3 x 0.785 x sin 50)

= 0.963 kN

$F_{t,L}$ = $\frac{n}{2} \times P \times \sin\alpha$

= 4/2 x 1 x sin 50

= 2 kN

Load Combination

$F_{t,U}$ = 1.4 Ft, D

= 1.4 x 0.963

= 0.963 kN

$F_{t,U}$ $= 1.2\,Ft,D \;+\; 1.6\,Ft,D$

= (1.2 x 0.963) + (1.6 x 2)

= 3.6065 kN

selected $F_{t.U}$ = 3.6065 kN

Sagrod Bar Area

Asr $= \frac{Ft.10^3}{\emptyset Fy}$

$= \frac{3.6065\ \text{x}10^3}{0.9\ \text{x}\ 240}$

= 16,6969 kN

2.7.1.3 Truss Load Plan

1. Load P1

Truss own weight $= \frac{\alpha}{2} \times weight\ truss$

= 2/2 x 0.5

= 0.5 kN

Gording Weight $= L\ \times gording\ weight\ per\ m'$

= 4 x 0.0855

= 0.34 kN

Roof Weight $= \frac{\frac{a}{2}+b}{\cos\alpha} \times L \times roof\ weight$

= 2/2+cos 50x1.2x0,3519

= 1.4171 kN

Ceiling Weight $= \left(\frac{a}{b} + b\right) \times L \times ceiling\ weight$

= (2/2 +1.1570) x 1.2 x 0.348

= 0.90077059 kN

Total **(Load P1)** = 3.159844694 kN

2. Load P2

Truss own weight $= a\ \times\ truss\ weight$

= 2 x 0.5

= 1 kN

Gording Weight $= L\ \times gording\ weight\ per\ m'$

= 1.2 x 0.0855

= 0.10 kN

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

= (2/cos 50) x 1.2 x 0.3519

= 1.313919774 kN

Ceiling Weight = $a \times L \times ceiling\ weight$

= 2 x 1.2 x 0.348

= 0.8352 kN

Total **(Load P2)** = 3.251719774 kN

3. Load P3

Truss own weight = $a \times truss\ weight$

= 2 x 0.5

= 1 kN

Gording Weight = $2 \times L \times gording\ weight\ per\ m'$

= 2 x 1.2 x 0.855

= 0.21 kN

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

= (2/cos 50) x 1.2 x 0.3519

= 1.313919774 kN

Ceiling Weight = $a \times L \times ceiling\ weight$

= 2 x 1.2 x 0.348

= 0.8352 kN

Total **(Load P3)** = 3.354319774 kN

Wind Load

Load W1 $= \frac{\left(\frac{a}{2}+b\right)}{\cos a} \times Cti \times L \times Qw$

$= \frac{\left(\frac{2}{2}+1\right)}{\cos 50}$ x 0.4 x 1.2 x 0.25

= 0.3734 kN

Load W2 $= \frac{a}{\cos a} \times Cti \times L \times Qw$

$= \frac{2}{\cos 50}$ x 0.4 x 1.2 x 0.25

= 0.3133 kN

Load W3 $= \frac{1}{2}\frac{a}{\cos} \times Cti \times L \times Qw$

$= \frac{1}{2}$ x $\frac{2}{\cos 50}$ x 0.4 x 1.2 x 0.25

= 0.1566 kN

Load W4 $= \frac{1}{2}\frac{a}{\cos a} \times Cis \times L \times Qw$

$= \frac{1}{2}$ x $\frac{2}{\cos 50}$ x (-0.6) x 1.2 x 0.25

= - 0.2350 kN

Load W5 $= \frac{a}{\cos a} \times Cis \times L \times Qw$

$= \frac{2}{\cos 50}$ x (-0.6) x 1.2 x 0.25

= - 0.4699 kN

Load W6 $= \frac{\left(\frac{a}{2}+b\right)}{\cos a} \times Cis \times L \times Qw$

$= \frac{\frac{2}{2}+1}{\cos 50}$ x (-0.6) x 1.2 x 0.25

= -0.3831 kN

2.7.2 Dormitory Second Roof

- **Spesification**
  - C Channel Profile : C 200 x 75 x 20
  - Thickness : 2
  - Area :
  - Unit Weight : 8.72
  - Ix : 6760000 mm^4
  - Iy : 800000 mm^4
  - Zx (W3) : 67600 mm^3
  - Zy (W2) : 15000 mm^3
  - Bitumen Roof Weight : 0.13 kN/m
  - Length : 2 m
  - Purlin Spacing : 1.74 m
  - α : 35˚
  - Ø : 0.9
  - L : 1.2 m
  - E : 200000

2.7.2.1 Gording Design Plan

Gording's weight = 0.0855knm

Roof's weight $= \frac{Purlin\ Spacing}{\cos a}\ x\ Bitumen\ Roof\ Weight$

= 2/ COS 35 x 0.49

= 1.1964 knm

Ceiling's weight = $Purlin\ Spacing\ x$ 0.2

= 2 x 0.2

= 0.4 knm

Dead Load (D) plan gording q

= $Weight\ of\ Gording\ +\ Roof\ +\ Ceiling$

= 0.0855 + 1.1964 + 4

= 1.682 knm

Live Load (P) = 1 knm

GORDING MOMENT PLAN

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

= 1/8 x 1.682 x COS 35 x 4^2

= 0.2480 knm

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

= 1/4 x 1 x COS 35 x 1.2

= 0.246 knm

$M_{2,D}$ $= \frac{1}{8} \times q \times sin\ a \times \frac{L^2}{3}$

= 1/8 x 1.682 x sin 35 x $1.2/3^2$

= 0.0193 knm

$M_{2,L}$ $= \frac{1}{4} \times P \times sin\ a \times \frac{L}{3}$

= 1/4 x 1 x sin 35 x 1.2/ 3

= 0.0574 knm

$M_{3,U}$ = 1.4 $M3, D$

= 1.4 x 0.2480

= 0.347 knm

$M_{3,U}$ = 1.2 $M3, D$ + 1.6 $M3, L$

= 1.2 x 0.2480 + 1.6 x 0.246

= 0.691 knm

Choose the big one that is 0.691 knm

$M_{2,U}$ $= 1.4\, M2, D$

$= 1.4 \times 0.0193$

$= 0.027$

$M_{2,U}$ $= 1.2\, M2, D + 1.6\, M2, L$

$= 1.2 \times 0.0193 + 1.6 \times 0.0574$

$= 0.115$

Choose the big one that is 0.115 knm

STRESS

$$fb = \frac{\mathrm{M3,U}}{\mathrm{ØW3}} + \frac{\mathrm{M2,U}}{\mathrm{ØWw}} \leq Fy\ with\ value\ Ø = 0.9$$

= 0.691 / (0.9 x 67600) + 0.115 / (0.9 x 15000)

= 0.0000198669 x 1000000

= 19.86689493 (because 220.8624 ≤ 240 MPa, the C profile stress is safe)

GORDING DEFLECTION CHECK

$$\delta\ 2 = \frac{5}{384} \times \frac{q \cos\ \alpha\ L^4}{EI} + \frac{1}{48} + \frac{P \cos \alpha\ L^3}{EI}$$

δ 2 = 5/384 x (1.682 x cos 35° x 4000^4 / 200000 x 6760000) +

(1/48) x (1 x cos 35°) x (4000^3) / 20000 x 6760000

= 3.3975

$$\delta\ 3 = \frac{5}{384}\ X\ \frac{q \sin\ \alpha}{EI} + (\frac{1}{3})^4 + \frac{1}{8}\frac{P \sin \alpha}{EI} + (\frac{L}{3})^3$$

δ 3 = 5/384 x (1.682 x sin 35°)/ (200000 x 800000) x $(4000/3)^4$ + (1/48 x 1 x sin 35) / (200000 x 800000) x $(4000/3)^3$

= 1.2567

δ $= \sqrt{\delta\ 2^2 + \delta\ 3^2} \leq \frac{1}{240} L$

δ = $\sqrt{}1.2567^2 + 1{,}2567^2$

= 3.6225 ≤ 16.667 because the gording deflection is 7.5087 ≤ 16.667 then the gording deflection is safe.

2.7.2.2 Sagrod Design Plan

Number of gording (n) under nok = 2

$F_{t,D}$ $= n(\frac{L}{3} \times\ q \times \sin\alpha)$

= 2(1.2/2 x 1.682 x sin 35)

= 1.158 kN

$F_{t,L}$ $= \frac{n}{2} \times P \times \sin\alpha$

= 2/2 x 1 x sin 35

= 1 kN

LOADING COMBINATION

$F_{t,U}$ $= 1.4\ Ft, D$

= 1.4 x 1.158

= 1.6207 kN

$F_{t,U}$ $= 1.2\ Ft, D\ +\ 1.6\ Ft, D$

= (1.2 x 1.158) + (1.6 x 1.158)

= 2.3069 kN

selected $F_{t.U}$ = 2.3069 kN

AREA SAGROD BAR

Asr $= \frac{Ft.10^3}{\emptyset Fy}$

= 2.3069 x 10^3/0.9 x 240

= 10.6799 kN

2.7.2.3 Truss Load Plan

Load P1 :

Truss own weight $= \frac{\alpha}{2} \times weight\ truss$

= 2/2 x 14

= 14 kN

Gording Weight $L\ \times gording\ weight\ per\ m' =$

= 4 x 0.0855

= 0.34 kN

Roof Weight $= \frac{\frac{\alpha}{2}+b}{\cos\alpha} \times L \times roof\ weight$

= 2/2+cos 35 x 1.2 x 1.1964

= 3.5052 kN

Ceiling Weight $=\left(\frac{a}{b}+b\right)\times L\times ceiling\ weight$

=(2/2 +1) x 1.2 x 0.4

= 0.96 kN

LOAD P1 = 18.80716 kN

Load P2

Truss own weight $= a\ \times\ truss\ weight$

= 2 x 14

= 28 kN

Gording Weight $= L\ \times gording\ weight\ per\ m'$

= 1.2 x 0.0855

= 0.10 kN

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

= (2 /cos 35) x 1.2 x 1.1964

= 3.505163483 kN

Ceiling Weight $= a\times L\times ceiling\ weight$

= 2 x 1.2 x 0.4

= 0.96 kN

LOAD P2 = 32.56776348 kN

Load P3

Truss own weight $= a\ \times\ truss\ weight$

= 2 x 14

= 28 kN

Gording Weight $= 2\times L\ \times gording\ weight\ per\ m'$

= 2 x 1.2 x 0.855

= 0.21 kN

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

= (2/cos 35) x 1.2 x 1.1964

= 3.505163483 kN

Ceiling Weight $= a\times L\times ceiling\ weight$

= 2 x 1.2 x 0.4

= 0.96 kN

LOAD P3 = 32.67036348 kN

WIND LOAD

Load W1 $= \frac{\left(\frac{a}{2}+b\right)}{\cos a} \times Cti \times L \times Qw$

= (2/2+1)/cos 50 x 0.4 x 1.2 x 0.25

= 0.3734 kN

Load W2 $= \frac{a}{\cos a} \times Cti \times L \times Qw$

= 2/cos 50 x 0.4 x 1.2 x 0.25

= 0.3133 kN

Load W3 $= \frac{1}{2}\frac{a}{\cos} \times Cti \times L \times Qw$

= ½ x 2/cos 50 x 0.4 x 1.2 x 0.25

= 0.1566 kN

Load W4 $= \frac{1}{2}\frac{a}{\cos a} \times Cis \times L \times Qw$

= = ½ x 2/cos 50 x (-0.6) x 1.2 x 0.25

= -0.2350 kN

Load W5 $= \frac{a}{\cos a} \times Cis \times L \times Qw$

= 2/cos 50 x (-0.6) x 1.2 x 0.25

= -0.4699 kN

Load W6 $= \frac{\left(\frac{a}{2}+b\right)}{\cos a} \times Cis \times L \times Qw$

= (2/2+1)/cos 50 x (-0.6) x 1.2 x 0.25

= -0.3831 kN

2.7.3 Educational Roof

- **Specification**

- Profil Kanal C	C 200 x 75 x 20	
- Thickness	2	
- Area	11.11	
- Unit Weight	8.72	
- Ix	6760000	mm^4
- Iy	800000	mm^4
- Zx (W3)	67600	mm^3
- Zy (W2)	15000	mm^3
- Bitumen Roof Weight	0.13	kN/m
- L	1200	mm
- Purlin Spacing	1.8	m
- α	40	degree
- Ø	0.9	
- L	1.2	m
- E	200000	

WEIGHT TRUSS

H = 3.6

Tilt = 7.2

Overstek = 1.8 x COS 40

= 1.378879998

= 7.2+ 1.378879998

= 8.578879998

= 1.378879998 x 8.578879998

= 74.80783358

2.7.3.1 Gording Design Plan

Gording's weight = 0.0855 knm

Roof's weight $= \frac{Purlin\ Spacing}{cos\ a} \times Bitumen\ Roof\ Weight$

= 1.74 / COS 40 x 0.13

= 0.3055 knm

Ceiling's weight $= Purlin\ Spacing\ x\ 0.2$

= 0.36 knm

Dead Load (D) plan gording q

= $Weight\ of\ Gording\ +\ Roof\ +\ Ceiling$

= 0.0855 + 0.3055 + 0.36

= 0.751 knm

Live Load (P) = 1 knm

GORDING MOMENT PLAN

$M_{3,D}$ = $\frac{1}{8} \times q \times cos\ a \times L2$

= 1/8 x 0.751 x COS 40 x 1.2^2

= 0.1035 knm

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

= 1/4 x 1 x COS 40 x 1.2

= 0.230 knm

$M_{2,D}$ = $\frac{1}{8} \times q \times sin\ a \times \frac{L}{3}^2$

= 1/8 x 0.751 x sin 40 x $1.2/3^2$

= 0.0097 knm

$M_{2,L}$ = $\frac{1}{4} \times P \times sin\ a \times \frac{L}{3}$

= 1/4 x 1 x sin 40 x 1.2/ 3

= 0.0643

$M_{3,U}$ = $1.4\ M3, D$

= 1.4 x 0.1035

= 0.145 knm

$M_{3,U}$ = $1.2\ M3, D\ +\ 1.6\ M3, L$

= 1.2 x 0.1035 + 1.6 x 0.230

= 0.492 knm

Choose the big one that is 0.492 knm

$M_{2,U}$ = $1.4\ M2, D$

= 1.4 x 0.0097

= 0.014

$M_{2,U}$ = $1.2\ M2, D\ +\ 1.6\ M2, L$

= 1.2 x 0.0097 + 1.6 x 0.0643

= 0.114

Choose the big one that is 0.114 knm

STRESS

$fb = \frac{\text{M3,U}}{\text{ØW3}} + \frac{\text{M2,U}}{\text{ØWw}} \leq Fy\ with\ value$ Ø = 0.9

= 0.492 / (0.9 x 67600) + 0.114 / (0.9 x 15000)

= 0.0000165625 x 1000000

= 16.56250853 (because 111.439 ≤ 240 MPa, the C profile stress is safe)

GORDING DEFLECTION CHECK

$$\delta\ 2 = \frac{5}{384}\ X\ \frac{q \cos\ \alpha\ L^4}{EI} + \frac{1}{48} + \frac{P \cos \alpha\ L^3}{EI}$$

δ 2 = 5/384 x (0.751 x cos 40° x 1200^4 / 200000 x 6760000) + (1/48) x (1 x cos 40°) x (1200^3) / 20000 x 6760000

= 0.0115

$$\delta\ 3 = \frac{5}{384} \times \frac{q \sin\ \alpha}{EI} + (\frac{1}{3})^4 + \frac{1}{8}\frac{P \sin \alpha}{EI} + (\frac{L}{3})^3$$

δ 3 = 5/384 x (0.751 x sin 40°)/ (200000 x 800000) x $(1200/3)^4$ + (1/48 x 1 x sin 40) / (200000 x 800000) x $(1200/3)^3$

= 0.0010

δ $= \sqrt{\delta\ 2^2 + \delta\ 3^2} \leq \frac{1}{240} L$

δ = √0.0010^2 + $0{,}0115^2$

= 0.0116 ≤ 5.000 because the gording deflection is 2.0618 ≤ 12.5 then the gording deflection is safe.

2.7.3.2 Sagrod Design Plan

Number of gording (n) under nok = 4

$F_{t,D}$ = n$(\frac{L}{3} \times\ q \times \sin \alpha)$

= 4(1.2/3 x 0.751 x sin 40)

= 0.772 kN

$F_{t,L}$ = $\frac{n}{2} \times P \times \sin \alpha$

= 4/2 x 1 x sin 40

= 1kN

LOADING COMBINATION

$F_{t,U}$ = 1.4 Ft, D

= 1.4 x 0.772

= 1.0813 kN

$F_{t,U}$ = 1.2 Ft, D + 1.6 Ft, D

= (1.2 x 0.772) + (1.6 x 1)

= 2.9837 kN

selected $F_{t.U}$ = 2.9837 kN

AREA SAGROD BAR

Asr = $\frac{Ft.10^3}{\emptyset Fy}$

= 2.9837 x 10^3/0.9 x 240

= 138135 kN

2.7.3.3 Truss Load Plan

Load P1 :

Truss own weight = $\frac{\alpha}{2} \times weight\ truss$

= 2/2 x 0.5

= 0.5 kN

Gording Weight $L \times gording\ weight\ per\ m'$

= 4 x 0.0855

= 0.34 kN

Roof Weight = $\frac{\frac{a}{2}+b}{\cos\alpha} \times L \times roof\ weight$

= 2/2+ 1.37888 x cos 40 x 1.2 x 0.3055

= 1.1383 kN

Ceiling Weight =$\left(\frac{a}{b}+b\right) \times L \times ceiling\ weight$

=(2/2 +1.37888) x 1.2 x 0.36

= 1.027676159 kN

LOAD P1 = 3.007989183 Kn

Load P2

Truss own weight = $a \times truss\ weight$

= 2 x 0.5

= 1 kN

Gording Weight = $L \times gording\ weight\ per\ m'$

= 1.2 x 0.0855

= 0.10 kN

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

= (2/cos 40) x 1.2 x 0.3055

= 0.957015928 kN

Ceiling Weight = $a \times L \times ceiling\ weight$

= 2 x 1.2 x 0.36

= 0.864 kN

LOAD P2 = 2.923615928 kN

Load P3

Truss own weight = $a \times truss\ weight$

= 2 x 0.5

= 1 kN

Gording Weight = $2 \times L \times gording\ weight\ per\ m'$

= 2 x 1.2 x 0.855

= 0.21 kN

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

= (2/cos 40) x 1.2 x 0.3055

= 0.957015928 kN

Ceiling Weight = $a \times L \times ceiling\ weight$

= 2 x 1.2 x 0.36

= 0.864 kN

LOAD P3 = 3.026215928 kN

WIND LOAD

Load W1 = $\frac{\left(\frac{a}{2}+b\right)}{\cos a} \times Cti \times L \times Qw$

= (2/2+1)/cos 40 x 0.4 x 1.2 x 0.25

= 0.3133 kN

Load W2 $= \frac{a}{\cos a} \times Cti \times L \times Qw$

= 2/cos 40 x 0.4 x 1.2 x 0.25

= 0.3133 kN

Load W3 $= \frac{1}{2}\frac{a}{cos} \times Cti \times L \times Qw$

= ½ x 2/cos 50 x 0.4 x 1.2 x 0.25

= 0.1566 kN

Load W4 $= \frac{1}{2}\frac{a}{\cos a} \times Cis \times L \times Qw$

= = ½ x 2/cos 40 x (-0.6) x 1.2 x 0.25

= -0.2350 kN

Load W5 $= \frac{a}{\cos a} \times Cis \times L \times Qw$

= 2/cos 40 x (-0.6) x 1.2 x 0.25

= -0.4699 kN

Load W6 $= \frac{\left(\frac{a}{2}+b\right)}{\cos a} \times Cis \times L \times Qw$

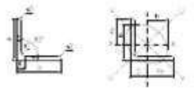
= (2/2+1)/cos 40 x (-0.6) x 1.2 x 0.25

= -0.3831 kN

2.7.4 Truss Element Design Planning

In planning the truss design element, it needs to do modelling work on SAP2000 Software to obtain some data that needed in calculations. Designing the truss elements for dormitory building and educational building in SAP2000 Software, is using 2L profiles with dimensions 60 x 60 x 6 (See figure 2.40). Another needed data in the calculation is obtained from the profile table, where it can be seen in figure 2.40. Calculation of truss element design planning includes compression bar and tension bar.

Standard Sectional Dimension of **Equal Angle** Steel and Its
Sectional Area, Unit Weight and Sectional Characteristic



Note :

Sectional Dimension					Sectional Properties												Note
A x B	t	K	r1	r2	Center of grav. (c)	Sec.of Area	Unit Weight	Geometrical Moment of Inertia (cm^4)			Radius of Gyration of Area (cm)			Modulus of Section (cm^3)			
mm mm	mm	mm	mm	mm	cm	cm²	kg/m	Ix=Iy	Iv	Iu	ix=iy	iv	iu	Sx=Sy	Sv	Su	
L 25 x 25	3.0	7.0	4.0	2.0	0.72	1.43	1.12	0.80	0.33	1.26	0.75	0.48	0.94	0.45	0.33	0.71	
L 30 x 30	3.0	7.0	4.0	2.0	0.84	1.73	1.36	1.42	0.59	2.26	0.91	0.58	1.14	0.66	0.50	1.07	
L 40 x 40	3.0	7.5	4.5	2.0	1.09	2.34	1.84	3.53	1.46	5.60	1.23	0.79	1.55	1.21	0.95	1.98	
	4.0	10.0	6.0	3.0	1.12	3.08	2.42	4.48	1.87	7.12	1.21	0.78	1.52	1.55	1.18	2.52	
	5.0	9.5	4.5	3.0	1.17	3.76	2.95	5.42	2.25	8.59	1.20	0.77	1.51	1.92	1.36	3.04	
L 45 x 45	4.0	10.5	6.5	3.0	1.24	3.49	2.74	6.50	2.70	10.30	1.36	0.88	1.72	1.99	1.54	3.24	
	5.0	11.5	6.5	3.0	1.28	4.30	3.38	7.91	3.29	12.50	1.36	0.87	1.70	2.46	1.82	3.93	
L 50 x 50	4.0	10.5	6.5	3.0	1.37	3.89	3.05	9.06	3.76	14.40	1.53	0.98	1.92	2.50	1.94	4.07	
	5.0	11.5	6.5	3.0	1.41	4.80	3.77	11.10	4.58	17.50	1.52	0.98	1.91	3.09	2.30	4.95	
	6.0	12.5	6.5	4.5	1.44	5.64	4.43	12.60	5.23	20.00	1.49	0.96	1.88	3.54	2.57	5.66	
L 60 x 60	4.0	10.5	6.5	3.0	1.61	4.69	3.68	16.00	6.62	25.40	1.85	1.19	2.33	3.64	2.91	5.99	
	5.0	11.5	6.5	3.0	1.66	5.80	4.55	19.60	8.09	31.20	1.84	1.18	2.32	4.52	3.45	7.35	
	6.0	14.0	8.0	4.0	1.69	6.91	5.42	22.80	8.28	36.24	1.82	1.09	2.29	5.29	3.46	8.54	
L 65 x 65	5.0	13.5	8.5	3.0	1.77	6.37	5.00	25.30	10.50	40.10	1.99	1.28	2.51	5.35	4.19	8.72	
	6.0	14.5	8.5	4.0	1.81	7.53	5.91	29.40	12.20	46.60	1.98	1.27	2.49	6.27	4.77	10.14	
	8.0	16.5	8.5	6.0	1.88	9.76	7.66	36.80	15.30	58.30	1.94	1.25	2.44	7.97	5.75	12.68	
L 70 x 70	6.0	14.5	8.5	4.0	1.93	8.13	6.38	37.10	15.30	58.90	2.14	1.37	2.69	7.32	5.61	11.90	
	7.0	16.0	9.0	4.5	1.97	9.40	7.38	42.40	17.64	67.01	2.12	1.37	2.67	8.43	6.33	13.54	
L 75 x 75	6.0	14.5	8.5	4.0	2.06	8.73	6.85	46.10	19.00	73.20	2.30	1.48	2.90	8.47	6.52	13.80	
	8.0	18.0	10.0	5.0	2.13	11.50	9.03	58.90	24.51	93.41	2.26	1.46	2.85	10.97	8.14	17.61	
	8.0	17.5	8.5	6.0	2.17	12.69	9.96	64.40	26.70	102.00	2.25	1.45	2.84	12.08	8.70	19.23	
	12.0	20.5	8.5	6.0	2.29	15.56	13.00	81.90	34.50	129.00	2.22	1.44	2.79	15.72	10.65	24.32	
L 80 x 80	6.0	14.5	8.5	4.0	2.18	9.33	7.32	56.40	23.20	89.60	2.46	1.58	3.10	9.59	7.53	15.84	
	8.0	18.0	10.0	5.0	2.26	12.30	9.66	72.30	29.55	115.17	2.42	1.55	3.06	12.60	9.25	20.36	
L 90 x 90	6.0	16.0	10.0	5.0	2.42	10.55	8.28	80.70	33.40	128.00	2.77	1.78	3.48	12.26	9.76	20.11	
	7.0	17.0	10.0	5.0	2.46	12.22	9.59	93.00	38.30	148.00	2.76	1.77	3.48	14.22	11.01	23.26	
	9.0	20.0	10.0	5.5	2.54	15.50	12.17	116.00	48.01	184.49	2.74	1.76	3.45	17.96	13.37	28.99	
	10.0	20.0	10.0	7.0	2.57	17.00	13.35	125.00	51.70	199.00	2.71	1.74	3.42	19.44	14.22	31.27	
	13.0	23.0	10.0	7.0	2.69	21.71	17.04	156.00	65.30	248.00	2.68	1.73	3.38	24.72	17.17	38.97	
L 100 x 100	7.0	17.0	10.0	5.0	2.71	13.62	10.69	129.00	53.20	205.00	3.08	1.98	3.88	17.70	13.88	28.99	
	8.0	18.0	10.0	7.0	2.75	15.47	12.14	146.00	58.82	234.09	3.07	1.95	3.89	20.14	15.13	33.11	
	10.0	20.0	10.0	7.0	2.82	19.00	14.92	175.00	72.00	278.00	3.03	1.95	3.83	24.37	18.05	39.32	
	13.0	23.0	10.0	7.0	2.94	24.31	19.08	220.00	91.10	348.00	3.01	1.94	3.78	31.16	21.91	49.21	
L 120 x 120	8.0	20.0	12.0	5.0	3.24	18.76	14.73	258.00	106.00	410.00	3.71	2.38	4.67	29.45	23.13	48.32	
	11.0	24.0	13.0	6.5	3.36	25.40	19.94	341.00	140.27	542.15	3.66	2.35	4.62	39.47	29.52	63.89	
	12.0	25.0	13.0	6.5	3.40	27.50	21.59	388.00	151.87	581.90	3.66	2.35	4.60	42.79	31.58	68.58	
L 130 x 130	90.0	21.0	12.0	6.0	3.53	22.74	17.85	366.00	150.00	583.00	4.01	2.57	5.06	38.65	30.05	63.42	
	12.0	24.0	12.0	8.5	3.64	29.76	23.36	467.00	192.00	743.00	3.96	2.54	5.00	49.89	37.30	80.83	
	15.0	27.0	12.0	8.5	3.76	36.75	28.85	568.00	234.00	902.00	3.93	2.52	4.95	61.47	44.01	98.12	
L 150 x 150	12.0	26.0	14.0	7.0	4.14	34.77	27.29	740.00	304.00	1180.00	4.61	2.96	5.83	68.14	51.92	111.25	
	15.0	29.0	14.0	10.0	4.24	42.74	33.55	888.00	365.00	1410.00	4.56	2.92	5.74	82.53	60.87	132.94	
	19.0	33.0	14.0	10.0	4.40	53.38	41.90	1090.00	451.00	1730.00	4.52	2.91	5.69	102.83	72.48	165.11	
L 175 x 175	12.0	27.0	15.0	11.0	4.73	40.52	31.81	1170.00	480.00	1860.00	5.37	3.44	6.78	91.62	71.76	150.31	
	15.0	30.0	15.0	11.0	4.85	50.21	39.41	1440.00	589.00	2290.00	5.36	3.43	6.75	113.83	85.87	185.06	
L 200 x 200	15.0	32.0	17.0	12.0	5.46	57.75	45.33	2180.00	891.00	3470.00	6.14	3.93	7.75	149.93	115.39	245.37	
	20.0	37.0	17.0	12.0	5.67	76.00	59.66	2820.00	1160.00	4490.00	6.09	3.91	7.69	196.79	144.66	317.49	
	25.0	42.0	17.0	12.0	5.86	93.75	73.59	3420.00	1410.00	5420	6.04	3.88	7.60	241.87	170.14	383.25	
L 250 250	25.0	49.0	24.0	12.0	7.10	119.40	93.73	695.000	2860.00	1100	7.63	4.89	9.60	388.27	284.83	622.25	
	35.0	59.0	24.0	18.0	7.45	162.60	127.64	9110.00	3790.00	1440	7.49	4.83	9.41	519.09	359.72	814.59	

Table 2.27 Profile L

2.7.4.1. Educational Roof

1. Exterior Compression Bar

a. Bending Check

Calculation of bending checks can use the formula below:

$$\lambda = \frac{b}{t} \quad \text{………………………………………………..……………………}(2.3)$$

$$\lambda r = 0.45\sqrt{\frac{E}{Fy}} \quad \text{…………………………………………………………..…...}(2.4)$$

If λ < λr so the cross section is non slender. But if on the contrary, it is categorized as a slim cross section.

The result of the calculation formulas (2.3) and (2.4) are as follows:

Known:

Compression bar = 2 kN

Tension bar = 46.032 kN

$\lambda = \frac{60}{6}$

$= 10$

$\lambda r = 0.45\sqrt{\frac{200.000}{240}}$

$= 12.9904$

So, it can be concluded that it is included in the non-slim cross-section.

b. Bending check (X-X axis)

Calculation of bending checks about the x-x axis using the formula :

$$\frac{KL}{rx} \qquad (2.5)$$

$$Fe = \frac{\pi^2 E}{(\frac{KL}{r})^2} \qquad (2.6)$$

$$4{,}71\sqrt{\frac{E}{Fy}} \qquad (2.7)$$

Calculation result from formulas 2.51, 2.52, 2.53 that are :

$\frac{KL}{rx} = \frac{2 \times 1031.7}{18.2} = 113.374$

$Fe = \frac{\pi^2 \times 200.000}{134{,}1428^2} = 153.57$ MPa

$4{,}71\sqrt{\frac{200.000}{240}} = 135.97$

It can be seen that $\frac{KL}{rx} > 4{,}71\sqrt{\frac{E}{Fy}}$ so, F_{cr} is taken from the equation:

$$F_{cr} = 0{,}877Fe \qquad (2.8)$$

$= 0{,}877 \times 153.57$

$= 134.68$ Mpa

c. Torsion bending check

Compressed structural components are connected using bolts so it is necessary to know the a/r value using the formula:

$$\frac{a}{r} \qquad (2.9)$$

$\frac{a}{r} = \frac{1031.7}{18.2} = 56.6868$ because of a/r < 40 then use the equation:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)^2 + \left(\frac{K_i a}{r_i}\right)^2} \qquad (2.10)$$

$$= \sqrt{(113.374)^2 + (0,5 \times 56.6868)^2}$$

$= 116.863$

The result of $(\frac{KL}{r})_m > 4.71\sqrt{\frac{E}{Fy}}$ so, F_{cry} can use equation (2.8) while to find out the value of Fe then using formula (2.9) with result 144.536 Mpa. Thus Fcry obatained result is 126.758 Mpa. Fcrz value can use the formula :

$$F_{crz} = (\frac{GJ}{A \times r0}) \text{ ....} (2.11)$$

$$F_{crz} = (\frac{77.200 \times 2673}{1382 \times 1085.02})$$

$= 137.617$ MPa

F_{cr} for doubled angled structural components using formula below :

$$F_{cr} = \left(\frac{Fcry + Fcrz}{2H}\right)\left[1 - \sqrt{1 - \frac{4.F_{cry}.F_{rcz}.H}{(F_{cry} + F_{crz})^2}}\right] \text{ ....} (2.12)$$

$$= \left(\frac{126.758+136.617}{2 \times 0,8102}\right)\left[1 - \sqrt{1 - \frac{4 \times 126.758 \times 136.617 \times 0.8102}{(126.758 + 136.617)^2}}\right]$$

$= 91.8198$ MPa

d. Compressive Strength Design

It is known that Fcr on bending check is 134.68 Mpa and Fcr on torsion bending check is 91.8198 Mpa, so Fcr is chosen which has a smaller value of 91.8198 MPa. Thus the design compressive strength value can be calculated using the formula:

$$\emptyset_c P_n = 0,9 \times F_{cr} \times A_g \text{ ....} (2.13)$$

$= 0,9 \times 91.8198 \times 1382$

$= 114.206$ kN

The result show that $\emptyset_c P_n >$ Maximum compressive force (2 kN) so the design compressive strength is safe.

2. Interior Compression Bar

The interior compression rod calculations are generally the same as the exterior compression bar calculations. However, because this calculation is carried out on the interior, there will be some differences in the data which will affect the results.

a. Bending check

It is known that the value of the compression bar is 16.8 kN and the tension bar is 17.092 kN. Therefore, the calculation of bending checks can use formula (2.3) with a result of 10 and formula (2.4) with 12.99. It can be seen that $\lambda < \lambda r$, it can be concluded that the section is included in the category of non-slim section.

b. Bending check (X-X axis)

This calculation follows formula (2.5) with result 279.692, formula (2.6) with result 25.233 Mpa, and formula (2.7) with result 135.966 . It can be seen that $\frac{KL}{rx} > 4,71\sqrt{\frac{E}{Fy}}$ so, F_{cr} is taken from equation (2.8) With result 22.1293 MPa.

c. Torsion bending check

Compressed structural components are connected using bolts so it is necessary to know the a/r value using the formula (2.9) with result 139.846 where a/r > 40 so using equation (2.10) with result 288.3. Result known $(\frac{KL}{r})_m > 4,71\sqrt{\frac{E}{Fy}}$ so, F_{cry} using equation (2.8) while to find out the value of Fe then using formula with result 23.7487 Mpa. Thus, Fcry obtained result is 20.8276 Mpa. Fcrz value can use the formula (2.11) with result 137.617 MPa.

F_{cr} for doubled angled structural components using formula (2.12) with result 20.1702 MPa.

d. Design Compressive Strength

It is known that Fcr on bending check is 22.1293 Mpa and Fcr on torsion bending check is 20.1702 Mpa, so Fcr is chosen which has a smaller value of 20.1702 MPa. Thus, the design compressive strength value can be calculated using the formula (2.13) with result 25087.7 kN. From the result, it can be seen that $\emptyset_c P_n$ > Maximum compressive force (16.8 kN) so the design compressive strength is safe.

3. Exterior Tension Bar

a. Tension Bar Slenderness

$\lambda = \frac{L}{r}$...(2.14)

$= \frac{1679.9}{18.2}$

= 92.3022 < 300 so it's safe.

b. Tensile Melting Conditions

The nominal tensile strength due to tensile yielding is obtained through the equation:

$P_n = F_y A_g$...(2.15)

$= 240 \times 1382$

= 331680 N

Tensile yield check using the formula:

$\emptyset P_n = \emptyset F_y A_g$...(2.16)

$= 0{,}9 \times 240 \times 1382$

= 298512 N = 298,512 kN

It is known that ØPn > Pu (46.032 kN) then it is safe.

4. Interior Tension Bar

In general, the calculation of the interior tensile rod is the same as the exterior tensile rod, only the data differs, which affects the final result.

a. Tension Bar Slenderness

This calculation uses the formula (2.14) with a result of 133.418 which is smaller than 300, so it is safe.

b. Tensile Melting Conditions

The nominal tensile strength due to yielding in tension is obtained through formula (2.15) with result 331680 kN. Yield in tension check (ØPn) uses formula (2.16) with result 298.512 kN. It is known that ØPn > Pu (17.092 kN) then it is safe.

2.7.4.2. Dormitory Building Roof

The calculation of the truss element design plan uses the same formula as on the roof of a educational building, but some data will be different so that the final result between the two roofs is also different.

1. Exterior Compression Bar

a. Bending Check

Calculation of bending check can use formulas (2.3) and (2.4)

If $\lambda < \lambda r$ then the cross section is non slender. But if on the contrary, it is categorized as a slim cross section.

The results of the calculation formulas (2.3) and (2.4) are as follows:

Compression bar = 67.829 kN

Tension bar = 33.975 kN

$\lambda = \frac{60}{6}$

$= 10$

$\lambda r = 0{,}45\sqrt{\frac{200.000}{240}}$

$= 12{,}99$

So it can be concluded that it is included in the non-slim cross-section.

b. Bending check (X-X axis)

The calculation of bending checks about the x-x axis uses formula (2.4) with a result of 431.319 MPa, formula (2.5) with result 10.6104 MPa, and formula (2.6) with result 135.966 MPa. It can be seen that $\frac{KL}{rx} > 4{,}71\sqrt{\frac{E}{Fy}}$ then Fcr is taken from (2.7) with result 9.3053 MPa.

c. Torsion bending check

Compressed structural components are connected using bolts so it’s necessary to know the a/r value using formula (2.10)

$\frac{a}{r} = \frac{3925}{18,2}$ = 215.659, because a/r > 40 then use equation (2.10) with result 444.593 it can seen the result $(\frac{KL}{r})_m > 4,71\sqrt{\frac{E}{Fy}}$ then, Fcry can use equation (2.10) while to find out the value of Fe then using formula (2.11) with result 9.9863 Mpa. Thus Fcry obtained results of 8.7579 Mpa. The Fcrz value can use the formula (2.11) with a result of 137.617 MPa. Fcr for double angled structural members uses the formula (2.12) with a yield of 8.6479 MPa.

d. Design Compressive Strength

It is known that Fcr on bending check is 9.3053 Mpa and Fcr on torsion bending check is 8.6479 Mpa, so Fcr is chosen which has a smaller value of 8.6479 MPa. Thus the value of the design compressive strength can be calculated using the formula (2.13) with a result of 10756.3 kN. The result show that $\emptyset_c P_n$ > Maximum compressive force (67.829 kN) so the design compressive strength is safe.

2. Interior Compression bar

The interior compression rod calculations are generally the same as the exterior compression bar calculations. However, because this calculation is carried out on the interior, there will be some differences in the data which will affect the results.

a. Bending check

It is known that the value of the compression bar is 24.413 kN and the tension bar is 16.938 kN. Therefore, the calculation of bending checks can use formula (2.3) with a result of 10 and formula (2.4) with 12.99. It can be seen that $\lambda < \lambda r$, it can be concluded that the section is included in the category of non-slim section.

b. Bending check (X-X axis)

This calculation follows formula (2.5) with result 299.066 Mpa, formula (2.6) with result 22.0697 Mpa, and formula (2.7)with result 135.966 Mpa. It can be seen that $\frac{KL}{rx} > 4,71\sqrt{\frac{E}{Fy}}$ so, F_{cr} is taken from equation (2.8) With result 19.3551 Mpa.

c. Torsion bending check

Compressed structural components are connected using bolts so it is necessary to know the a/r value using the formula (2.9) with result 149.533 where a/r > 40 so using equation (2.10) with result 308.27 Mpa. Result known $(\frac{KL}{r})_m > 4,71\sqrt{\frac{E}{Fy}}$ so, F_{cry} using equation (2.11) while to find out the value of Fe then using formula (2.12) with result

20.7715 Mpa. Thus Fcry obtained result is 18.2166 Mpa. Fcrz value can use the formula (2.11) with result 137.617 MPa.

F_{cr} for doubled angled structural components using formula (2.12) with result 17.7196 MPa.

d. Design Compressive Strength

It is known that Fcr on bending check is 19.3551 Mpa and Fcr on torsion bending check is 17.7196 Mpa, so Fcr is chosen which has a smaller value of 17.7196 Mpa. Thus the design compressive strength value can be calculated using the formula (2.13) with result 22039.6 kN. From the result, it can be seen that $\emptyset_c P_n$ > Maximum compressive force (24.413 kN) so the design compressive strength is safe.

3. Exterior Tension Bar

a. Tension Bar Slenderness

The continuity of the tension bar uses the (2.14) formula with a result of 83.7363 which is smaller than 300, so it is safe.

b. Tensile Melting Conditions

The nominal tensile strength due to yielding in tension is obtained through equation (2.15) with a result of 331680 kN. Checking the tensile yield (ØPn) using formula (2.16) with a result of 298512 kN. It is known that ØPn > Pu (33.975 kN) then it is safe.

4. Interior Tension Bar

In general, the calculation of the interior tensile rod is the same as the exterior tensile rod, only the data differs, which affects the final result.

a. Tension Bar Slenderness

This calculation uses the formula (2.17) with a result of 143.412 which is smaller than 300, so it is safe.

b. Tensile Melting Conditions

The nominal tensile strength due to yielding in tension is obtained through formula (2.15) with result 331680 kN. Yield in tension check (ØPn) uses formula (2.16) with result 298512 kN. It is known that ØPn > Pu (16.938 kN) then it is safe.

2.7.5 Truss Connection Design

In Steel construction, each part of the elements of the structure is connected to each other by *fastener* or connectors. In frame structures, both roofs and steel bridges, portal structures where the rods gather are called gusset points. This connecting plate is called a gusset plate, where the rods were fastened using a fastener on the gusset plate. There are several types of fasteners that are often used, namely *rivets*, *bolts*, and *welded*. In planning the truss connection on the two joglo roofs of the public library using bolt connections.

2.7.4.1 Dormitory

It is known that bolt A325-X with M-20 diameter bolt is used, the gusset plates are connected from ASTM A36 steel (Fy 240 Mpa; fu 370 MPa). The tensile strength of the dormitory building roof on the exterior profile is 46.032 kN and on the interior profile is 17.092 kN. Planning bolt connections for the truss as follows:

1. **Melt Tensile Check on Gross Section**

The plate size used is 6×250 mm, so the gross cross-section is 1500 mm2.
The tensile yield is calculated using the formula:

$ØPn = 0.9 \times Fy \times Ag$ ……………………………………………………….…………(2.17)

$= 0.9 \times 240 \times 1500$

$= 324000$ N

$= 324$ kN > 46.032 kN (exterior profile)

$= 324$ kN > 17.092 kN (interior profile)

It is known that the tensile yield that occurs at the gross cross-section is greater than the strength of the roof tensile rod, so it can be concluded that it is safe.

2. **Tensile Collapse Check on Net Section**

Tensile collapse check can be calculated using the formula:

An = $(250-2 \times (22+2)) \times 6 = 1212$ mm^2

Max An = $0.85Ag$……………………………………………………..(2.18)

$= 0.85 \times 1500$

$= 1275 \text{ mm}^2$

$Ae = An \quad = 1212 \text{ mm}^2$

$ØPn \quad = 0.75 \times Fu \times Ae$ ……………………………………………..(2.19)

$= 0.75 \times 370 \times 1212$

$= 336330 \text{ N}$

$= 336.33 \text{ kN} > 46.032 \text{ kN}$ (eksterior profile)

$= 336.33 \text{ kN} > 17.092 \text{ kN}$ (interior profile)

It is known that the net cross-sectional tensile failure is greater than that of the roof tension rod, so it can be concluded that it is safe.

3. Bolt Support Strength

Calculation of bolt bearing strength can use the formula:

$Rn \quad = 2.4dtFu$ ……………………………………………………………..………(2.20)

$= 2.4 \times 20 \times 6 \times 370$

$= 106560 \text{ N}$

$= 106.56 \text{ kN}$

$ØRn \quad = Ø \times Rn$ ……………………………………………………………..……….(2.21)

With :

$Ø \quad = 0.75$

$= 0.75 \times 106{,}56$

$= 79.92 \text{ kN}$

4. Bolt Shear Strength

Calculation of bolt shear strength using the formula:

$Rn \quad = FnvAb$ …………………………………...………………………(2.22)

With :

Fn = shear stress

Ab = cross - sectional area

$Rn \quad = FnvAb$

$= 457 \times (1/4 \times \pi \times 20^2) \times 2$

$= 287141.5 \text{ N}$

Calculation of ∅Rn using the formula with a result of 215.3561 kN. Thus, the smallest value between the bearing strength of the bolt and the shear strength of the bolt is chosen, namely 79.92 kN.

5. **Number of Bolts Calculation**

$$\text{Bolt Amount} = \frac{\text{plate}}{\text{Rn}} \quad (2.23)$$

$$= \frac{250}{79.92}$$

$$= 3.1281 \text{ Pieces}$$

From the calculation, it's rounded up into 3 pieces of bolts.

2.7.4.2 Educational Building

It is known that bolt A325-X with M-20 diameter bolt is used, the gusset plates are connected from ASTM A36 steel (Fy 240 Mpa; fu 370 MPa). The tensile strength of the educational building roof on the exterior profile is 33.975 kN and on the interior profile is 16.938 kN. Planning bolt connections for the truss as follows:

1. **Melt Tensile Inspection on Gross Section**

The size of the plate used is 6×250 mm so that the gross cross section is 1500 mm2. Yield tensile is calculated using formula 2.17 with the result of 324 kN which is greater than the strength of the tensile rods on the exterior and interior of the roof so it is safe.

2. **Examination of Tensile Collapse at Net Section**

It is known that An/Ae is 1212 mm2. The tensile failure check can be calculated using the formula 2.18 with a yield of 1275 mm2 and 2.19 with a yield of 336.33 kN. It is known that the net cross-sectional tensile failure is greater than that of the exterior and interior roof tension rods, so it can be concluded that it is safe.

3. **Bolt bearing strength**

Calculation of bolt bearing strength using formula 2.20 with a result of 106.56 kN and formula 2.21 with a result of 79.92 kN

4. **Bolt Shear Strength**

Calculation of bolt shear strength using formula 2.22 with a result of 287141.5 N. Calculation of ∅Rn using the formula 2.22 with a result of 215.3561 kN. Thus, the smallest value between the bearing strength of the bolt and the shear strength of the bolt is chosen, namely 79.92 kN.

5. Calculation of the Number of Bolts

The number of bolts is calculated using the formula 2.23 with a result of 3.1281 rounded up to 3 bolts.

2.8 Beam Design

Based on Table 21.2.2 SNI 2847:2019 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 2.39.

Tabel 21.2.2 – Faktor reduksi kekuatan (ϕ) untuk momen, gaya aksial, atau kombinasi momen dan gaya aksial

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

Table 2.28 strength reduction factor for moments, axial forces, or a combination of moments and axial forces

Main beam and support beam, using the same formulas to do the designing. There's some different data that affect the results. The calculation using the formula below, as follows:

2.8.1 Beam Calculation (Ex. Main Beam 1)

- **Structure Material**
  - Concrete compressive strength (fc') = 25 MPa
  - Steel yield stress (deform) for flexural reinforcement (fy) = 400 MPa
  - The yield stress of (plain) steel for shear reinforcement (fy) = 240 MPa
- Beam Dimensions
  - Beam width (b) = 200 mm
  - Beam height (h) = 400 mm
  - The diameter of the reinforcement (deform) used (D) = 12 mm

- The diameter of the stirrups (plain) used (P) = 10 mm
- The net thickness of the concrete cover (ts) = 20 mm

- Shear Force and Moment Plan

  The data source is come from ETABS application output.

  - Positive design moment due to factored load (Mu+) = 20.240 kNm
  - Negative design moment due to factored load (Mu-) = -30.155 kNm
  - Design shear due to factored load (Vu) = 30.302 kN

2.8.1.1 Reinforcement Calculation

Concrete Stress Distribution Factor

For : fc' ≤ 30 MPa, β_1 = 0.85

For : fc' > 30 MPa, $\beta 1 = 0.85 - 0.05 \times \frac{f'c - 30}{7}$

Form factor of concrete stress distribution,

β_1 = 0.85

Reinforcement ratio in balance condition,

$$\rho_b = \beta 1 \times 0.85 \times \frac{f'c}{fy} \times \frac{600}{(600+fy)}$$

$$= \beta 1 \times 0.85 \times \frac{25}{400} \times \frac{600}{(600+400)}$$

$$= 0.0217$$

Maximum moment resistance factor,

$$Rmax = 0.75 \times \rho b \times fy \times [1 - \frac{1}{2} \times 0.75 \times \rho b \times \frac{fy}{(0.85 \times fc')}]$$

$$= 0.75 \times 0.0217 \times 400 \times [1 - \frac{1}{2} \times 0.75 \times 0.0217 \times \frac{400}{(0.85 \times 25)}]$$

$$= 6.5736$$

Flexural strength reduction factor,

ϕ =0.80

Distance of reinforcement to the outside of the concrete,

$d_s \quad = ts + \emptyset + \frac{D}{2}$

$= 20 + 10 + \frac{12}{2}$

$= 36.00$ mm

Amount of reinforcement in one row,

$n_s \quad = \frac{(b - 2 \times ds)}{(25 + D)}$

$= \frac{(200 - 2 \times 36.00)}{(25 + 12)}$

$= 3.46 \approx 3$

So, the amount of reinforcement in one row is 3 pcs.

Center to center horizontal distance between bars,

$X \quad = \frac{b - (ns \times D) - (2 \times ds)}{(ns - 1)}$

$= \frac{200 - (3 \times 12) - (2 \times 36.00)}{(3 - 1)}$

$= 46.00$ mm

Center to center vertical distance between bars,

$Y \quad = D + 25 = 12 + 25 = 37$ mm

1. Positive Moment Reinforcement

Design nominal positive moment,

$M_n \quad = Mu \times \phi = 20.240 \text{ x } 0.80 = 25.300$ kNm

Estimated distance of the center of the flexural reinforcement to the concrete side,

$d' \quad = 40$ mm

Effective beam height,

$d \quad = h - d'$

$\quad = 400 \text{ mm} - 40 \text{ mm} = 360.00 \text{ mm}$

Moment resistance factor,

$$R_n = \frac{Mn \times 10^6}{(b \times d^2)}$$

$$= \frac{25.3 \times 10^6}{(200 \times 360^2)}$$

$$= 0.9761$$

$R_n < R_{max}$ (OK)

Required reinforcement ratio:

$$\rho = 0.85 \times \frac{f'c}{fy} \times [1 - \sqrt{[1 - \frac{2 \times Rn}{(0.85 \times f'c)}]}$$

$$= 0.85 \times \frac{25}{400} \times [1 - \sqrt{[1 - \frac{2 \times 0.9761}{(0.85 \times 25)}]}$$

$$= 0.00250$$

Minimum reinforcement ratio,

$$\rho \min = \frac{\sqrt{f'c}}{(4 \times fy)}$$

$$= \frac{\sqrt{25}}{(4 \times 400)}$$

$$= 0.00313$$

$$\rho \min = \frac{1.4}{fy}$$

$$= \frac{1.4}{400}$$

$$= 0.00350$$

The ratio of reinforcement used,

$\rho \quad = 0.00350$

Required reinforcement area,

As $= \rho \times b \times d$

$= 0.00350 \times 200 \times 560$

= 252 mm2

The amount of reinforcement required,

n $= \frac{As}{(\frac{\pi}{4} \times D^2)}$

$= \frac{252}{(\frac{\pi}{4} \times 12^2)}$

= 2.228

Used reinforcement, 3 D 12

Area of used reinforcement,

As $= n \times \frac{\pi}{4} \times D^2$

$= 2.228 \times \frac{\pi}{4} \times 12^2$

= 339 mm^2

Number of rows of reinforcement,

nb $= \frac{n}{ns} = \frac{2.228}{3} = 1.00$

nb < 3 (OK)

Table 2. 29 n distance

Line To	Amount n_i	Distance y_i	Amount x Distance $n_i \times y_i$
1	3	36	108
2	0	0	0
3	0	0	0
n=	3	$\Sigma [n_i * y_i] =$	108

Location of center of gravity of reinforcement,

$$d' = \frac{\Sigma [\ ni \times yi\ ]}{n}$$

$$= \frac{108}{3}$$

$$= 36.00 \text{ mm}$$

36.00 < 40 estimated d' (OK)

Effective beam height

$$d = h - d' = 400 - 36 = 364.00 \text{ mm}$$

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

$$= \frac{339 \times 400}{(\ 0.85 \times 25 \times 200\ )}$$

$$= 31.933 \text{ mm}$$

Nominal moment,

$$Mn = As \times fy \times (\ d - \frac{a}{2}) \times 10^{-6}$$

$$= 339 \times 400 \times (\ 364 - \frac{31.933}{2}) \times 10^{-6}$$

$$= 47.234 \text{ kNm}$$

The beam moment resistance,

ϕMn = 0.80 x 47.234 = 37.787 kNm

Terms:

$\phi Mn \geq Mu^{+}$

37.787 > 20.240 SAFE (OK)

Positive Moment Reinforcement Result Recap

Beam Type	ϕMn	Mu$^+$	ϕMn > Mu$^+$	
Dormitory Building				
Main Beam 1	37,787	20.240	37,787 > 20.240	**SAFE**
Main Beam 2	32,359	23.222	32,359 > 23.222	**SAFE**
Main Beam 3	18,338	15.262	18,338 > 15.262	**SAFE**
Main Beam 4	14,719	7.924	14,719 > 7.924	**SAFE**
Main Beam 5	20,660	18.285	20,660 > 18.285	**SAFE**
Support Beam 1	14,719	1.018	14,719 > 1.018	**SAFE**
Support Beam 2	21,958	17.054	21,958 > 17.054	**SAFE**
Support Beam 3	14.719	0.383	14.719 > 0.383	**SAFE**
Educational Building				
Main Beam 1	38,134	24,265	38,134 > 24,265	**SAFE**
Main Beam 2	21,958	17,554	21,958 > 17,554	**SAFE**
Main Beam 3	21,958	14,864	21,958 > 14,864	**SAFE**
Main Beam 4	14,719	9,486	14,719 > 9,486	**SAFE**
Main Beam 5	11,100	6,093	11,100 > 6,093	**SAFE**
Main Beam 6	18,338	10,195	18,338 > 10,195	**SAFE**
Main Beam 7	14,719	5,666	14,719 > 5,666	**SAFE**
Main Beam 8	14,719	4,044	14,719 > 4,044	**SAFE**
Main Beam 9	5,743	0,740	5,743 > 0,740	**SAFE**
Support Beam 1	14,719	3,952	14,719 > 3,952	**SAFE**
Support Beam 2	21,958	10,919	21,958 > 10,919	**SAFE**
Support Beam 3	21,958	13,319	21,958 > 13,319	**SAFE**
Support Beam 4	38,134	18,553	38,134 > 18,553	**SAFE**

Table 2. 30 Positive Moment Reinforcement Result Recap

2. Negative Moment Reinforcement

Design nominal negative moment,

$$Mn = \frac{Mu-}{\phi}$$

$= \frac{30.155}{0.80}$

= - 37.694 kNm

Estimated distance of the center of the flexural reinforcement to the concrete side, d' = 40 mm

Effective beam height,

d $= h - d'$

= 400 - 40

= 360.00 mm

Moment resistance factor,

Rn $= Mn \times \frac{106}{(b \times d^2)}$

$= -37.694 \times \frac{106}{200 \times 360^2}$

= -1.4542

Rn < Rmax (OK)

Required reinforcement ratio:

ρ $= 0.85 \times \frac{f'c}{fy \times [1 - \sqrt{[1 - 2 \times Rn / (0.85 \times f'c)]}}$

$= 0.85 \times \frac{25}{400 \times [1 - \sqrt{[1 - 2 \times (-1.4542) / (0.85 \times 25)}}$

= 0.00352

Minimum reinforcement ratio,

ρ min $= \frac{\sqrt{f'c}}{(4 \times fy)}$

$= \frac{25}{(4 \times 400)}$

= 0.00280

Minimum reinforcement ratio,

ρ min $= \frac{1.4}{fy}$

$= \frac{1.4}{400}$

= 0.00313

The ratio of reinforcement used,

ρ = 0.00350

Required reinforcement area,

As $= \rho \times b \times d$

$= 0.00350 \times 200 \times 360$

= 252 mm^2

The amount of reinforcement required,

n $= \frac{As}{\left(\frac{P}{4} \times D^2\right)}$

$= \frac{252}{\left(\frac{0.00350}{4} \times 12^2\right)}$

= 2.228

Used reinforcement, 3 D 16

Area of used reinforcement,

As $= \frac{n \times \pi}{4 \times D^2}$

$= \frac{2.228 \times 3.14}{4 \times 12^2}$

339 mm^2

Number of rows of reinforcement,

nb $= \frac{n}{ns} = \frac{3}{3} = 1$

nb < 3 (OK)

Table 2. 31 n distance

Line To	Amount n_i	Distance y_i	Amount x Distance $n_i \times y_i$
1	3	36	108
2	0	0	0
3	0	0	0
n=	3	$\Sigma [n_i * y_i] =$	108

Location of center of gravity of reinforcement,

d' $= \frac{[ni \times yi]}{n}$

$= \frac{108}{3}$

= 36.00 mm

36.00 < 40 estimate d' (OK)

Effective beam height,

d = h - d'

$= 400 - 36.00$

= 364.0 mm

a $= \text{As} \times \frac{\text{fy}}{0.85 \times f'c \times b}$

$= 339 \times \frac{400}{0.85 \times 200}$

= 31.933 mm

Nominal moment,

Mn $= \text{As} \times \text{fy} \times (\text{d} - \frac{a}{2}) \times 10 - 6$

$$= 339 \times 400 \times \left(364 - \frac{31.933}{2}\right) \times 10 - 6$$

$$= 47.234 \text{ kNm}$$

The beam moment resistance,

$\phi \times Mn$

$= 0.80 \times 47.234$

= 37.787 kNm

Terms : $\phi \times Mn \geq Mu-$

37.787> - 30.155 SAFE (OK)

Negative Moment Reinforcement Recap

Beam Type	ϕMn	Mu^-	ϕMn > Mu^+	
Dormitory Building				
Main Beam 1	37,787	-30,155	37,787 > -30,155	**SAFE**
Main Beam 2	21.958	-35,591	32,359 > -35,591	**SAFE**
Main Beam 3	18,338	-7,822	18,338 > -7,822	**SAFE**
Main Beam 4	14,719	-16,230	14,719 > -16,230	**SAFE**
Main Beam 5	20,660	-96,015	20,660 > -96,015	**SAFE**
Support Beam 1	14,719	-0,692	14,719 > -0,692	**SAFE**
Support Beam 2	21,958	-26,049	21,958 > -26,049	**SAFE**
Support Beam 3	14.719	-1.183	14.719 > -1.183	**SAFE**
Educational Building				
Main Beam 1	38,134	-34,422	38,134 > -34,422	**SAFE**
Main Beam 2	21,958	-25,856	21,958 > -25,856	**SAFE**
Main Beam 3	21,958	-21,095	21,958 > -21,095	**SAFE**
Main Beam 4	14,719	-15,814	14,719 > -15,814	**SAFE**
Main Beam 5	11,100	-9,099	11,100 > -9,099	**SAFE**
Main Beam 6	18,338	-14,354	18,338 > -14,354	**SAFE**
Main Beam 7	14,719	-2,183	14,719 > -2,183	**SAFE**

Main Beam 8	14,719	-5,025	14,719 > -5,025	**SAFE**
Main Beam 9	5,743	0,166	5,743 > 0,166	**SAFE**
Support Beam 1	14,719	-0,231	14,719 > -0,231	**SAFE**
Support Beam 2	21,958	-17,946	21,958 > -17,946	**SAFE**
Support Beam 3	21,958	-21,920	21,958 > -21,920	**SAFE**
Support Beam 4	38,134	-25,712	38,134 > -25,712	**SAFE**

Table 2. 32 Negative Moment Reinforcement Result Recap

3. Shear Reinforcement

The design ultimate shear force,

Vu = 30.302 kN

Shear strength reduction factor,

ϕ = 0.60

Yield stress of shear reinforcement,

fy = 240 MPa

Concrete shear strength,

$$Vc = \frac{(\sqrt{f'c})}{fc \times b \times d \times 10^{-3}}$$

$$= \frac{(\sqrt{1})}{25 \times 200 \times 360 \times 0.001}$$

= 60.00 kN

Shear resistance of concrete,

ϕ × Vc

= 0.60 × 60

= 36.000 Kn

Requires shear reinforcement

Stirrup shear resistance,

ϕ × Vs = Vu - ϕ × Vc

$= -$ kN

The shear strength of stirrups,

Vs $= 30.302$ kN

Stirrups with cross-section are used: 2 P 10

Area of stirrup shear reinforcement,

$$Av = ns \times \frac{\pi}{4 \times P^2}$$

$$= 2 \times \frac{\pi}{4 \times 10^2}$$

$$= 157.08 \text{ mm}^2$$

Required stirrup distance:

$$s = \frac{Av \times fy \times d}{(Vs \times 10^3)}$$

$$= \frac{157.08 \times 240 \times 360}{30.302 \times 1000}$$

$$= 447.88 \text{ mm}$$

Maximum stirrup spacing,

$$smax = \frac{d}{2}$$

$$= \frac{364}{2}$$

$$= 182.00 \text{ mm}$$

Maximum stirrup distance,

Smax $= 250.00$ mm

The spacing of stirrups that must be used,

s $= 182.00$ mm

Take the stirrup distance:

s $= 180$ mm

Used stirrups, 2 P 10 180

2.9 Column Design

The design of a column is carried out by taking into account the applicable conditions. The cross-sectional dimensions are used by taking into account the smallest cross-sectional dimensions measured in a straight line through the geometric center and not less than 300 mm. The ratio of the smallest cross-sectional dimension to the perpendicular dimension is not less than 0.4.

In the design of the column must pay attention to the bending strength of the column which must meet $\sum M_{nc} \geq 1{,}2 \sum M_{nb}$ where $\sum M_{nc}$ is the sum of the strength nominal bending of columns framing into the joint, which is evaluated at the faces of the joint. $\sum M_{nb}$ which is the sum of the nominal flexural strength of the beam framing into the joint, which is evaluated at the faces of the joint. This calculation must get a result where the strong *column* – *weak* beam calculation is done using the equation

$(Mnc\ a + Mnc\ b) \geq 1{,}2(Mnb\ ki + Mnb\ ka)$ ....

To calculate the reinforcement must pay attention to several conditions. The area of the longitudinal reinforcement Ast shall not be less than 0.01Ag and not exceed 0.006Ag. Where lap splices are permitted only within the center of the existing column depth, these must also be designed as tension lap splices and must be enclosed by transverse reinforcement.

In terms of *transverse reinforcement*, the plastic hinge area of the column (the area 10 from the face of the beam-column connection, at both ends) shall provide an enclosing transverse reinforcement.

a) The *transverse reinforcement* shall consist of a single spiral or overlapping spirals or what is commonly referred to as *overlapping*, where the stirrups are round or square ties with or without cross ties.
b) Each end bend of square restraints and crossties shall engage the outermost longitudinal bar.
c) 25.7.2.2 is the allowable limit for reinforcing stirrups where cross ties are of the same bar size or larger than the diameter of the stirrup. Successive crossties shall be alternated ends along the longitudinal reinforcement and around the perimeter of the section.

d) The use of square stirrups or cross tie *transverse reinforcement* must function as a lateral support for longitudinal reinforcement must be appropriate.

e) Reinforcement shall be arranged so that the spacing hx between longitudinal reinforcement along the perimeter of a column section supported laterally by the cross-tie angles or the legs of the stirrup shall not exceed 350 mm.

f) The value of hx should not exceed 200 mm, this is because when Pu > 0.3Agf'c or f'c > 70 MPa in a column with a stirrup around the core of the column must have lateral support provided by the angle of the stirrup or seismic hook.

The requirement for transverse reinforcement must be spaced in all directions along which lo does not exceed the smallest value of

1. ¼ smallest dimension of structural member
2. 6 times the diameter of the longitudinal reinforcement
3. $100 \text{ mm} \leq S_0 = 100 + \left(\frac{350 - h_x}{3}\right) \leq 150 \text{ mm}$

The spacing of *transverse reinforcement* in the area outside l0 is given stirrups with spacing s not exceeding 6db and 150 mm.

The requirements for the shear strength of the SRPMK column must have a design shear force (Ve) which is determined by taking into account the maximum forces that can occur at the face of the beam-column connection in each structural member. The force on the beam-column connection shall be determined by using the maximum *probable moment* strength at each end of the member corresponding to the range of factored axial load Pu acting on the member. The design shear force (Ve) shall not be less than the factored shear obtained from the structural analysis results. Then the *transverse reinforcement* along lo must be designed to withstand shear assuming Vc = 0 this can occur if the earthquake shear force is at least 50% of the necessary shear strength is maximum along l0 and the factored axial compressive force Pu including the earthquake effect is less than Agf'c /20. The shear strength of the SRPMK column design can be calculated using the equation below.

$$V_e = \frac{M_{prc\ a} + M_{prc\ b}}{l_c} \text{ ....}$$

2.9.1 1st Floor Main Beam 1

It is known that the column is 500×500 with a height of 4 m, the results obtained from the etabs are:

Pu max = -102.8959 kN

Mx = 99.9246 kNm

My = 100.1611 kNm

Pu min = -537.0741 kN

Mx = -100.6356 kNm

My = -90.2057 kNm

Vu = -26.3396 kN

Has a beam height of 0.4 m. F'c= 25 MPa ; main reinforcement fy = 420 MPa; fy Sengkang = 280 MPa. The column has a main reinforcement diameter of 22 mm with a total of 12 reinforcement (set in SpColumn). It is also known that the diameter of the stirrup is 12 mm. Planned reinforcement 12D22.

The "ØMn" column output due to Pu max and Pu min from the SPColumn software can be seen in Figure 2.28

No	Pu	Mux	Muy	φMnx	φMny	φMn/Mu	NA Depth	dt Depth	εt	φ
	kN	kNm	kNm	kNm	kNm		mm	mm		
1	102	-100	-99	-267,56	-264,88	2,676	236	441	0,0054	0,9
2	537	90	100	262,97	292,19	2,922	275	441	0,00422	0,833

	ϕM_{nx}	ϕM_{ny}	ϕ	ϕM_{nx}	ϕM_{ny}
	kNm	kNm		kNm	kNm
Pu max	-267,56	-264,88	0,9	-297,2888889	-294,3111111
Pu min	262,97	292,19	0,8330	315,6902761	350,7683073

Figure 2. 28 1st Floor Main Beam 1 SPColumn Output

Mnc a = 297.289 kNm

Mnc b =315.690 kNm

MPR BI 1

Mprb, ki (-) = 50.5112 kNm

Mprb, ka (+) = 50.5112 kNm

(Mnc a + Mnc b)≥1,2(Mprb ki + Mprb ka)

(297.289 + 315.690) ≥1,2(50.5112 + 50.5112)

612.971 kNm ≥ 121.226 kNm

Because Mnc > 1.2 Mprb, the column meets the requirements (*Strong Column Weak Beam*)

The transverse reinforcement is based on column shear strength and concrete core confinement as follows:

a. From structural analysis, Vu = -26.3396 kNm

b. Based on Mpr beam left and right column:

Mprb, ki (-)	= 50.511 kNm
Mprb, ki (+)	= 50.511 kNm
Mprb, ka (-)	= 50.511 kNm
Mprb, ka (+)	= 50.511 kNm
Mprk of block	= 0.5x(50.511 + 50.511)
Mprk	= 50.511 kNm

Calculating the necessary shear strength in the following way:

$V_e = \frac{181,4839 + 181,4839}{(5-0,6)}$ = 28.061 kN

Value of Ve	= 28.061 kN

28.061 kN > Vu from structural analysis = 26.3396 kN

Then use Vu	= Ve = 28.061 kN
Diameter of stirrups	= 12 mm
Concrete cover	= 40 mm
D	= 500 – 40 – 12/2
	= 454 mm

The shear strength of concrete is neglected Vc = 0 (because Ve > Vu)

$$V_s = \frac{V_u}{\emptyset} - 0 = \frac{82{,}4926 \times 1000}{0{,}75} \quad = 37415.7037 \text{ N}$$

$$\frac{A_v}{s} = \frac{V_s}{f_{yt}d} = \frac{109990{,}2424}{280 \times 453{,}5} \quad = 0.2943 \text{ mm}^2\text{/mm} \quad \text{.......... (A)}$$

Calculations for restraint reinforcement by:

For Pu = -102895.9 N < 0.3f'c

Ag = 0.3 x 25 x 500 x 500

= 1875000 N

fc' = 25 Mpa < 70 Mpa, use the equations:

$$\frac{A_{sh}}{Sb_c} = 0{,}3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$$

$$\frac{A_{sh}}{Sb_c} = 0{,}09\frac{f'_c}{f_{yt}}$$

Bc = column width – concrete cover

= 500 – 2 x 40 = 420 mm

Ag = 500 x 500

= 250000 mm2

Ach = (b – 2 cover) x (h – 2 cover)

= (500 – 2 x 40) x (500 – 2 x 40)

= 176400 mm2

$$\frac{A_{sh}}{Sb_c} = 0{,}3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$$

$$= 0{,}3\left(\frac{250000}{176400} - 1\right)\frac{25}{280} = 0{,}0111758$$

$$\frac{A_{sh}}{s} = 0{,}0111758 \times 420 = 4{,}69387 \text{ mm}^2\text{/mm} \quad \text{.......... (B)}$$

$$\frac{A_{sh}}{Sb_c} = 0{,}09\frac{f'_c}{f_{yt}}$$

$$= 0{,}09\frac{25}{280} = 0{,}0070714$$

$\frac{A_{sh}}{S}$ $= 0{,}0070714 \times 420 = 2{,}97$ mm^2/mm.. (C)

From (A), (B), and (C) which determine (B)

$\frac{A_{sh}}{S}$ $= 4{,}69387$ mm^2/mm

Calculation of transverse reinforcement in the area along l0 by means of:

For example, taken S = 100 mm

Ash = 4.69387 x 100 = 469.39 mm2

13mm diameter, 1ft wide

Av = ¼ x π x 132 = 132.73 mm2

Number of legs of transverse reinforcement = __ /132.73 =__ > use n = 5

Transverse reinforcement 5D12-100

S max:

a. ¼ smallest column dimension = ¼ × 500 = 125 mm
b. 6 times the diameter of the longitudinal reinforcement = 6 × 22= 132 mm
c. Hx = (500-(2 × 40) - (2 × 13) - (22))/3 = 124 mm
d. $S_0 = 100 + \left(\frac{350\text{-}124}{3}\right) = 175.111$ mm

Thus, a spacing of 100 mm transverse reinforcement meets the requirements

Calculation of transverse reinforcement outside the area l_0 by:

Ve = 28.0617 kN

$V_c = 0{,}17\sqrt{30} b_w d = 0{,}17 \times \sqrt{25} \times 500 \times 453{,}5 = 192{,}7375\ kN$

Vc = 192,7375kN > Ve = 28.0617 kN

The distance of the transverse reinforcement is taken = 100 mm

Then use 5D12-100.

2.9.2 1st Floor Main Beam 2

It is known that the column is 500×500 with a height of 4 m, the results obtained from the etabs are:

Pu max = -109.6624 kN

Mx = 104.1577 kNm

My = 99.3028 kNm

Pu min = -523.5856 kN

Mx = -102.8991 kNm

My = -90.4949 kNm

Vu = 28.4109 kN

Has a beam height of 0.4 m. F'c= 25 MPa ; main reinforcement fy = 420 MPa; fy Sengkang = 280 MPa. The column has a main reinforcement diameter of 22 mm with a total of 12 reinforcement (set in SpColoumn). It is also known that the diameter of the stirrup is 12 mm. Planned reinforcement 12D22.

The "ØMn" column output due to Pu max and Pu min from the SPColumn software can be seen in Figure 2.29

No	Pu	Mux	Muy	ϕMnx	ϕMny	ϕMn/Mu	NA Depth	dt Depth	εt	ϕ
	kN	kNm	kNm	kNm	kNm		mm	mm		
1	109	-99	-104	-260,22	-273,36	2.629	237	661	0,00538	0,9
2	523	90	102	260,28	294,98	2.892	273	661	0,00425	0,836

	ϕM_{nx}	ϕM_{ny}	ϕ	ϕM_{nx}	ϕM_{ny}
	kNm	kNm		kNm	kNm
Pu max	-260,22	-273,36	0,9	-289,1333333	-303,7333333
Pu min	260,28	294,98	0,8360	311,3397129	352,84689

Figure 2. 29 1st Floor Main Beam 2 SPColumn Output

Mnc a = 289.1333 kNm

Mnc b = 311.3397 kNm

MPR BI 1

Mprb, ki (-) = 50.5112 kNm

Mprb, ka (+) = 50.5112 kNm

(Mnc a + Mnc b)≥1,2(Mprb ki + Mprb ka)

(289.1333 + 311.3397) ≥1,2(50.5112 + 50.5112)

600,473 kNm ≥ 121,226 kNm

Because Mnc > 1.2 Mprb, the column meets the requirements (*Strong Column Weak Beam*)

The transverse reinforcement is based on column shear strength and concrete core confinement as follows:

a. From structural analysis Vu = 26.4109 kNm

b. Based on Mpr beam left and right column:

Mprb, ki (-) = 50.5112 kNm

Mprb, ki (+) = 50.5112 kNm

Mprb, ka (-) = 50.5112 kNm

Mprb, ka (+) = 50.5112 kNm

Mprk of block = 0.5x(50.5112 + 50.5112)

Mprk = 50.5112 kNm

Calculating the necessary shear strength in the following way:

$$V_e = \frac{50.5112 + 50.5112}{(4-0,35)} = 27.6773 \text{ kN}$$

Value of Ve = 27.6773 kN

27.6773 kN > Vu from structural analysis = 26.4109 kN

Then use Vu = Ve = 26.4109 kN

Diameter of stirrup = 12 mm

Concrete cover = 40 mm

D = 500 – 40 – 12/2 = 454 mm

The shear strength of concrete is neglected Vc = 0 (because Ve > Vu)

$$V_s = \frac{V_u}{\phi} - 0 = \frac{26,4109 \times 1000}{0,75} = 36903.15982 \text{ N}$$

$$\frac{A_v}{s} = \frac{V_s}{f_{yt} d} = \frac{36903.15982}{280 \times 454} = 0,2903 \text{ mm}^2/\text{mm} \quad \text{...} \quad (A)$$

Calculations for restraint reinforcement by:

For Pu = -109662,4 N < 0.3f'c

Ag = 0.3 x 25 x 500 x 500

= 1875000 N

fc' = 25 Mpa < 70 Mpa, use the equations:

$$\frac{A_{sh}}{Sb_c} = 0{,}3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$$

$$\frac{A_{sh}}{Sb_c} = 0{,}09\frac{f'_c}{f_{yt}}$$

Bc = column width – concrete cover

= 500 – 2 x 40

= 420 mm

Ag = 500 x 500

= 250000 mm2

Ach = (b – 2 cover) x (h – 2 cover)

= (500 – 2 x 40) x (500 – 2 x 40)

= 176400 mm2

$$\frac{A_{sh}}{Sb_c} = 0{,}3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$$

$$= 0{,}3\left(\frac{250000}{176400}-1\right)\frac{25}{280} = 0{,}0111758$$

$$\frac{A_{sh}}{S} = 0{,}0111758\times420 = 4{,}69387\ mm^2/mm \ldots\ldots\ldots\ldots (B)$$

$$\frac{A_{sh}}{Sb_c} = 0{,}09\frac{f'_c}{f_{yt}} = 0{,}09\frac{25}{280} = 0{,}0070714$$

$$\frac{A_{sh}}{S} = 0{,}0070714\times420 = 2{,}97\ mm^2/mm \ldots\ldots\ldots\ldots (C)$$

From (A), (B), and (C) which determine (B)

$$\frac{A_{sh}}{S} = 4{,}69387\ mm^2/mm$$

Calculation of transverse reinforcement in the area along l0 by means of:

For example, taken S = 100 mm

Ash = 4.69387 x 100 = 469.39 mm2

12 mm diameter, 1ft wide

Av = ¼ x π x 12^2 = 113.10 mm2

Number of legs of transverse reinforce = 469.39 /113.10 =175,11 > use n = 5

Transverse reinforcement 5D12-100.

S max:

a. ¼ smallest column dimension = ¼ × 500 = 125 mm
b. 6 times the diameter of the longitudinal reinforcement = 6 × 22= 132 mm
c. Hx = (500-(2 × 40) - (2 × 13) - (22))/3 = 124 mm
d. $S_0 = 100 + \left(\frac{350\text{-}124}{3}\right)$ =175.333 mm

Thus, a spacing of 100 mm transverse reinforcement meets the requirements.

Calculation of transverse reinforcement outside the area l_0 by:

Ve = 82,49268 kN

$V_c = 0{,}17\sqrt{30} b_w d = 0{,}17 \times \sqrt{25} \times 500 \times 453{,}5 = 192{,}7375\ kN$

Vc = 192,7375kN > Ve = 82,49268 kN

The distance of the transverse reinforcement is taken = 100 mm

Then use 5D12-100.

2.9.3 1st Floor Main Beam 3

It is known that the column is 500×500 with a height of 4 m, the results obtained from the etabs are:

Pu max	= -128.8505 kN
Mx	= 93.5851 kNm
My	= 109.8649 kNm
Pu min	= -440.3568 kN
Mx	= -104.0993 kNm
My	= -79.085kNm
Vu	= - 29.6004 kN

Has a beam height of 0.4 m. F'c= 25 MPa ; main reinforcement fy = 420 MPa; fy Sengkang = 280 MPa. The column has a main reinforcement diameter of 22 mm with a total of 12 reinforcement (set in SpColoumn). It is also known that the diameter of the stirrup is 12 mm. Planned reinforcement 12D22.

The "ØMn" column output due to Pu max and Pu min from the SPColumn software can be seen in Figure 2.30

No	Pu	Mux	Muy	φMnx	φMny	φMn/Mu	NA Depth	dt Depth	εt	φ
	kN	kNm	kNm	kNm	kNm		mm	mm		
1	128	-109	-93	-289,6	-247,09	2,657	238	661	0,00532	0,9
2	440	79	104	240,31	315,36	3,042	261	655	0,00453	0,86

	ϕM_{nx}	ϕM_{ny}	ϕ	ϕM_{nx}	ϕM_{ny}
	kNm	kNm		kNm	kNm
Pu max	-289,6	-247,09	0,9	-321,7777778	-274,5444444
Pu min	240,31	316,36	0,8600	279,4302326	367,8604651

Figure 2. 30 1st Floor Main Beam 3 SPColumn Output

Mnc a = 321.7777 kNm

Mnc b = 279,4302 kNm

MPR BI 1

Mprb, ki (-) = 24,2117 kNm

Mprb, ka (+) = 24,2117 kNm

(Mnc a + Mnc b)≥1,2(Mprb ki + Mprb ka)

(321.7777 + 279,4302) ≥1,2(24,2117 + 24,2117)

601.2080 kNm ≥ 58.1080 kNm

Because Mnc > 1.2 Mprb, the column meets the requirements (*Strong Column Weak Beam*)

The transverse reinforcement is based on column shear strength and concrete core confinement as follows:

a. From structural analysis, Vu = - 29.6004 kNm

b. Based on Mpr beam left and right column:

Mprb, ki (-) = 24.2117 kNm

Mprb, ki (+) = 24.2117 kNm

Mprb, ka (-) = 24.2117 kNm

Mprb, ka (+) = 24.2117 kNm

Mprk of block = 0.5x(24.2117 + 24.2117)

Mprk = 24.2117 kNm

Calculating the necessary shear strength in the following way:

$V_e = \frac{24.2117 + 24.2117}{(4\text{-}0{,}3)}$ = 13.0874 kN

Value of Ve = 13,0874 kN

13,0874 kN > Vu from structural analysis = - 29.6004 kN

Then use Vu = Ve = 13.0874 kN

Diameter of stirrup = 12 mm

Concrete cover = 40 mm

D = 500 – 40 – 12/2

= 454 mm

The shear strength of concrete is neglected Vc = 0 (because Ve > Vu)

$V_s = \frac{V_u}{\emptyset} - 0 = \frac{13.0874 \times 1000}{0.75}$ = 17449.87387 N

$\frac{A_v}{s} = \frac{V_s}{f_{yt} d} = \frac{-}{280 \times 454}$ = 0.137270877 mm^2/mm................................. (A)

Calculations for restraint reinforcement by:

For Pu = -128850,5 N < 0.3f'c

Ag = 0.3 x 25 x 500 x 500 = 1875000 N

fc' = 25 Mpa < 70 Mpa, use the equations:

$$\frac{A_{sh}}{Sb_c} = 0{,}3 \left(\frac{A_g}{A_{ch}} - 1\right) \frac{f'_c}{f_{yt}}$$

$$\frac{A_{sh}}{Sb_c} = 0{,}09 \frac{f'_c}{f_{yt}}$$

Bc = column width – concrete cover

= 500 – 2 x 40

= 420 mm

Ag = 500 x 500

= 250000 mm2

Ach = (b – 2 cover) x (h – 2 cover)

= (500 – 2 x 40) x (500 – 2 x 40)

= 176400 mm2

$$\frac{A_{sh}}{Sb_c} = 0{,}3\left(\frac{A_g}{A_{ch}}\text{-}1\right)\frac{f'_c}{f_{yt}}$$

$$= 0{,}3\left(\frac{250000}{176400}\text{-}1\right)\frac{25}{280} = 0{,}0111758$$

$$\frac{A_{sh}}{s} = 0{,}0111758 \times 420 = 4{,}69387\ \text{mm}^2/\text{mm} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(B)}$$

$$\frac{A_{sh}}{Sb_c} = 0{,}09\frac{f'_c}{f_{yt}}$$

$$= 0{,}09\frac{25}{280} = 0{,}0070714$$

$$\frac{A_{sh}}{s} = 0{,}0070714 \times 420 = 2{,}97\ \text{mm}^2/\text{mm} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(C)}$$

From (A), (B), and (C) which determine (B)

$$\frac{A_{sh}}{s} = 4{,}69387\ \text{mm}^2/\text{mm}$$

Calculation of transverse reinforcement in the area along l0 by means of:

For example, taken S = 100 mm

Ash = 4.69387 x 100 = 469.39 mm2

12 mm diameter, 1ft wide

Av = ¼ x π x 12^2 = 113.10 mm2

Number of legs of transverse reinforce = 469.39 /113.10 =175,11 > use n = 5

Transverse reinforcement 5D12-100.

S max:

a. ¼ smallest column dimension = ¼ × 500 = 125 mm
b. 6 times the diameter of the longitudinal reinforcement = 6 × 22= 132 mm
c. Hx = (500-(2 × 40) - (2 × 13) - (22))/3 = 124 mm
d. $S_0 = 100+\left(\frac{350\text{-}124}{3}\right)$=175,333 mm

Thus, a spacing of 100 mm transverse reinforcement meets the requirements.

Calculation of transverse reinforcement outside the area l_0 by:

Ve = 13.0874 kN

$V_c = 0{,}17\sqrt{30} b_w d = 0{,}17 \times \sqrt{25} \times 500 \times 453{,}5 = 192{,}7375\ kN$

Vc = 192,7375kN > Ve = 13.0874 kN

The distance of the transverse reinforcement is taken = 100 mm

Then use 5D12-100.

2.9.4 1st Floor Main Beam 4

It is known that the column is 500×500 with a height of 4 m, the results obtained from the etabs are:

Pu max = 44.4976 kN

Mx = 99.0581 kNm

My = 95.9858 kNm

Pu min = -315.4937 kN

Mx = -102.178 kNm

My = -93.2218 kNm

Vu = -27.4684 kN

Has a beam height of 0.4 m. F'c= 25 MPa ; main reinforcement fy = 420 MPa; fy Sengkang = 280 MPa. The column has a main reinforcement diameter of 22 mm with a total of 12 reinforcement (set in SpColoumn). It is also known that the diameter of the stirrup is 12 mm. Planned reinforcement 12D22.

The "ØMn" column output due to Pu max and Pu min from the SPColumn software can be seen in Figure 2.31

No	Pu kN	Mux kNm	Muy kNm	φMnx kNm	φMny kNm	φMn/Mu	NA Depth mm	dt Depth mm	εt	φ
1	-44	-95	-95	-248,9	-259,38	2,620	224	[illegible]	0,00584	0,9
2	315	93	102	265,54	291,24	2,855	254	[illegible]	0,0048	0,883

	ϕM_{nx}	ϕM_{ny}	ϕ	ϕM_{nx}	ϕM_{ny}
	kNm	kNm		kNm	kNm
Pu max	-248,9	-259,38	0,9	-276,5555556	-288,2
Pu min	265,54	291,24	0,8830	300,7248018	329,8301246

Figure 2. 31 1st Floor Main Beam 4 SPColumn Output

Mnc a = 276.5555 kNm

Mnc b = 300.7248 kNm

MPR BI 1

Mprb, ki (-) = 24.2117 kNm

Mprb, ka (+) = 24.2117kNm

(Mnc a + Mnc b) ≥ 1,2(Mprb ki + Mprb ka)

(276.5555 + 300.7248) ≥ 1,2(24.2117 + 24.2117)

577.2803 kNm ≥ 58.1080 kNm

Because Mnc > 1.2 Mprb, the column meets the requirements (*Strong Column Weak Beam*)

The transverse reinforcement is based on column shear strength and concrete core confinement as follows:

a. From structural analysis Vu = -27,4684 kNm

b. Based on Mpr beam left and right column:

Mprb, ki (-) = 24.2117 kNm

Mprb, ki (+) = 24.2117 kNm

Mprb, ka (-) = 24.2117 kNm

Mprb, ka (+) = 24.2117 kNm

Mprk of block = 0.5x(24.2117 + 24.2117)

Mprk = 24.2117 kNm

Calculating the necessary shear strength in the following way:

$V_e = \frac{24.2117 + 24.2117}{(4-0,3)}$ = 12.9129 kN

Value of Ve = 12.9129 kN

12.9129 kN > Vu from structural analysis = -27,4684 kN

Then use Vu = Ve = 12.9129 kN

Diameter of stirrup = 12 mm

Concrete cover = 40 mm

D = 500 – 40 – 12/2

= 454 mm

The shear strength of concrete is neglected Vc = 0 (because Ve > Vu)

$V_s = \frac{V_u}{\emptyset} - 0 = \frac{12.9129 \times 1000}{0,75}$ = 17217.20889 N

$\frac{A_v}{s} = \frac{V_s}{f_{yt}d} = \frac{17217.20889}{280 \times 454}$ = 0.135440599 mm^2/mm .... (A)

Calculations for restraint reinforcement by:

For Pu = 44497,6 N < 0.3f'c

Ag = 0.3 x 25 x 500 x 500

= 1875000 N

fc' = 25 Mpa < 70 Mpa, use the equations:

$$\frac{A_{sh}}{Sb_c} = 0,3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$$

$$\frac{A_{sh}}{Sb_c} = 0,09\frac{f'_c}{f_{yt}}$$

Bc = column width – concrete cover

= 500 – 2 x 40 = 420 mm

Ag = 500 x 500

= 250000 mm2

Ach = (b – 2 cover) x (h – 2 cover)

= (500 – 2 x 40) x (500 – 2 x 40)

= 176400 mm2

$$\frac{A_{sh}}{Sb_c} = 0,3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$$

$$= 0,3\left(\frac{250000}{176400}\text{-}1\right)\frac{25}{280} = 0,0111758$$

$\frac{A_{sh}}{S}$ $= 0,0111758 \times 420 = 4,69387\ mm^2/mm$ .... (B)

$$\frac{A_{sh}}{Sb_c} = 0,09\frac{f'_c}{f_{yt}}$$

$$= 0,09\frac{25}{280} = 0,0070714$$

$\frac{A_{sh}}{S}$ $= 0,0070714 \times 420 = 2,97\ mm^2/mm$ .... (C)

From (A), (B), and (C) which determine (B)

$\frac{A_{sh}}{S}$ $= 4,69387\ mm^2/mm$

Calculation of transverse reinforcement in the area along l0 by means of:

For example, taken S = 100 mm

Ash = 4.69387 x 100 = 469.39 mm2

12 mm diameter, 1ft wide

Av = ¼ x π x 12^2 = 113.10 mm2

Number of legs of transverse reinforce = 469.39 /113.10 =175,11 > use n = 5

Transverse reinforcement 5D12-100.

S max:

a. ¼ smallest column dimension = ¼ × 500 = 125 mm
b. 6 times the diameter of the longitudinal reinforcement = 6 × 22= 132 mm
c. Hx = (500-(2 × 40) - (2 × 13) - (22))/3 = 124 mm
d. $S_0 = 100+\left(\frac{350\text{-}124}{3}\right)$=175,333 mm

Thus, a spacing of 100 mm transverse reinforcement meets the requirements

Calculation of transverse reinforcement outside the area l_0 by:

Ve = 12.9129 kN

V_c=$0,17\sqrt{30}b_w d = 0,17\times\sqrt{25}\times500\times453,5 = 192,7375\ kN$

Vc = 192,7375kN > Ve = 12.9129 kN

The distance of the transverse reinforcement is taken = 100 mm

Then use 5D12-100.

2.9.5 1st Floor Main Beam 5

It is known that the column is 500×500 with a height of 4 m, the results obtained from the etabs are:

Pu max = -172,1264 kN

Mx = 82,1241 kNm

My = 99,3636 kNm

Pu min = -514,4654 kN

Mx = -119,4241 kNm

My = -85,3571 kNm

Vu = -42,1734 kN

Has a beam height of 0.4 m. F'c= 25 MPa ; main reinforcement fy = 420 MPa; fy Sengkang = 280 MPa. The column has a main reinforcement diameter of 22 mm with a total of 12 reinforcement (set in SpColoumn). It is also known that the diameter of the stirrup is 12 mm. Planned reinforcement 12D22.

The "ØMn" column output due to Pu max and Pu min from the SPColumn software can be seen in Figure 2.32

No	Pu	Mux	Muy	ϕM_{nx}	ϕM_{ny}	$\phi M_n/M_u$	NA Depth	dt Depth	ε_t	ϕ
	kN	kNm	kNm	kNm	kNm		mm	mm		
1	172	-99	-82	-256,13	-212,15	2,587	230	461	0,00564	0,9
2	514	85	119	210,2	294,28	2,473	250	453	0,00466	0,871

	ϕM_{nx}	ϕM_{ny}	ϕ	ϕM_{nx}	ϕM_{ny}
	kNm	kNm		kNm	kNm
Pu max	-256,13	-212,15	0,9	-284,5888889	-235,7222222
Pu min	210,2	294,28	0,8710	241,3318025	337,8645235

Figure 2. 32 1st Floor Main Beam 5 SPColumn Output

Mnc a = 284.5888 kNm

Mnc b = 241.3318 kNm

MPR BI 1

Mprb, ki (-) = 24.2117 kNm

Mprb, ka (+) = 24.2117 kNm

(Mnc a + Mnc b) ≥ 1,2(Mprb ki + Mprb ka)

(284.5888 + 241.3318) ≥ 1.2(24.2117 + 24.2117)

525.9206 kNm ≥ 58.1080 kNm

Because Mnc > 1.2 Mprb, the column meets the requirements (*Strong Column Weak Beam*)

The transverse reinforcement is based on column shear strength and concrete core confinement as follows:

a. From structural analysis Vu = -42.1734 kNm

b. Based on Mpr beam left and right column:

Mprb, ki (-) = 24.2117 kNm

Mprb, ki (+) = 24.2117 kNm

Mprb, ka (-) = 24.2117 kNm

Mprb, ka (+) = 24.2117 kNm

Mprk of block = 0.5x(24.2117 + 24.2117)

Mprk = 24.2117 kNm

Calculating the necessary shear strength in the following way:

$V_e = \frac{24.2117 + 24.2117}{(4\text{-}0,2)}$ = 12.743 kN

Value of Ve = 12.743 kN

12.743 kN > Vu from structural analysis = -42.1734 kN

Then use Vu = Ve = 12.743 kN

Diameter of stirrup = 12 mm

Concrete cover = 40 mm

D = 500 – 40 – 12/2

= 454 mm

The shear strength of concrete is neglected Vc = 0 (because Ve > Vu)

$V_s = \frac{V_u}{\emptyset} - 0 = \frac{12.743 \times 1000}{0,75}$ = 16990.66667 N

$$\frac{A_v}{s} = \frac{V_s}{f_{yt}d} = \frac{16990.66667}{280 \times 454} = 0.133658485 \text{ mm}^2/\text{mm} \quad \text{......} (A)$$

Calculations for restraint reinforcement by:

For Pu = -172126.4 N < 0.3f'c

Ag = 0.3 x 25 x 500 x 500 = 1875000 N

fc' = 25 Mpa < 70 Mpa, use the equations:

$$\frac{A_{sh}}{Sb_c} = 0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$$

$$\frac{A_{sh}}{Sb_c} = 0.09\frac{f'_c}{f_{yt}}$$

Bc = column width – concrete cover

= 500 – 2 x 40

= 420 mm

Ag = 500 x 500

= 250000 mm2

Ach = (b – 2 cover) x (h – 2 cover)

= (500 – 2 x 40) x (500 – 2 x 40) = 176400 mm2

$$\frac{A_{sh}}{Sb_c} = 0{,}3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$$

$$= 0{,}3\left(\frac{250000}{176400} - 1\right)\frac{25}{280} = 0{,}0111758$$

$$\frac{A_{sh}}{S} = 0{,}0111758 \times 420 = 4{,}69387 \text{ mm}^2/\text{mm} \quad \text{......} (B)$$

$$\frac{A_{sh}}{Sb_c} = 0{,}09\frac{f'_c}{f_{yt}}$$

$$= 0{,}09\frac{25}{280} = 0{,}0070714$$

$$\frac{A_{sh}}{S} = 0{,}0070714 \times 420 = 2{,}97 \text{ mm}^2/\text{mm} \quad \text{......} (C)$$

From (A), (B), and (C) which determine (B)

$\frac{A_{sh}}{S}$ = 4,69387 mm^2/mm

Calculation of transverse reinforcement in the area along l0 by means of:

For example, taken S = 100 mm

Ash = 4.69387 x 100 = 469.39 mm2

12 mm diameter, 1ft wide

Av = ¼ x π x 12^2 = 113.10 mm2

Number of legs of transverse reinforce = 469.39 /113.10 =175,11 > use n = 5

Transverse reinforcement 5D12-100.

S max:

a. ¼ smallest column dimension = ¼ × 500 = 125 mm
b. 6 times the diameter of the longitudinal reinforcement = 6 × 22= 132 mm
c. Hx = (500-(2 × 40) - (2 × 13) - (22))/3 = 124 mm
d. $S_0 = 100 + \left(\frac{350\text{-}124}{3}\right)$ =175,333 mm

Thus, a spacing of 100 mm transverse reinforcement meets the requirements.

Calculation of transverse reinforcement outside the area l_0 by:

Ve = 12.743 kN

$V_c = 0{,}17\sqrt{30} b_w d = 0{,}17 \times \sqrt{25} \times 500 \times 453{,}5 = 192{,}7375\ kN$

Vc = 192,7375kN > Ve = 12.743 kN

The distance of the transverse reinforcement is taken = 100 mm

Then use 5D12-100.

2.9.6 2nd Floor Main Beam 1

It is known that the column is 500×500 with a height of 4 m, the results obtained from the etabs are:

Pu max = -65.4271 kN

Mx = 44.96 kNm

My = 59.9926 kNm

Pu min = -356.9387 kN

Mx = -47.6435 kNm

My = -42.5789 kNm

Vu = -21.6853 kN

Has a beam height of 0.4 m. F'c= 25 MPa ; main reinforcement fy = 420 MPa; fy Sengkang = 280 MPa. The column has a main reinforcement diameter of 22 mm with a total of 12 reinforcement (set in SpColoumn). It is also known that the diameter of the stirrup is 12 mm. Planned reinforcement 12D22.

The "ØMn" column output due to Pu max and Pu min from the SPColumn software can be seen in Figure 2.33

No	Pu	Mux	Muy	φMnx	φMny	φMn/Mu	NA Depth	dt Depth	εt	φ
	kN	kNm	kNm	kNm	kNm		mm	mm		
1	65	-59	-44	-299,21	-223,54	5.071	227	653	0.00563	0,9
2	356	42	47	262,97	294,27	6.261	258	661	0.00465	0,873

	ϕM_{nx}	ϕM_{ny}	ϕ	ϕM_{rx}	ϕM
	kNm	kNm		kNm	kN
Pu max	-299,21	-223,14	0,9	-332,4555556	-247,933333
Pu min	262,97	294,27	0,8730	301,2256586	337,079037

Figure 2. 33 2nd Floor Main Beam 1 SPColumn Output

Mnc a = 332.4555 kNm

Mnc b = 301.2256 kNm

MPR BI 1

Mprb, ki (-) = 32.4264 kNm

Mprb, ka (+) = 32.4264 kNm

(Mnc a + Mnc b) ≥ 1,2(Mprb ki + Mprb ka)

(332.4555 + 301.2256) ≥ 1,2(32.4264 + 32.4264)

633.6812 kNm ≥ 77.8233 kNm

Because Mnc > 1.2 Mprb, the column meets the requirements (*Strong Column Weak Beam*)

The transverse reinforcement is based on column shear strength and concrete core confinement as follows:

a. From structural analysis Vu = -21.6853 kNm

b. Based on Mpr beam left and right column:

Mprb, ki (-) = 32.4264 kNm

Mprb, ki (+) = 32.4264 kNm

Mprb, ka (-) = 32.4264 kNm

Mprb, ka (+) = 32.4264 kNm

Mprk of block = 0.5x(32.4264 + 32.4264)

Mprk = 32.4264 kNm

Calculating the necessary shear strength in the following way:

$V_e = \frac{32.4264 + 32.4264}{(4-0.4)}$ = 18.0146 kN

Value of Ve = 18.0146 kN

18.0146 kN > Vu from structural analysis = -21.6853 kN

Then use Vu = Ve = -21.6853 kN

Diameter of stirrup = 12 mm

Concrete cover = 40 mm

D = 500 – 40 – 12/2

= 454 mm

The shear strength of concrete is neglected Vc = 0 (because Ve > Vu)

$V_s = \frac{V_u}{\emptyset} - 0 = \frac{18.0146 \times 1000}{0.75}$ = 24019.5555 N

$\frac{A_v}{s} = \frac{V_s}{f_{yt}d} = \frac{24019.5}{280 \times 453{,}5}$ = 0.188951822 mm^2/mm .... (A)

Calculations for restraint reinforcement by:

For Pu = -65427.1 N < 0.3f'c

Ag = 0.3 x 25 x 500 x 500

= 1875000 N

fc' = 25 Mpa < 70 Mpa, use the equations:

$$\frac{A_{sh}}{Sb_c} = 0{,}3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$$

$$\frac{A_{sh}}{Sb_c} = 0{,}09\frac{f'_c}{f_{yt}}$$

Bc = column width – concrete cover

= 500 – 2 x 40

= 420 mm

Ag = 500 x 500

= 250000 mm2

Ach = (b – 2 cover) x (h – 2 cover)

= (500 – 2 x 40) x (500 – 2 x 40)

= 176400 mm2

$$\frac{A_{sh}}{Sb_c} = 0{,}3\left(\frac{A_g}{A_{ch}}\text{-}1\right)\frac{f'_c}{f_{yt}}$$

$$= 0{,}3\left(\frac{250000}{176400}\text{-}1\right)\frac{25}{280} = 0{,}0111758$$

$$\frac{A_{sh}}{S} = 0{,}0111758 \times 420 = 4{,}69387\ \text{mm}^2/\text{mm} \ldots\ldots\ldots\ldots (B)$$

$$\frac{A_{sh}}{Sb_c} = 0{,}09\frac{f'_c}{f_{yt}}$$

$$= 0{,}09\frac{25}{280} = 0{,}0070714$$

$$\frac{A_{sh}}{S} = 0{,}0070714 \times 420 = 2{,}97\ \text{mm}^2/\text{mm} \ldots\ldots\ldots\ldots (C)$$

From (A), (B), and (C) which determine (B)

$$\frac{A_{sh}}{S} = 4{,}69387\ \text{mm}^2/\text{mm}$$

Calculation of transverse reinforcement in the area along l0 by means of:

For example, taken S = 100 mm

Ash = 4.69387 x 100 = 469.39 mm2

12 mm diameter, 1ft wide

Av = ¼ x π x 12^2 = 113.10 mm2

Number of legs of transverse reinforce = 469.39 /113.10 =175,11 > use n = 5

Transverse reinforcement 5D12-100

S max:

a. ¼ smallest column dimension = ¼ × 500 = 125 mm
b. 6 times the diameter of the longitudinal reinforcement = 6 × 22= 132 mm
c. Hx = (500-(2 × 40) - (2 × 13) - (22))/3 = 124 mm
d. $S_0 = 100+\left(\frac{350\text{-}124}{3}\right)$=175,333 mm

Thus, a spacing of 100 mm transverse reinforcement meets the requirements

Calculation of transverse reinforcement outside the area l_0 by:

Ve = 18.0146 kN

V_c= $0.17\sqrt{30}b_w d$=$0.17\times\sqrt{25}\times500\times453.5$ = 192.7375 kN

Vc = 192.7375kN > Ve = 18.0146 kN

The distance of the transverse reinforcement is taken = 100 mm

Then use 5D12-100.

2.9.7 2nd Floor Main Beam 2

It is known that the column is 500×500 with a height of 4 m, the results obtained from the etabs are:

Pu max	= -69.8864 kN
Mx	= 55,9191 kNm
My	= 57.3015 kNm
Pu min	= -348.0503 kN
Mx	= -53,6283 kNm
My	= -40,6711 kNm
Vu	= 25,3576 kN

Has a beam height of 0.4 m. F'c= 25 MPa ; main reinforcement fy = 420 MPa; fy Sengkang = 280 MPa. The column has a main reinforcement diameter of 22 mm with a total of 12 reinforcement (set in SpColoumn). It is also known that the diameter of the stirrup is 12 mm. Planned reinforcement 12D22.

The "ØMn" column output due to Pu max and Pu min from the SPColumn software can be seen in Figure 2.34

No	Pu	Mux	Muy	φMnx	φMny	φMn/Mu	NA Depth	dt Depth	εt	φ
	kN	kNm	kNm	kNm	kNm		mm	mm		
1	69	-57	-55	-268,21	-258,8	4,706	233	661	0,0055	0,9
2	348	40	53	239,62	317,5	5,991	252	654	0,00478	0,881

	ϕM_{nx}	ϕM_{ny}	ϕ	ϕM_{ex}	ϕM_{ey}
	kNm	kNm		kNm	kNm
Pu max	268,21	258,8	0,9	298,0111111	287,5555556
Pu min	239,62	317,5	0,8810	271,9863791	360,3859251

Figure 2. 34 2nd Floor Main Beam 2 SPColumn Output

Mnc a = 298.0111 kNm

Mnc b = 271.9863 kNm

MPR BI 1

Mprb, ki (-) = 32.4264 kNm

Mprb, ka (+) = 32.4264 kNm

(Mnc a + Mnc b) ≥ 1,2 (Mprb ki + Mprb ka)

(298.0111 + 271.9863) ≥ 1,2(32.4264 + 32.4264)

569.9974 kNm ≥ 77.8233 kNm

Because Mnc > 1.2 Mprb, the column meets the requirements (*Strong Column Weak Beam*)

The transverse reinforcement is based on column shear strength and concrete core confinement as follows:

a. From structural analysis Vu = 16.3576 kNm

b. Based on Mpr beam left and right column:

Mprb, ki (-) = 32.4264 kNm

Mprb, ki (+) = 32.4264 kNm

Mprb, ka (-) = 32.4264 kNm

Mprb, ka (+) = 32.4264 kNm

Mprk of block = 0.5x(32.4264 + 32.4264)

Mprk = 32.4264 kNm

Calculating the necessary shear strength in the following way:

$V_e = \frac{32.4264 + 32.4264}{(4\text{-}0,35)}$ = 17.7678 kN

Value of Ve = 17.7678 kN

17.7678 kN > Vu from structural analysis = 16,3576 kN

Then use Vu = Ve = 17.7678 kN

Diameter of stirrup = 12 mm

Concrete cover = 40 mm

D = 500 – 40 – 12/2

= 454 mm

The shear strength of concrete is neglected Vc = 0 (because Ve > Vu)

$V_s = \frac{V_u}{\emptyset} - 0 = \frac{17.7678 \times 1000}{0,75}$ = 23690.52055 N

$\frac{A_v}{s} = \frac{V_s}{f_{yt} d} = \frac{23690.52055}{280 \times 454}$ = 0.18636344 mm^2/mm .... (A)

Calculations for restraint reinforcement by:

For Pu = -69886,4 N < 0.3f'c

Ag = 0.3 x 25 x 500 x 500 = 1875000 N

fc' = 25 Mpa < 70 Mpa, use the equations:

$$\frac{A_{sh}}{Sb_c} = 0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$$

$$\frac{A_{sh}}{Sb_c} = 0.09\frac{f'_c}{f_{yt}}$$

Bc = column width – concrete cover

= 500 – 2 x 40

= 420 mm

Ag = 500 x 500

= 250000 mm2

Ach = (b – 2 cover) x (h – 2 cover)

= (500 – 2 x 40) x (500 – 2 x 40) = 176400 mm2

$$\frac{A_{sh}}{Sb_c} = 0,3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$$

$$= 0,3\left(\frac{250000}{176400}-1\right)\frac{25}{280} = 0,0111758$$

$\frac{A_{sh}}{S}$ $= 0,0111758\times420 = 4,69387\ mm^2/mm$.................................... (B)

$$\frac{A_{sh}}{Sb_c} = 0,09\frac{f'_c}{f_{yt}}$$

$$= 0,09\frac{25}{280} = 0,0070714$$

$\frac{A_{sh}}{S}$ $= 0,0070714\times420 = 2,97\ mm^2/mm$.. (C)

From (A), (B), and (C) which determine (B)

$\frac{A_{sh}}{S}$ $= 4,69387\ mm^2/mm$

Calculation of transverse reinforcement in the area along l0 by means of:

For example, taken S = 100 mm

Ash = 4.69387 x 100 = 469.39 mm2

12 mm diameter, 1ft wide

Av = ¼ x π x 12^2 = 113.10 mm2

Number of legs of transverse reinforce = 469.39 /113.10 =175,11 > use n = 5

Transverse reinforcement 5D12-100

S max:

a. ¼ smallest column dimension = ¼ × 500 = 125 mm
b. 6 times the diameter of the longitudinal reinforcement = 6 × 22= 132 mm
c. Hx = (500-(2 × 40) - (2 × 13) - (22))/3 = 124 mm
d. $S_0 = 100+\left(\frac{350\text{-}124}{3}\right)$=175.333 mm

Thus, a spacing of 100 mm transverse reinforcement meets the requirements.

Calculation of transverse reinforcement outside the area l_0 by:

Ve = 17.7678 kN

$V_c = 0{,}17\sqrt{30} b_w d = 0{,}17 \times \sqrt{25} \times 500 \times 453{,}5 = 192{,}7375\ kN$

Vc = 192,7375kN > Ve = 17.7678 kN

The distance of the transverse reinforcement is taken = 100 mm

Then use 5D12-100.

2.9.8 2nd Floor Main Beam 3

It is known that the column is 500×500 with a height of 4 m, the results obtained from the etabs are:

Pu max = -80.6775 kN

Mx = 37.2992 kNm

My = 80.9504 kNm

Pu min = -293.129 kN

Mx = -58,2935 kNm

My = -51.002 kNm

Vu = -26,2753kN

Has a beam height of 0.4 m. F'c= 25 MPa ; main reinforcement fy = 420 MPa; fy Sengkang = 280 MPa. The column has a main reinforcement diameter of 22 mm with a total of 12 reinforcement (set in SpColoumn). It is also known that the diameter of the stirrup is 12 mm. Planned reinforcement 12D22.

The "ØMn" column output due to Pu max and Pu min from the SPColumn software can be seen in Figure 2.35

No	Pu	Mux	Muy	φMnx	φMny	φMn/Mu	NA Depth	dt Depth	εt	φ
	kN	kNm	kNm	kNm	kNm		mm	mm		
1	80	-80	-37	-344,63	-159,39	4,308	199	112	0,00624	0,9
2	293	51	58	260,35	296,08	5,105	252	561	0,00488	0,888

	ϕM_{nx}	ϕM_{ny}	ϕ	ϕM_{nx}	ϕM_{ny}
	kNm	kNm		kNm	kNm
Pu max	-344,63	-159,39	0,9	-382,9222222	-177,1
Pu min	260,35	296,08	0,8880	293,1869369	333,4234234

Figure 2. 35 2nd Floor Main Beam 3 SPColumn Output

Mnc a = 382.9222 kNm

Mnc b = 293.1869 kNm

MPR BI 1

Mprb, ki (-) = 24.2117 kNm

Mprb, ka (+) = 24.2117 kNm

(Mnc a + Mnc b) ≥ 1,2(Mprb ki + Mprb ka)

(382.9222 + 293.1869) ≥ 1,2(24.2117 + 24.2117)

676.1091 kNm ≥ 58.1080 kNm

Because Mnc > 1.2 Mprb, the column meets the requirements (*Strong Column Weak Beam*)

The transverse reinforcement is based on column shear strength and concrete core confinement as follows:

a. From structural analysis Vu = -29.6004 kNm

b. Based on Mpr beam left and right column:

Mprb, ki (-) = 24.2117 kNm

Mprb, ki (+) = 24.2117 kNm

Mprb, ka (-) = 24.2117 kNm

Mprb, ka (+) = 24.2117 kNm

Mprk of block = 0.5x(24.2117 + 24.2117)

Mprk = 24.2117 kNm

Calculating the necessary shear strength in the following way:

$V_e = \frac{24.2117 + 24.2117}{(4-0,3)}$ = 13.0874 kN

Value of Ve = 13.0874 kN

13.0874 kN > Vu from structural analysis = -29.6004 kN

Then use Vu = Ve = 13.0874 kN

Diameter of stirrup = 12 mm

Concrete cover = 40 mm

D = 500 – 40 – 12/2

= 454 mm

The shear strength of concrete is neglected Vc = 0 (because Ve > Vu)

$$V_s = \frac{V_u}{\emptyset} - 0 = \frac{13.0874 \times 1000}{0,75} = 17449.87387 \text{ N}$$

$$\frac{A_v}{s} = \frac{V_s}{f_{yt} d} = \frac{17449.87387}{280 \times 453,5} = 0.137270877 \text{ mm}^2/\text{mm} \quad \text{(A)}$$

Calculations for restraint reinforcement by:

For Pu = -128850,5 N < 0.3f'c

Ag = 0.3 x 25 x 500 x 500 = 1875000 N

fc' = 25 Mpa < 70 Mpa, use the equations:

$$\frac{A_{sh}}{Sb_c} = 0,3 \left(\frac{A_g}{A_{ch}} - 1\right) \frac{f'_c}{f_{yt}}$$

$$\frac{A_{sh}}{Sb_c} = 0,09 \frac{f'_c}{f_{yt}}$$

Bc = column width – concrete cover

= 500 – 2 x 40

= 420 mm

Ag = 500 x 500

= 250000 mm2

Ach = (b – 2 cover) x (h – 2 cover)

= (500 – 2 x 40) x (500 – 2 x 40)

= 176400 mm2

$$\frac{A_{sh}}{Sb_c} = 0,3 \left(\frac{A_g}{A_{ch}} - 1\right) \frac{f'_c}{f_{yt}}$$

$$= 0{,}3\left(\frac{250000}{176400}\text{-}1\right)\frac{25}{280} = 0{,}0111758$$

$\frac{A_{sh}}{s}$ $= 0{,}0111758 \times 420 = 4{,}69387\ mm^2/mm$................................... (B)

$$\frac{A_{sh}}{Sb_c} = 0{,}09\frac{f'_c}{f_{yt}}$$

$$= 0{,}09\frac{25}{280} = 0{,}0070714$$

$\frac{A_{sh}}{s}$ $= 0{,}0070714 \times 420 = 2{,}97\ mm^2/mm$... (C)

From (A), (B), and (C) which determine (B)

$\frac{A_{sh}}{s}$ $= 4{,}69387\ mm^2/mm$

Calculation of transverse reinforcement in the area along l0 by means of:

For example, taken S = 100 mm

Ash = 4.69387 x 100 = 469.39 mm2

12 mm diameter, 1ft wide

Av = ¼ x π x 12^2 = 113.10 mm2

Number of legs of transverse reinforce = 469.39 /113.10 =175,11 > use n = 5

Transverse reinforcement 5D12-100

S max:

a. ¼ smallest column dimension = ¼ × 500 = 125 mm
b. 6 times the diameter of the longitudinal reinforcement = 6 × 22= 132 mm
c. Hx = (500-(2 × 40) - (2 × 13) - (22))/3 = 124 mm
d. $S_0 = 100+\left(\frac{350\text{-}124}{3}\right)$=175,333 mm

Thus, a spacing of 100 mm transverse reinforcement meets the requirements

Calculation of transverse reinforcement outside the area l_0 by:

Ve = 13.0874 kN

$V_c = 0{,}17\sqrt{30}b_w d = 0{,}17 \times \sqrt{25} \times 500 \times 453{,}5 = 192{,}7375\ kN$

Vc = 192,7375kN > Ve = 13.0874 kN

The distance of the transverse reinforcement is taken = 100 mm

Then use 5D12-100.

2.9.9 2^{nd} Floor Main Beam 4

It is known that the column is 500×500 with a height of 4 m, the results obtained from the etabs are:

Pu max = 31.5843 kN

Mx = 42.8979kNm

My = 48.2265kNm

Pu min = -207.9834kN

Mx = -51.6438 kNm

My = -41.1375 kNm

Vu = -23.7151 kN

Has a beam height of 0.4 m. F'c= 25 MPa ; main reinforcement fy = 420 MPa; fy Sengkang = 280 MPa. The column has a main reinforcement diameter of 22 mm with a total of 12 reinforcement (set in SpColoumn). It is also known that the diameter of the stirrup is 12 mm. Planned reinforcement 12D22.

The "ØMn" column output due to Pu max and Pu min from the SPColumn software can be seen in Figure 2.36

No	Pu	Mux	Muy	φMnx	φMny	φMn/Mu	NA Depth	dt Depth	εt	φ
	kN	kNm	kNm	kNm	kNm		mm	mm		
1	-31	-48	-42	-272,24	-238,21	5.672	225	561	0,0058	0,9
2	207	41	51	244,27	303,84	5.958	242	657	0,00514	0,9

	ϕM_{nx}	ϕM_{ny}	ϕ	ϕM_{nx}	ϕM_{ny}
	kNm	kNm		kNm	kNm
Pu max	-272,24	-238,21	0,9	-302,4888889	-264,6777778
Pu min	244,27	303,84	0,9000	271,4111111	337,6

Figure 2. 36 2nd Floor Main Beam 4 SPColumn Output

Mnc a = 302.4888 kNm

Mnc b = 271.4111 kNm

MPR BI 1

Mprb, ki (-) = 24.2117 kNm

Mprb, ka (+) = 24.2117 kNm

(Mnc a + Mnc b) ≥ 1,2(Mprb ki + Mprb ka)

(302.4888 +271.4111) ≥ 1,2(24,2117+ 24,2117)

573.9 kNm ≥ 58.1080 kNm

Because Mnc > 1.2 Mprb, the column meets the requirements (*Strong Column Weak Beam*)

The transverse reinforcement is based on column shear strength and concrete core confinement as follows:

a. From structural analysis Vu = -23.7151 kNm

b. Based on Mpr beam left and right column:

Mprb, ki (-)	= 24.2117 kNm
Mprb, ki (+)	= 24.2117 kNm
Mprb, ka (-)	= 24.2117 kNm
Mprb, ka (+)	= 24.2117 kNm
Mprk of block	= 0.5x(24.2117 + 24.2117)
Mprk	= 24.2117 kNm

Calculating the necessary shear strength in the following way:

$V_e = \frac{24.2117 + 24.2117}{(4-0,3)}$ = 12.9129 kN

Value of Ve = 12.9129 kN

12.9129 kN > Vu from structural analysis = -23.7151 kN

Then use Vu	= Ve = 12.9129 kN
Diameter of stirrup	= 12 mm
Concrete cover	= 40 mm
D	= 500 – 40 – 12/2
	= 454 mm

The shear strength of concrete is neglected Vc = 0 (because Ve > Vu)

$V_s = \frac{V_u}{\emptyset} - 0 = \frac{12.9129 \times 1000}{0,75}$ = 17217.2088 N

$\frac{A_v}{s} = \frac{V_s}{f_{yt}d} = \frac{17217.2088}{280 \times 453,5}$ = 0.135440599 mm^2/mm .... (A)

Calculations for restraint reinforcement by:

For Pu = 31584.3 N < 0.3f'c

Ag = 0.3 x 25 x 500 x 500 = 1875000 N

fc' = 25 Mpa < 70 Mpa, use the equations:

$$\frac{A_{sh}}{Sb_c} = 0,3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$$

$$\frac{A_{sh}}{Sb_c} = 0,09\frac{f'_c}{f_{yt}}$$

Bc = column width – concrete cover

= 500 – 2 x 40

= 420 mm

Ag = 500 x 500

= 250000 mm2

Ach = (b – 2 cover) x (h – 2 cover)

= (500 – 2 x 40) x (500 – 2 x 40) = 176400 mm2

$$\frac{A_{sh}}{Sb_c} = 0,3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$$

$$= 0,3\left(\frac{250000}{176400} - 1\right)\frac{25}{280} = 0,0111758$$

$\frac{A_{sh}}{S}$ = 0,0111758×420 = 4,69387 mm^2/mm.................................. (B)

$$\frac{A_{sh}}{Sb_c} = 0.09\frac{f'_c}{f_{yt}}$$

$$= 0,09\frac{25}{280} = 0.0070714$$

$\frac{A_{sh}}{S}$ = 0,0070714×420 =2.97 mm^2/mm.. (C)

From (A), (B), and (C) which determine (B)

$\frac{A_{sh}}{S}$ = 4,69387 mm^2/mm

Calculation of transverse reinforcement in the area along l0 by means of:

For example, taken S = 100 mm

Ash = 4.69387 x 100 = 469.39 mm2

12 mm diameter, 1ft wide

Av = ¼ x π x 12^2 = 113.10 mm2

Number of legs of transverse reinforce = 469.39 /113.10 =175.11 > use n = 5

Transverse reinforcement 5D12-100

S max:

a. ¼ smallest column dimension = ¼ × 500 = 125 mm
b. 6 times the diameter of the longitudinal reinforcement = 6 × 22= 132 mm
c. Hx = (500-(2 × 40) - (2 × 13) - (22))/3 = 124 mm
d. $S_0 = 100+\left(\frac{350\text{-}124}{3}\right)$=175.333 mm

Thus, a spacing of 100 mm transverse reinforcement meets the requirements.

Calculation of transverse reinforcement outside the area l_0 by:

Ve = 12.9129 kN

V_c= 0,17$\sqrt{30}b_w d$ = 0,17×$\sqrt{25}$×500×453,5 = 192,7375 kN

Vc = 192,7375kN > Ve = 12.9129 kN

The distance of the transverse reinforcement is taken = 100 mm

Then use 5D12-100.

2.9.10 2nd Floor Main Beam 5

It is known that the column is 500×500 with a height of 4 m, the results obtained from the etabs are:

Pu max = -109.5854 kN

Mx = 60.8556 kNm

My = 55.5325 kNm

Pu min = -343.0406 kN

Mx = -99.573 kNm

My = -36.7215 kNm

Vu = -44.2221 kN

Has a beam height of 0.4 m. F'c= 25 MPa ; main reinforcement fy = 420 MPa; fy Sengkang = 280 MPa. The column has a main reinforcement diameter of 22 mm with a total of 12 reinforcement (set in SpColoumn). It is also known that the diameter of the stirrup is 12 mm. Planned reinforcement 12D22.

The "ØMn" column output due to Pu max and Pu min from the SPColumn software can be seen in Figure 2.37

No	Pu	Mux	Muy	φMnx	φMny	φMn/Mu	NA Depth	dt Depth	εt	φ
	kN	kNm	kNm	kNm	kNm		mm	mm		
1	109	-55	-40	-219,08	-239	3,983	225	562	0,00583	0,9
2	343	36	99	122,64	337,25	3,407	154	594	0,00623	0,9

	ϕM_{nx}	ϕM_{ny}	ϕ	ϕM_{ex}	ϕM_{ey}
	kNm	kNm		kNm	kNm
Pu max	-219,08	-239	0,9	-243,1222222	-265,5555556
Pu min	122,64	337,26	0,9000	136,2666667	374,7333333

Figure 2. 37 2nd Floor Main Beam 5 SPColumn Output

Mnc a = 243.4222 kNm

Mnc b = 136.2666 kNm

MPR BI 1

Mprb, ki (-) = 24.2117 kNm

Mprb, ka (+) = 24.2117 kNm

(Mnc a + Mnc b) ≥ 1,2(Mprb ki + Mprb ka)

(243.4222 + 136.2666) ≥ 1,2(24.2117 + 24.2117)

379.6888 kNm ≥ 58.1080 kNm

Because Mnc > 1.2 Mprb, the column meets the requirements (*Strong Column Weak Beam*)

The transverse reinforcement is based on column shear strength and concrete core confinement as follows:

a. From structural analysis Vu = -44.2221 kNm

b. Based on Mpr beam left and right column:

Mprb, ki (-) = 24.2117 kNm

Mprb, ki (+) = 24.2117 kNm

Mprb, ka (-) = 24.2117 kNm

Mprb, ka (+) = 24.2117 kNm

Mprk of block = 0.5x(24.2117 + 24.2117)

Mprk = 24.2117 kNm

Calculating the necessary shear strength in the following way:

$V_e = \frac{24.2117 + 24.2117}{(4-0.2)}$ = 12.743 kN

Value of Ve = 12.743 kN

12.743 kN > Vu from structural analysis = -44.2221 kN

Then use Vu = Ve = 12.743 kN

Diameter of stirrup = 12 mm

Concrete cover = 40 mm

D = 500 – 40 – 12/2

= 454 mm

The shear strength of concrete is neglected Vc = 0 (because Ve > Vu)

$V_s = \frac{V_u}{\emptyset} - 0 = \frac{12.743 \times 1000}{0,75}$ = 16990.66667 N

$\frac{A_v}{s} = \frac{V_s}{f_{yt}d} = \frac{16990.66667}{280 \times 454}$ = 0.133658485 mm^2/mm................................. (A)

Calculations for restraint reinforcement by:

For Pu = -109585.4 N < 0.3f'c

Ag = 0.3 x 25 x 500 x 500 = 1875000 N

fc' = 25 Mpa < 70 Mpa, use the equations:

$$\frac{A_{sh}}{Sb_c} = 0.3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$$

$$\frac{A_{sh}}{Sb_c} = 0.09\frac{f'_c}{f_{yt}}$$

Bc = column width – concrete cover

= 500 – 2 x 40

= 420 mm

Ag = 500 x 500

= 250000 mm2

Ach = (b – 2 cover) x (h – 2 cover)

= (500 – 2 x 40) x (500 – 2 x 40) = 176400 mm2

$$\frac{A_{sh}}{Sb_c} = 0.3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$$

$$= 0.3\left(\frac{250000}{176400}-1\right)\frac{25}{280} = 0.0111758$$

$$\frac{A_{sh}}{S} = 0.0111758 \times 420 = 4.69387\ mm^2/mm \text{.................................... (B)}$$

$$\frac{A_{sh}}{Sb_c} = 0.09\frac{f'_c}{f_{yt}}$$

$$= 0.09\frac{25}{280} = 0.0070714$$

$$\frac{A_{sh}}{S} = 0.0070714 \times 420 = 2.97\ mm^2/mm \text{.. (C)}$$

From (A), (B), and (C) which determine (B)

$$\frac{A_{sh}}{S} = 4.69387\ mm^2/mm$$

Calculation of transverse reinforcement in the area along l0 by means of:

For example, taken S = 100 mm

Ash = 4.69387 x 100 = 469.39 mm2

12 mm diameter, 1ft wide

$Av = ¼ \times \pi \times 12^2 = 113.10$ mm2

Number of legs of transverse reinforce = 469.39 /113.10 =175,11 > use n = 5

Transverse reinforcement 5D12-100

S max:

a. ¼ smallest column dimension = ¼ × 500 = 125 mm
b. 6 times the diameter of the longitudinal reinforcement = 6 × 22= 132 mm
c. Hx = (500-(2 × 40) - (2 × 13) - (22))/3 = 124 mm
d. $S_0 = 100+\left(\frac{350\text{-}124}{3}\right)$=175,333 mm

Thus, a spacing of 100 mm transverse reinforcement meets the requirements

Calculation of transverse reinforcement outside the area l_0 by:

Ve = 12.743 kN

V_c=0,17$\sqrt{30}b_w d$ =0,17×$\sqrt{25}$×500×453,5 = 192,7375 kN

Vc = 192,7375kN > Ve = 12.743 kN

The distance of the transverse reinforcement is taken = 100 mm

Then use 5D12-100.

2.9.11 3rd Floor Main Beam 1

It is known that the column is 500×500 with a height of 4 m, the results obtained from the etabs are:

Pu max	= -28.0854 kN
Mx	= 36.9177 kNm
My	= 38.6086 kNm
Pu min	= -171.351 kN
Mx	= -32.4736 kNm
My	= -58.5033 kNm
Vu	= -31.3752 kN

Has a beam height of 0.4 m. F'c= 25 MPa ; main reinforcement fy = 420 MPa; fy Sengkang = 280 MPa. The column has a main reinforcement diameter of 22 mm with a total of 12 reinforcement (set in SpColoumn). It is also known that the diameter of the stirrup is 12 mm. Planned reinforcement 12D22.

The "ØMn" column output due to Pu max and Pu min from the SPColumn software can be seen in Figure 2.38

No	Pu	Mux	Muy	φMnx	φMny	φMn/Mu	NA Depth	dt Depth	εt	φ
	kN	kNm	kNm	kNm	kNm		mm	mm		
1	28	-38	-36	-267,15	-253,09	7.030	230	563	0,00552	0,9
2	171	58	32	341,24	188,15	5.880	220	530	0,00561	0,9

	ϕM_{nx}	ϕM_{ny}	ϕ	ϕM_{ex}	ϕM_{ey}
	kNm	kNm		kNm	kNm
Pu max	-267,15	-253,09	0,9	-296,8333333	-281,2111111
Pu min	341,04	188,16	0,9000	378,9333333	209,0666667

Figure 2. 38 3rd Floor Main Beam 1 SPColumn Output

Mnc a = 296.8333 kNm

Mnc b = 378.9333 kNm

MPR BI 1

Mprb, ki (-) = 32.4265 kNm

Mprb, ka (+) = 32.4265 kNm

(Mnc a + Mnc b) ≥ 1,2(Mprb ki + Mprb ka)

(296.8333 + 378.9333) ≥ 1,2(32.4265 + 32.4265)

675.7667 kNm ≥ 77.8236 kNm

Because Mnc > 1.2 Mprb, the column meets the requirements (*Strong Column Weak Beam*)

The transverse reinforcement is based on column shear strength and concrete core confinement as follows:

a. From structural analysis Vu = -31.3752 kNm

b. Based on Mpr beam left and right column:

Mprb, ki (-) = 32.4265 kNm

Mprb, ki (+) = 32.4265 kNm

Mprb, ka (-) = 32.4265 kNm

Mprb, ka (+) = 32.4265 kNm

Mprk of block = 0.5x(32.4265 + 32.4265)

Mprk = 32.4265 kNm

Calculating the necessary shear strength in the following way:

$V_e = \frac{32.4265 \ + 32.4265}{(4\text{-}0.4)}$ = 18.0147 kN

Value of Ve = 18.0147 kN

18.0147 kN > Vu from structural analysis = -31.3752 kN

Then use Vu = Ve = 18.0147 kN

Diameter of stirrup = 12 mm

Concrete cover = 40 mm

D = 500 – 40 – 12/2

= 454 mm

The shear strength of concrete is neglected Vc = 0 (because Ve > Vu)

$V_s = \frac{V_u}{\emptyset} - 0 = \frac{18.0147 \times 1000}{0{,}75} = 24019.62963$ N

$\frac{A_v}{s} = \frac{V_s}{f_{yt} d} = \frac{24019.62963}{280 \times 453{,}5} = 0.188952404\ mm^2/mm$ .... (A)

Calculations for restraint reinforcement by:

For Pu = -28085.4 N < 0.3f'c

Ag = 0.3 x 25 x 500 x 500

= 1875000 N

fc' = 25 Mpa < 70 Mpa, use the equations:

$$\frac{A_{sh}}{Sb_c} = 0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$$

$$\frac{A_{sh}}{Sb_c} = 0.09\frac{f'_c}{f_{yt}}$$

Bc = column width – concrete cover

= 500 – 2 x 40

= 420 mm

Ag = 500 x 500

= 250000 mm2

Ach = (b – 2 cover) x (h – 2 cover)

= (500 – 2 x 40) x (500 – 2 x 40)

= 176400 mm2

$$\frac{A_{sh}}{Sb_c} = 0.3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$$

$$= 0.3\left(\frac{250000}{176400}-1\right)\frac{25}{280} = 0.0111758$$

$$\frac{A_{sh}}{s} = 0.0111758 \times 420 = 4.69387\ mm^2/mm \text{ ....} (B)$$

$$\frac{A_{sh}}{Sb_c} = 0.09\frac{f'_c}{f_{yt}}$$

$$= 0.09\frac{25}{280} = 0.0070714$$

$$\frac{A_{sh}}{s} = 0.0070714 \times 420 = 2.97\ mm^2/mm \text{ ....} (C)$$

From (A), (B), and (C) which determine (B)

$\frac{A_{sh}}{s} = 4{,}69387\ mm^2/mm$

Calculation of transverse reinforcement in the area along l0 by means of:

For example, taken S = 100 mm

Ash = 4.69387 x 100 = 469.39 mm2

12 mm diameter, 1ft wide

Av = ¼ x π x 12^2 = 113.10 mm2

Number of legs of transverse reinforce = 469.39 /113.10 =175,11 > use n = 5

Transverse reinforcement 5D12-100

S max:

a. ¼ smallest column dimension = ¼ × 500 = 125 mm
b. 6 times the diameter of the longitudinal reinforcement = 6 × 22= 132 mm
c. Hx = (500-(2 × 40) - (2 × 13) - (22))/3 = 124 mm

d. $S_0 = 100 + \left(\frac{350\text{-}124}{3}\right) = 175{,}333$ mm

Thus, a spacing of 100 mm transverse reinforcement meets the requirements

Calculation of transverse reinforcement outside the area l_0 by:

Ve = 18.0147 kN

$V_c = 0{,}17\sqrt{30}b_w d = 0{,}17 \times \sqrt{25} \times 500 \times 453{,}5 = 192{,}7375\ kN$

Vc = 192,7375kN > Ve = 18.0147 kN

The distance of the transverse reinforcement is taken = 100 mm

Then use 5D12-100.

2.9.12 3rd Floor Main Beam 2

It is known that the column is 500×500 with a height of 4 m, the results obtained from the etabs are:

Pu max = -30.2479 kN

Mx = 42.0299 kNm

My = 35.5657 kNm

Pu min = -167.6629 kN

Mx = -45.8179 kNm

My = -54.8356 kNm

Vu = -20.0759 kN

Has a beam height of 0.4 m. F'c= 25 MPa ; main reinforcement fy = 420 MPa; fy Sengkang = 280 MPa. The column has a main reinforcement diameter of 22 mm with a total of 12 reinforcement (set in SpColoumn). It is also known that the diameter of the stirrup is 12 mm. Planned reinforcement 12D22.

The "ØMn" column output due to Pu max and Pu min from the SPColumn software can be seen in Figure 2.39

No	Pu	Mux	Muy	φMnx	φMny	φMn/Mu	NA Depth	dt Depth	εt	φ
	kN	kNm	kNm	kNm	kNm		mm	mm		
1	30	-35	-42	-236,51	-283,81	6.757	230	[illegible]	0,00562	0,9
2	167	54	45	296,01	246,68	5.482	241	[illegible]	0,00522	0,9

	ϕM_{nx}	ϕM_{ny}	ϕ	ϕM_{nx}	ϕM_{ny}
	kNm	kNm		kNm	kNm
Pu max	-236,51	-283,81	0,9	-262,7888889	-315,3444444
Pu min	296,01	246,68	0,9000	328,9	274,0888889

Figure 2. 39 3rd Floor Main Beam 2 SPColumn Output

Mnc a = 262.7888 kNm

Mnc b = 328.9 kNm

MPR BI 1

Mprb, ki (-) = 32.4265 kNm

Mprb, ka (+) = 32.4265 kNm

(Mnc a + Mnc b) ≥ 1,2(Mprb ki + Mprb ka)

(262.7888 + 328.9) ≥ 1,2(32.4265 + 32.4265)

591.6888 kNm ≥ 77.8236 kNm

Because Mnc > 1.2 Mprb, the column meets the requirements (*Strong Column Weak Beam*)

The transverse reinforcement is based on column shear strength and concrete core confinement as follows:

a. From structural analysis Vu = -20.0759 kNm

b. Based on Mpr beam left and right column:

Mprb, ki (-) = 32.4265 kNm

Mprb, ki (+) = 32.4265 kNm

Mprb, ka (-) = 32.4265 kNm

Mprb, ka (+) = 32.4265 kNm

Mprk of block = 0.5x(32.4265 + 32.4265)

Mprk = 32.4265 kNm

Calculating the necessary shear strength in the following way:

$V_e = \frac{32.4265 + 32.4265}{(4-0.35)}$ = 17.7679 kN

Value of Ve = 17.7679 kN

17.7679 kN > Vu from structural analysis = -20.0759 kN

Then use Vu = Ve = 17.7679 kN

Diameter of stirrup = 12 mm

Concrete cover = 40 mm

D = 500 – 40 – 12/2

= 454 mm

The shear strength of concrete is neglected Vc = 0 (because Ve > Vu)

$$V_s = \frac{V_u}{\phi} - 0 = \frac{17.7679 \times 1000}{0,75} = 23690.59361 \text{ N}$$

$$\frac{A_v}{s} = \frac{V_s}{f_{yt} d} = \frac{23690.59361}{280 \times 453,5} = 0.186364015 \text{ mm}^2/\text{mm} \ldots\ldots (A)$$

Calculations for restraint reinforcement by:

For Pu = -30247.9 N < 0.3f'c

Ag = 0.3 x 25 x 500 x 500

= 1875000 N

fc' = 25 Mpa < 70 Mpa, use the equations:

$$\frac{A_{sh}}{Sb_c} = 0,3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$$

$$\frac{A_{sh}}{Sb_c} = 0,09\frac{f'_c}{f_{yt}}$$

Bc = column width – concrete cover

= 500 – 2 x 40

= 420 mm

Ag = 500 x 500

= 250000 mm2

Ach = (b – 2 cover) x (h – 2 cover)

= (500 – 2 x 40) x (500 – 2 x 40)

= 176400 mm2

$$\frac{A_{sh}}{Sb_c} = 0{,}3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$$

$$= 0{,}3\left(\frac{250000}{176400}-1\right)\frac{25}{280} = 0{,}0111758$$

$$\frac{A_{sh}}{S} = 0{,}0111758 \times 420 = 4{,}69387\ mm^2/mm \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (B)$$

$$\frac{A_{sh}}{Sb_c} = 0{,}09\frac{f'_c}{f_{yt}}$$

$$= 0{,}09\frac{25}{280} = 0{,}0070714$$

$$\frac{A_{sh}}{S} = 0{,}0070714 \times 420 = 2{,}97\ mm^2/mm \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (C)$$

From (A), (B), and (C) which determine (B)

$$\frac{A_{sh}}{S} = 4{,}69387\ mm^2/mm$$

Calculation of transverse reinforcement in the area along l0 by means of:

For example, taken S = 100 mm

Ash = 4.69387 x 100 = 469.39 mm2

12 mm diameter, 1ft wide

Av = ¼ x π x 12^2 = 113.10 mm2

Number of legs of transverse reinforce = 469.39 /113.10 =175,11 > use n = 5

Transverse reinforcement 5D12-100

S max:

a. ¼ smallest column dimension = ¼ × 500 = 125 mm
b. 6 times the diameter of the longitudinal reinforcement = 6 × 22= 132 mm
c. Hx = (500-(2 × 40) - (2 × 13) - (22))/3 = 124 mm
d. $S_0 = 100 + \left(\frac{350-124}{3}\right) = 175{,}333$ mm

Thus, a spacing of 100 mm transverse reinforcement meets the requirements.

Calculation of transverse reinforcement outside the area l_0 by:

Ve = 17.7679 kN

$V_c = 0{,}17\sqrt{30}b_w d = 0{,}17 \times \sqrt{25} \times 500 \times 453{,}5 = 192{,}7375\ kN$

Vc = 192,7375kN > Ve = 17.7679 kN

The distance of the transverse reinforcement is taken = 100 mm

Then use 5D12-100.

2.9.13 3rd Floor Main Beam 3

It is known that the column is 500×500 with a height of 4 m, the results obtained from the etabs are:

Pu max = -33.7122 kN

Mx = 54.2299 kNm

My = 67.5293 kNm

Pu min = -144.2445 kN

Mx = -32.7982 kNm

My = -96.8783 kNm

Vu = -23.4653 kN

Has a beam height of 0.4 m. F'c= 25 MPa ; main reinforcement fy = 420 MPa; fy Sengkang = 280 MPa. The column has a main reinforcement diameter of 22 mm with a total of 12 reinforcement (set in SpColoumn). It is also known that the diameter of the stirrup is 12 mm. Planned reinforcement 12D22.

The "ØMn" column output due to Pu max and Pu min from the SPColumn software can be seen in Figure 2.40

No	Pu	Mux	Muy	φMnx	φMny	φMn/Mu	NA Depth	dt Depth	εt	φ
	kN	kNm	kNm	kNm	kNm		mm	mm		
1	33	-67	54	-287,71	-231,88	4.294	228	658	0,00565	0,9
2	144	95	32	370,53	123,51	1.880	181	582	0,00675	0,9

	ϕM_{nx}	ϕM_{ny}	ϕ	ϕM_{nx}	ϕM_{ny}
	kNm	kNm		kNm	kNm
Pu max	-287,71	-231,88	0,9	-319,6777778	-257,6444444
Pu min	370,53	123,51	0,9000	411,7	137,2333333

Figure 2. 40 3rd Floor Main Beam 3 SPColumn Output

Mnc a = 319.6777 kNm

Mnc b = 411.7 kNm

MPR BI 1

Mprb, ki (-) = 24.2118 kNm

Mprb, ka (+) = 24.2118 kNm

(Mnc a + Mnc b) ≥ 1,2(Mprb ki + Mprb ka)

(319.6777 + 411.7) ≥ 1,2(24.2118 + 24.2118)

731.3777 kNm ≥ 58.1083 kNm

Because Mnc > 1.2 Mprb, the column meets the requirements (*Strong Column Weak Beam*)

The transverse reinforcement is based on column shear strength and concrete core confinement as follows:

a. From structural analysis Vu = -23.4653 kNm

b. Based on Mpr beam left and right column:

Mprb, ki (-) = 24.2118 kNm

Mprb, ki (+) = 24.2118 kNm

Mprb, ka (-) = 24.2118 kNm

Mprb, ka (+) = 24.2118 kNm

Mprk of block = 0.5x(24.2118 + 24.2118)

Mprk = 24.2118 kNm

Calculating the necessary shear strength in the following way:

$V_e = \frac{24.2118 + 24.2118}{(4-0.3)}$ = 13.0874 kN

Value of Ve = 13.0874 kN

13.0874 kN > Vu from structural analysis = -23.4653 kN

Then use Vu = Ve = 13.0874 kN

Diameter of stirrup = 12 mm

Concrete cover = 40 mm

D = 500 – 40 – 12/2

= 454 mm

The shear strength of concrete is neglected Vc = 0 (because Ve > Vu)

$$V_s = \frac{V_u}{\emptyset} - 0 = \frac{13.0874 \times 1000}{0,75} \quad = 17449.94595 \text{ N}$$

$$\frac{A_v}{s} = \frac{V_s}{f_{yt}d} = \frac{17449.94595}{280 \times 453,5} \quad = 0.137271444 \text{ mm}^2/\text{mm} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(A)}$$

Calculations for restraint reinforcement by:

For Pu = -33712.2 N < 0.3f'c

Ag = 0.3 x 25 x 500 x 500

= 1875000 N

fc' = 25 Mpa < 70 Mpa, use the equations:

$$\frac{A_{sh}}{Sb_c} = 0,3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$$

$$\frac{A_{sh}}{Sb_c} = 0,09\frac{f'_c}{f_{yt}}$$

Bc = column width – concrete cover

= 500 – 2 x 40

= 420 mm

Ag = 500 x 500

= 250000 mm2

Ach = (b – 2 cover) x (h – 2 cover)

= (500 – 2 x 40) x (500 – 2 x 40)

= 176400 mm2

$$\frac{A_{sh}}{Sb_c} = 0,3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$$

$$= 0,3\left(\frac{250000}{176400} - 1\right)\frac{25}{280} = 0,0111758$$

$$\frac{A_{sh}}{S} = 0,0111758 \times 420 = 4,69387 \text{ mm}^2/\text{mm} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(B)}$$

$$\frac{A_{sh}}{Sb_c} = 0{,}09\frac{f'_c}{f_{yt}}$$

$$= 0{,}09\frac{25}{280} = 0{,}0070714$$

$$\frac{A_{sh}}{S} = 0{,}0070714 \times 420 = 2{,}97\ mm^2/mm \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (C)$$

From (A), (B), and (C) which determine (B)

$$\frac{A_{sh}}{S} = 4{,}69387\ mm^2/mm$$

Calculation of transverse reinforcement in the area along l0 by means of:

For example, taken S = 100 mm

Ash = 4.69387 x 100 = 469.39 mm2

12 mm diameter, 1ft wide

Av = ¼ x π x 12^2 = 113.10 mm2

Number of legs of transverse reinforce = 469.39 /113.10 =175,11 > use n = 5

Transverse reinforcement 5D12-100.

S max:

a. ¼ smallest column dimension = ¼ × 500 = 125 mm
b. 6 times the diameter of the longitudinal reinforcement = 6 × 22= 132 mm
c. Hx = (500-(2 × 40) - (2 × 13) - (22))/3 = 124 mm
d. $S_0 = 100 + \left(\frac{350\text{-}124}{3}\right) = 175{,}333$ mm

Thus, a spacing of 100 mm transverse reinforcement meets the requirements.

Calculation of transverse reinforcement outside the area l_0 by:

Ve = 13.0874 kN

$V_c = 0{,}17\sqrt{30}b_wd = 0{,}17 \times \sqrt{25} \times 500 \times 453{,}5 = 192{,}7375\ kN$

Vc = 192,7375kN > Ve = 13.0874 kN

The distance of the transverse reinforcement is taken = 100 mm

Then use 5D12-100.

2.9.14 3rd Floor Main Beam 4

It is known that the column is 500×500 with a height of 4 m, the results obtained from the etabs are:

Pu max = 17.6532 kN

Mx = 41.2942 kNm

My = 29.3837 kNm

Pu min = -94.8577 kN

Mx = -28.5541 kNm

My = -39.8497 kNm

Vu = -17.4495 kN

Has a beam height of 0.4 m. F'c= 25 MPa ; main reinforcement fy = 420 MPa; fy Sengkang = 280 MPa. The column has a main reinforcement diameter of 22 mm with a total of 12 reinforcement (set in SpColoumn). It is also known that the diameter of the stirrup is 12 mm. Planned reinforcement 12D22.

The "ØMn" column output due to Pu max and Pu min from the SPColumn software can be seen in Figure 2.41

No	Pu	Mux	Muy	ϕM_{nx}	ϕM_{ny}	$\phi M_n/M_u$	NA Depth	dt Depth	ε_t	ϕ
	kN	kNm	kNm	kNm	kNm		mm	mm		
1	-17	-29	-41	-209,58	-296,31	7227	237	[illegible]	[illegible]	0,9
2	94	[illegible]	28	306,3	219,9	[illegible]	228	[illegible]	[illegible]	0,9

	ϕM_{nx}	ϕM_{ny}	ϕ	ϕM_{nx}	ϕM_{ny}
	kNm	kNm		kNm	kNm
Pu max	-209,58	-296,31	0,9	-232,8666667	-329,2333333
Pu min	306,3	219,9	0,9000	340,3333333	244,3333333

Figure 2. 41 3rd Floor Main Beam 4 SPColumn Output

Mnc a = 232.8667 kNm

Mnc b = 340.3333 kNm

MPR BI 1

Mprb, ki (-) = 24.2118 kNm

Mprb, ka (+) = 24.2118 kNm

(Mnc a + Mnc b) ≥ 1,2(Mprb ki + Mprb ka)

(232.8667 + 340.3333) ≥ 1,2(24.2118 + 24.2118)

573.2 kNm ≥ 58.1083 kNm

Because Mnc > 1.2 Mprb, the column meets the requirements (*Strong Column Weak Beam*)

The transverse reinforcement is based on column shear strength and concrete core confinement as follows:

a. From structural analysis Vu = -17,4495 kNm

b. Based on Mpr beam left and right column:

Mprb, ki (-) = 24.2118 kNm

Mprb, ki (+) = 24.2118 kNm

Mprb, ka (-) = 24.2118 kNm

Mprb, ka (+) = 24.2118 kNm

Mprk of block = 0.5x(24.2118 + 24.2118)

Mprk = 24.2118 kNm

Calculating the necessary shear strength in the following way:

$V_e = \frac{24.2118 + 24.2118}{(4-0.25)}$ = 12.9129 kN

Value of Ve = 12.9129 kN

12.91296 kN > Vu from structural analysis = -17,4495 kN

Then use Vu = Ve = 12.9129 kN

Diameter of stirrup = 12 mm

Concrete cover = 40 mm

D = 500 – 40 – 12/2

= 454 mm

The shear strength of concrete is neglected Vc = 0 (because Ve > Vu)

$V_s = \frac{V_u}{\emptyset} - 0 = \frac{12.91296 \times 1000}{0{,}75}$ = 17217.28 N

$\frac{A_v}{s} = \frac{V_s}{f_{yt} d} = \frac{17217.28}{280 \times 453{,}5}$ = 0.135441158 mm^2/mm .... (A)

Calculations for restraint reinforcement by:

For Pu = 17653.2 N < 0.3f'c

Ag = 0.3 x 25 x 500 x 500

= 1875000 N

fc' = 25 Mpa < 70 Mpa, use the equations:

$$\frac{A_{sh}}{Sb_c} = 0{,}3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$$

$$\frac{A_{sh}}{Sb_c} = 0{,}09\frac{f'_c}{f_{yt}}$$

Bc = column width – concrete cover

= 500 – 2 x 40

= 420 mm

Ag = 500 x 500

= 250000 mm2

Ach = (b – 2 cover) x (h – 2 cover)

= (500 – 2 x 40) x (500 – 2 x 40)

= 176400 mm2

$$\frac{A_{sh}}{Sb_c} = 0{,}3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$$

$$= 0{,}3\left(\frac{250000}{176400}-1\right)\frac{25}{280} = 0{,}0111758$$

$$\frac{A_{sh}}{S} = 0{,}0111758 \times 420 = 4{,}69387\ mm^2/mm \ldots\ldots\ldots\ldots\ldots\ldots (B)$$

$$\frac{A_{sh}}{Sb_c} = 0{,}09\frac{f'_c}{f_{yt}}$$

$$= 0{,}09\frac{25}{280} = 0{,}0070714$$

$$\frac{A_{sh}}{S} = 0{,}0070714 \times 420 = 2{,}97\ mm^2/mm \ldots\ldots\ldots\ldots\ldots\ldots (C)$$

From (A), (B), and (C) which determine (B)

$\frac{A_{sh}}{S}$ = 4,69387 mm^2/mm

Calculation of transverse reinforcement in the area along l0 by means of:

For example, taken S = 100 mm

Ash = 4.69387 x 100 = 469.39 mm2

12 mm diameter, 1ft wide

Av = ¼ x π x 12^2 = 113.10 mm2

Number of legs of transverse reinforce = 469.39 /113.10 =175,11 > use n = 5

Transverse reinforcement 5D12-100

S max:

a. ¼ smallest column dimension = ¼ × 500 = 125 mm
b. 6 times the diameter of the longitudinal reinforcement = 6 × 22= 132 mm
c. Hx = (500-(2 × 40) - (2 × 13) - (22))/3 = 124 mm
d. $S_0 = 100+\left(\frac{350\text{-}124}{3}\right)$=175,333 mm

Thus, a spacing of 100 mm transverse reinforcement meets the requirements.

Calculation of transverse reinforcement outside the area l_0 by:

Ve = 12.9129 kN

V_c=0,17$\sqrt{30}$$b_w$d = 0,17×$\sqrt{25}$×500×453,5 = 192,7375 kN

Vc = 192,7375kN > Ve = 12.9129 kN

The distance of the transverse reinforcement is taken = 100 mm

Then use 5D12-100.

2.9.15 3rd Floor Main Beam 5

It is known that the column is 500×500 with a height of 4 m, the results obtained from the etabs are:

Pu max = -47.1475 kN

Mx = 118.2426 kNm

My = 35.0146 kNm

Pu min = -170.3012 kN

Mx = -82.7535 kNm

My = -55.5872 kNm

Vu = -54.4619 kN

Has a beam height of 0.4 m. F'c= 25 MPa ; main reinforcement fy = 420 MPa; fy Sengkang = 280 MPa. The column has a main reinforcement diameter of 22 mm with a total of 12 reinforcement (set in SpColoumn). It is also known that the diameter of the stirrup is 12 mm. Planned reinforcement 12D22.

The "ØMn" column output due to Pu max and Pu min from the SPColumn software can be seen in Figure 2.42

No	Pu	Mux	Muy	ϕMnx	ϕMny	ϕMn/Mu	NA Depth	dt Depth	εt	ϕ
	kN	kNm	kNm	kNm	kNm		mm	mm		
1	47	-35	-118	-107,34	-301,89	3.067	163	368	0,00739	0,9
2	170	35	82	215,88	321,86	3.925	232	546	0,00537	0,9

	ϕM_{nx}	ϕM_{ny}	ϕ	ϕM_{nx}	ϕM_{ny}
	kNm	kNm		kNm	kNm
Pu max	-107,34	-301,89	0,9	-119,2666667	-402,1
Pu min	215,88	321,86	0,9000	239,8666667	357,6222222

Figure 2. 42 3rd Floor Main Beam 5 SPColumn Output

Mnc a = 119.2667 kNm

Mnc b =239.8667 kNm

MPR BI 1

Mprb, ki (-) = 24.2118 kNm

Mprb, ka (+) = 24.2118 kNm

(Mnc a + Mnc b) ≥ 1,2(Mprb ki + Mprb ka)

(119.2667 + 239.8667) ≥ 1,2(24.2118 + 24.2118)

359.1333 kNm ≥ 58.1083 kNm

Because Mnc > 1.2 Mprb, the column meets the requirements (*Strong Column Weak Beam*)

The transverse reinforcement is based on column shear strength and concrete core confinement as follows:

a. From structural analysis Vu = -54.4619 kNm

b. Based on Mpr beam left and right column:

Mprb, ki (-) = 24.2118 kNm

Mprb, ki (+) = 24.2118 kNm

Mprb, ka (-) = 24.2118 kNm

Mprb, ka (+) = 24.2118 kNm

Mprk of block = 0.5x(24.2118 + 24.2118)

Mprk = 24.2118 kNm

Calculating the necessary shear strength in the following way:

$V_e = \frac{24.2118 + 24.2118}{(4\text{-}0.2)}$ = 12.7430 kN

Value of Ve = 12.7430 kN

12.7430 kN > Vu from structural analysis = -54.4619 kN

Then use Vu = Ve = 12.7430 kN

Diameter of stirrup = 12 mm

Concrete cover = 40 mm

D = 500 – 40 – 12/2

= 454 mm

The shear strength of concrete is neglected Vc = 0 (because Ve > Vu)

$V_s = \frac{V_u}{\phi} - 0 = \frac{12.7430 \times 1000}{0{,}75}$ = 16990.73684 N

$\frac{A_v}{s} = \frac{V_s}{f_{yt} d} = \frac{16990.74}{280 \times 453.5}$ = 0.133659037 mm^2/mm .... (A)

Calculations for restraint reinforcement by:

For Pu = -47147.5 N < 0.3f'c

Ag = 0.3 x 25 x 500 x 500

= 1875000 N

fc' = 25 Mpa < 70 Mpa, use the equations:

$$\frac{A_{sh}}{Sb_c} = 0{,}3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$$

$$\frac{A_{sh}}{Sb_c} = 0{,}09\frac{f'_c}{f_{yt}}$$

Bc = column width – concrete cover

= 500 – 2 x 40

= 420 mm

Ag = 500 x 500

= 250000 mm2

Ach = (b – 2 cover) x (h – 2 cover)

= (500 – 2 x 40) x (500 – 2 x 40)

= 176400 mm2

$$\frac{A_{sh}}{Sb_c} = 0,3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$$

$$= 0,3\left(\frac{250000}{176400}-1\right)\frac{25}{280} = 0,0111758$$

$$\frac{A_{sh}}{S} = 0,0111758 \times 420 = 4,69387\ mm^2/mm \ldots\ldots\ldots\ldots\ldots\ldots (B)$$

$$\frac{A_{sh}}{Sb_c} = 0,09\frac{f'_c}{f_{yt}}$$

$$= 0,09\frac{25}{280} = 0,0070714$$

$$\frac{A_{sh}}{S} = 0,0070714 \times 420 = 2,97\ mm^2/mm \ldots\ldots\ldots\ldots\ldots\ldots (C)$$

From (A), (B), and (C) which determine (B)

$$\frac{A_{sh}}{S} = 4,69387\ mm^2/mm$$

Calculation of transverse reinforcement in the area along l0 by means of:

For example, taken S = 100 mm

Ash = 4.69387 x 100 = 469.39 mm2

12 mm diameter, 1ft wide

Av = ¼ x π x 12^2 = 113.10 mm2

Number of legs of transverse reinforce = 469.39 /113.10 =175,11 > use n = 5

Transverse reinforcement 5D12-100

S max:

a. ¼ smallest column dimension = ¼ × 500 = 125 mm

b. 6 times the diameter of the longitudinal reinforcement = 6 × 22= 132 mm

c. Hx = (500-(2 × 40) - (2 × 13) - (22))/3 = 124 mm

d. $S_0 = 100+\left(\frac{350\text{-}124}{3}\right)$=175,333 mm

Thus, a spacing of 100 mm transverse reinforcement meets the requirements.

Calculation of transverse reinforcement outside the area l_0 by:

Ve = 12.7430 kN

V_c=$0{,}17\sqrt{30}b_w d = 0{,}17\times\sqrt{25}\times500\times453{,}5 = 192{,}7375\ kN$

Vc = 192,7375kN > Ve = 12.7430 kN

The distance of the transverse reinforcement is taken = 100 mm

Then use 5D12-100.

2.10 Floor Slab Design

The slab is one part of the horizontal structure which can be affected by the length of the span and the load on the slab. Slabs need to be strengthened using steel reinforcement because the structure bears more bending moments and shear forces so that the plan of floor slabs must have the same height and not be slanted. The thickness of the floor slab can be determined from the load, allowable deflection, and span width.

2.10.1 One Way Slab

The one-way plate reinforcement system is only supported on both sides so that the plate experiences a deflection in a direction perpendicular to the support side. If the slab is supported on all four sides, almost 95% of the load will be distributed in the short span direction resulting in a one way slab. There are several conditions for the use of moment efficiency which are regulated in SNI 2847:2019 article 6.5.2, as follows :

a. The difference in span length is not too far, with the span length limit not exceeding 20% of the shortest span.

b. The loads that work are distributed.

c. Live load < 3x dead load.

Assalaffiyah Islamic Boarding School have 16 one way slabs type. 5 slabs for dormitory building and 11 slabs for educational building. The following is the result of calculating floor slabs with one-way reinforcement :

Structure Material Data

Plat Code = A

f'c = 25 Mpa

Floor Slab Data

fy	= 240 Mpa	Cly	= 11
Lx	= 1.95 m	Ctx	= 83
Ly	= 4.3 m	Cty	= 57
h	= 120 mm	ts	= 20 mm
Coef.Moment	$= \frac{Ly}{Lx} = \frac{4.3}{1.95} = 2.2$ m	Ø	= 10 mm

1. **Dead Load**

Own Weight of Floor Plate

Unit Weight = 24

Thickness = 0.12 m

Q = $24 \times 0.12 = 2.88$ kN/m^2

2. **Floor Finishing Weight**

Unit Weight = 21

Thickness = 0.05 m

Q = 21 x 0.05 = 1.05 kN/m^2

Ceiling and Frame Weight = 0.2

ME Installation Weight = 0.5

QD

$= \text{QDL} + \text{QFloor} + \text{ceiling and frame weight} + \text{QFloor}$

= 2.88 + 1.05 + 0.2 + 1.05

= 4.63 kN/m^2

3. **Live Load**

Live Load Building Floors = 480 kg/m^2

QL = 4.8 kN/m^2

Factored Plan Load

QU = (1.2 x QD) + (1.2 x QL)

= (1.2 x 4.63) + (1.6 x 4.8)

= 13.2 kN/m^2

4. Slab Moment Due to Factored Load

Mulx = Clx $\times$ 0.001 $\times$ QU x Lx^2

= 41 x 0.001 x 13.2 x 1.95^2

= 2.0635 kNm/m

Muly = Cly $\times$ 0.001 $\times$ QU $\times$ Ly^2

= 11 x 0.001 x 13.2 x 4.3^2

= 2.6921 kNm/m

Mutx = Ctx $\times$ 0.001 $\times$ QU $\times$ Lx^2

= 83 x 0.001 x 13.2 x 1.95^2

= 4.1774 kNm/m

Muty = Cty x 0.001 x QU x Ly^2

= 57 x 0.001 x 13.2 x 4.3^2

= 13.9498 kNm/m

Mu = 13,9498 kNm/m

Slab Reinforcement

fc' $\leq$ 30 MPa

β1 = 0.85

β1 = -

Form Factor of Concrete Stress Distribution

β1 = 0.85

$$\rho b = \frac{0.85 \text{ x } 0.85 \text{ x fc'}}{fy \times \frac{600}{(600+240)}}$$

$$= \frac{0.85 \text{ x } 0.85 \text{ x } 25'}{240 \times \frac{600}{(600+240)}}$$

= 0.05375744

$$\text{Rmax} = 0.75\ x\ \rho b\ x\ fy\ x\ \frac{(1-0.5\ x\ 0.75\ x\ \rho b\ x\ fy)}{(0.85\ x\ fc')}$$

$$= 0.75 \; x \; 0.05375744 \; x \; 240 \; x \; \frac{(1-0.5 \; x \; 0.75 \; x \; 0.05375744 \; x \; 240)}{(0.85 \; x \; 25)}$$

= 7.47324418

ϕ = 0.8

ds $= Ø + \frac{ts}{2}$

$= 10 + \frac{20}{2}$

= 20 mm

d = h − ds

= 120 – 20

= 100 mm

b = 1000 mm

Mn $= \frac{Mu}{\phi}$

$= \frac{4.1774}{0.18}$

= 5.221726088 kNm

Rn $= \frac{(\text{Mn x 1000000})}{(\text{b x Mn})^2}$

= 0.522172609 kNm

(Rn < Rmax) = OK

(0.522172609 < 7.47324418) = OK

ρ $= \frac{0.85 \text{ x fc'}}{\text{fy x } (1-(\frac{1-2 \text{ x Rn}}{(0.85 \text{ x fc'})})}$

$= \frac{0.85 \times 25}{240 \; x \; (1-\frac{1-2 \; x \; 0.52217}{(0.85 \; x \; 25})}$

= 0.002203129

ρmin $= \frac{1.4}{fy}$

$= \frac{1.4}{240}$

= 0.005833333

ρ used = 0.005833333

As = ρ used × b × d

$= 0.005833333 \times 1000 \times 100$

= 583.3333 mm^2

s $= \frac{\pi}{4^2} \times \frac{\phi}{As}$

$= \frac{3.14}{4^2} \times \frac{1000}{583.3333}$

= 134.6396852

smax = $2 \times h$

= 2×120

= 240 mm

Smax = 200 mm

s = 134.6396852 mm

s = 130 mm

Reinforcement Used = D10 - 130 mm

As $= \frac{\pi}{4} \times \phi \times \frac{b}{s}$

$= \frac{3.14}{4} \times 10 \times \frac{1000}{130}$

= 604.1524334 mm

Ec $= 4700 \times fc'$

$= 4700 \times 25$

= 23500 Mpa

Es $= 20000\ Mpa$

Q $= QD + QL$

= 4.63 + 4.8

= 9.430

Lx = Lx $\times$ 1000

= 1.95 x 1000

= 1950 mm

Lx/240 = $\frac{1950}{240}$

= 8.125 mm

lg $= \left(\frac{1}{12}\right) \times b \times h3$

$= (\frac{1}{12}) \times 1000 \times 1203$

= 144000000 mm^3

fr = 0.7 $\times$ √ fc'

= 0.7 x √ 25

= 3.5 Mpa

$$n = \frac{Es}{Ec}$$
$$= \frac{200000}{23500}$$
$$= 8.510638298$$
$$c = n \times \frac{As}{b}$$
$$= 8.510638298 \times (\frac{604.152}{1000})$$
$$= 5.141722837$$
$$Lcr = \frac{1}{3} \times b \times c^3 + n \times As \times (d - c)^2$$
$$= \frac{1}{3} \times 1000\ x\ 5.141723 + 8.51064\ x\ 604.12\ x\ (100 - 5.14172)2$$
$$= 46311010$$
$$yt = \frac{h}{2}$$
$$= \frac{120}{2}$$
$$= 60\ mm^4$$
$$Mcr = \frac{fr \times lg}{yt}$$
$$= \frac{3.5 \times 144000000}{60}$$
$$= 8400000\ mm$$
$$Ma = \frac{1}{8} \times Q \times Lx^2$$
$$= \frac{1}{8} \times 9.430 \times 19502$$
$$= 4482197\ Nmm$$
$$Ie = \left(\frac{Mcr}{Ma}\right)^3 \times Ig + (1 - \left(\frac{Mcr}{Ma}\right)^3 \times lcr$$
$$= \left(\frac{8400000}{4482197}\right)^3 \times 144000000 + (1 - \left(\left(\frac{8400000}{4482197}\right)\right)^3 \times 46311010$$
$$= 689310594\ Nmm$$
$$\delta e = \frac{5}{384} \times Q \times \frac{Lx4}{Ec} \times Ie$$
$$= \frac{5}{384} \times 9.430 \times \frac{1950^4}{23500} \times 689310594$$
$$= 0.110\ mm^4$$
$$\rho = \frac{As}{b} \times d'$$
$$= \frac{604.152}{1000\ x\ 100}$$
$$= 0.006$$

$\zeta \quad = 2$

$\lambda \quad = 1 + (50 \times \rho)$

$= 1 + (50 \times 0.006)$

$= 1.302$

$\delta g \quad = \lambda \times \frac{5}{384} \times Q \text{ x } \frac{Lx^4}{Ec} \times Ie$

$= 1.302 \times \frac{5}{384} \times 9.430 \times \frac{19504}{23500} \times 689310594$

$= 0.143$ mm

$\delta total \quad = \delta e + \delta g$

$= 0.110 + 0.143$

$= 0.252$ mm

Terms ($\delta tot \leq Lx / 240$)

$= 0.252 \leq 8.13$ (OK)

One Way Slab Result Recap

Slab Code	**Smax**	**S**	**Used Reinforcement**	**δtot**	$\frac{Lx}{240}$	$\delta tot < \frac{Lx}{240}$
Dormitory Building						
A	200	134.64	D10 - 130	4.647	11.25	**OK**
C	200	134.64	D10 - 130	0.019	6.25	**OK**
E	200	134.64	D10 - 130	0.019	6.25	**OK**
G	200	134.64	D10 - 130	0.019	6.25	**OK**
Educational Building						
A	200	134.64	D10 - 130	0.252	8.13	**OK**
C	200	134.64	D10 - 130	0.636	8.96	**OK**
D	200	134.64	D10 - 130	0.321	8.33	**OK**
E	200	134.64	D10 - 130	0.066	7.08	**OK**
F	200	134.64	D10 - 130	0.636	8.96	**OK**
G	200	134.64	D10 - 130	0.321	8.33	**OK**
H	200	134.64	D10 - 130	0.066	7.08	**OK**
I	200	134.64	D10 - 130	0.636	8.96	**OK**
J	200	134.64	D10 - 130	0.321	8.33	**OK**

K	200	134.64	D10 - 130	0.066	7.08	**OK**
L	200	134.64	D10 - 130	0.252	8.13	**OK**

2.10.2 Two Way Slab

In two way slab reinforcement the load received will be distributed by the slab in two directions with all four sides supported, if the slab rests on all four sides and the ratio Ly/Lx < 2 then the entire load will be transferred to all sides. 2 Approach methods in conducting system analysis and design two way structure in accordance with SNI2847:2019, among others.

a. Direct Design Method, DMM this method is formulated in SNI 2847:2019 article 8.10 where this method is limited to slab systems that are loaded by uniform loads. The use of a number of efficiency to determine the magnitude of the design moment at a critical location.
b. Equivalent Frame Method, EFM this method is formulated in SNI 2847:2019 article 8.11. The 3-dimensional building structure is divided into several two-dimensional equivalent frames, where the frame structures are then analyzed separately floor by floor in the longitudinal and transverse directions.

The analysis and design of the 2-way plate is focused on the direct design method (DMM) where in the analysis and design there are several limitations on its use which are regulated in article 8.10.2, these include:

a. There are at least 3 straight spans in each direction (8.10.2.1)
b. The lengths of adjacent spans, measured between the axes to the axes of the supports in each direction, do not differ and are more than one third of the longest span (8.10.2.2)
c. The rectangular slab with the ratio between the long span and the short span is measured from the axis to the support axis and does not exceed 2 (8.10.2.3)
d. Column positions may deviate by a maximum distance of 10% from the span length and the lines connecting adjacent column axes (8.10.2.4)
e. The load that is calculated is only the gravity load and is distributed evenly throughout the plate panel (8.10.2.5)
f. The live load shall not exceed 2 times the dead load (8.10.2.6)
g. For a slab panel with beams between supports on all sides (8.10.2.7)

Structure Material Data

Plat Code	= B		
Fc’	= 25 Mpa		
Floor Slab Data			
Fy	= 240 Mpa	Cly	= 16
Lx	= 2.15 m	Ctx	= 79
Ly	= 3.5 m	Cty	= 57
h	= 120 mm	ts	= 20 mm
Coef.Moment	$= \frac{Ly}{Lx} = \frac{3.5}{2.15}$ = 1.6 m	Ø	= 10 mm

1. Dead Load

Own Weight of Floor Plate

Unit Weight = 24

Thickness = 0.12 m

Q = 24×0.12 = 2.88 kN/m^2

2. Floor Finishing Weight

Unit Weight = 21

Thickness = 0.05 m

Q = 21 x 0.05 = 1.05 kN/m^2

Ceiling and Frame Weight = 0.2

ME Installation Weight = 0.5

QD = 2.88 + 1.05 + 0.2 + 1.05 = 4.63 kN/m^2

3. Live Load

Live Load Building Floors = 480 kg/m^2

QL = 4.8 kN/m^2

Factored Plan Load

QU = (1.2 x QD) + (1.2 x QL)

= (1.2 x 4.63) + (1.6 x 4.8)

= 13.236 kN/m^2

4. Slab Moment Due to Factored Load

Mulx = Clx $\times$ 0.001 $\times$ QU x Lx^2

= 37 x 0.001 x 13.236 x 2.15^2

= 2.2638 kNm/m

Muly = Cly $\times$ 0.001 $\times$ QU $\times$ Ly^2

= 16 x 0.001 x 13.236 x 3.5^2

= 2.5943 kNm/m

Mutx = Ctx $\times$ 0.001 $\times$ QU $\times$ Lx^2

= 79 x 0.001 x 13.236 x 2.15^2

= 4.8335 kNm/m

Muty = Cty x 0.001 x QU x Ly^2

= 57 x 0.001 x 13.236 x 2.15^2

= 9.2420 kNm/m

Mu = 9.2420 kNm/m

Slab Reinforcement

fc' $\leq$ 30 MPa

$\beta 1$ = 0.85

$\beta 1$ = -

Form Factor of Concrete Stress Distribution

$\beta 1$ = 0.85

$$\rho b = \frac{0.85 \text{ x } 0.85 \text{ x fc'}}{\frac{\text{fy x } 600}{(600 + 240)}}$$

$$= \frac{0.85 \text{ x } 0.85 \text{ x } 25}{\frac{240 \text{ x } 600}{(600 + 240)}}$$

= 0.05375744

Rmax = 0.75 x ρb x fy x $(\frac{1-0.5 \text{ x } 0.75 \text{ x } \rho b \text{ x } fy}{(0.85 \text{ x } fc')})$

= 0.75 x 0.05375744 x 240 x $(\frac{1-0.5 \text{ x } 0.75 \text{ x } 0.05375744 \text{ x } 240}{(0.85 \text{ x } 25)})$

= 7.47324418

ϕ = 0.8

ds = Ø + ts

= 10 + $\frac{20}{2}$

= 20 mm

d = h − ds

= 120 – 20

= 100 mm

b = 1000 mm

Mn = $\frac{Mu}{Ø}$

= $\frac{4.8335}{0.8}$

= 0.604186174 kNm

Rn = $\frac{(Mn \text{ x } 1000000)}{(b \text{ x } Mn)^2}$

= $\frac{(0.604186174 \text{ x } 1000000)}{(b \text{ x } 0.604186174)^2}$

= 0.604186174kNm

(Rn < Rmax) = OK

(0.604186174 < 7.47324418) = OK

ρ = $\frac{0.85 \text{ x } fc'}{fy \text{ x } (1-(\frac{1-2 \text{ x } Rn}{0.85 \text{ x } fc'})}$

= $\frac{0.85 \text{ x } 25}{240 \text{ x } (1-(\frac{1-2 \text{ x } 0.64186174}{0.85 \text{ x } 25})}$

= 0.002554286

ρmin = $\frac{1.4}{fy}$

= $\frac{1.4}{240}$

= 0.005833333

ρ used = 0.005833333

As = ρ used × b × d

= 0.005833333 x 1000 x 100

= 583.3333 mm^2

s $= \frac{\pi}{4^2} \times \frac{Ø}{As}$

$= \frac{\pi}{4^2}$ x $\frac{1000}{583.3333}$

= 134.6396852

smax = 2 × h

= 2 x 120

= 240 mm

Smax = 200 mm

s = 134.6396852 mm

s = 130 mm

Reinforcment used = 10D – 130

As $= \frac{\pi}{4} \times Ø \times \frac{b}{s}$

= 604.1524334 mm

Ec = 4700 × fc'

= 4700 x 25

= 23500 Mpa

Es = 20000 Mpa

Q = QD + QL

= 4.63 + 4.8

= 9.430

Lx = Lx × 1000

= 2.15 x 1000

= 2150 mm

Lx/240 $= \frac{2150}{240}$

= 8.958333333 mm

lg $= (\frac{1}{12}) \times b \times h3$

$= (\frac{1}{12})$ x 1000 x 120^3

= 144000000 mm^3

fr = 0.7 × √ fc'

= 0.7 x √ 25

= 3.5 Mpa

n $= \frac{Es}{Ec}$

$= \frac{200000}{23500}$

= 8.510638298

c = n × $\frac{As}{b}$

= 8.510638298 x $(\frac{604.152}{1000})$

= 5.141722837

Lcr $= \frac{1}{3} \times$ b x c ^{3} + n × As × (d – c)^2

$= \frac{1}{3}$ x 1000 x 5.14172^3 + 8.51064 x 604.12 x (100-5.14172)^2

= 46311010

yt $= \frac{h}{2}$

$= \frac{120}{2}$

= 60 mm^4

Mcr = fr × $\frac{lg}{yt}$

= 3.5 x $\frac{144000000}{60}$

= 8400000 mm

Ma $= \frac{1}{8} \times$ Q × Lx^2

$= \frac{1}{8}$ x 9.430 x 2150^2

= 5448772 Nmm

Ie $= (\frac{\mathrm{Mcr}}{\mathrm{Ma}})^3 \times \mathrm{Ig} + (1 - (\frac{\mathrm{Mcr}}{\mathrm{Ma}})^3 \times \mathrm{lcr}$

$= (\frac{8400000}{5448772})^3$ x 144000000 + (1-$(\frac{8400000}{5448772})^3$ x 46311010

= 404232357 Nmm

δe $= \frac{5}{384} \times \mathrm{Q} \times \frac{\mathrm{Lx}^4}{\mathrm{Ec}} \times \mathrm{Ie}$

$= \frac{5}{384}$ x 9.430 x $\frac{2150^4}{23500}$ x 404232357

= 0.276 mm^4

ρ $= \frac{\mathrm{As}}{\mathrm{b\ x\ d}}$

$= \frac{604.152}{1000 \times 100}$

$= 0.006$

$\zeta \quad = 2$

$\lambda \quad = 1 + (50\, x\, \rho)$

$= 1 + (50 \times 0.006)$

$= 1.302$

$\delta g \quad = \frac{\lambda \times 5}{\frac{384 \times Q \times Lx^4}{Ec \times Ie}}$

$= \frac{1.302 \times 5}{\frac{384 \times 9.430 \times 2150^4}{23500 \times 404232357}}$

$= 0.360$ mm

$\delta total \quad = \delta e + \delta g$

$= 0.276 + 0.360$

$= 0.636$ mm

Terms ($\delta tot \leq \frac{Lx}{240}$)

$= 0.636 \leq 8.96$ (OK)

Two Way Slab Result Recap

Slab Code	Smax	S	Used Reinforcement	δtot	$\frac{Lx}{240}$	δtot < $\frac{Lx}{240}$
Dormitory Building						
B	200	134.64	D10 - 130	10.277	12.50	**OK**
D	200	134.64	D10 - 130	10.277	12.50	**OK**
F	200	134.64	D10 - 130	0.066	7.08	**OK**
H	200	134.64	D10 - 130	6.178	11.67	**OK**
I	200	134.64	D10 - 130	0.066	7.08	**OK**
Educational Building						
B	200	134.64	D10 - 130	0.636	8.96	**OK**
M	200	134.64	D10 - 130	0.321	8.33	**OK**

2.11 Stairs Planning

In the design, the stairs plan required the minimum landing width. It's as wide as the stairs. So, the landing width is half the width of L1. Where the Optrede height (O) is between 0.15 m – 0.2 m. This makes the number of steps between floors same as the floor height divided by O ($n_{tg} = \frac{h_{lt}}{o}$). The size of the antrede (A) is determined to be 0.28 m – 0.3 m. So, the width of the stairs (Ltg) is $\left(\frac{1}{2}\frac{h_{lt}}{O} - 1\right)A$, the angle inclination of stairs can be calculated by $\propto = tan^{-1}\left(\frac{O}{A}\right)$ and stairs slab thickness (h_{tg}) is estimated.

The calculation of stairs load can be calculated using an equation below. For qtg load as follows,

Stairs own load = $\frac{h_{tg}}{\cos\alpha} \times concrete\ volume\ weight$ =…….kN/m^2....………(2.103)

Stairs step load = $\frac{1}{2}O \times concrete\ volume\ weight$ =……. kN/m^2 …………(2.104)

Tiles and spacing load = 0,05 x tiles volume weight =.….. kN/m^2…………..(2.105)

Railings load (estimated) =…..,, kN/m^2…………..(2.106)

For qbd load calculation is using equation below.

Stairs own laod = htg x berat volume beton =…..……….. kN/m^2…..(2.107)

Tiles and spacing load = 0,05 x berat volume ubin =……..…….. kN/m^2…..(2.108)

Railings load (estimated) = 1,0 kN/m^2…..................(2.109)

With cognizing the equation below,

$M_u = 1{,}4M_{DL}$……………………………………………………………..(2.110)

$M_u = 1{,}2M_{DL} + 1{,}6M_{LL}$……………………………………………..(2.111)

$V_u = 1{,}4V_{DL}$……………………………………………………………..(2.112)

$V_u = 1{,}2V_{DL} + 1{,}6V_{LL}$……………………………………………..(2.113)

In the design of stairs moment (Mur) produces area of tension reinforcement (Atg) in mm^2. The design shear force (Vur) is used to check the thickness of the stairs (htg) with Vc ≥ Vur jika Vc < Vur so, the thickness of the stairs needs to be enlarged. In designing the moment of staircase foundation slab plan, the equation used is,

$M_u = \frac{1}{2}\frac{(\sigma_{umax}+\sigma_{umin})}{2}\left(\frac{B}{2} + e - \frac{1}{2}b_{tg}\right)^2$ ……………………………………….(2.114)

And the calculation of the shear force of the stair foundation slab plan is calculated using the equation as follows,

$V_u = \frac{(\sigma_{umax}+\sigma_{umin})}{2}\left(\frac{B}{2} + e - \frac{1}{2}b_{tg}\right)^2$ ……………………………………….(2.115)

2.11.1 Dormitory and Educational Building Stairs Design

Stairs Dimension

Specification :

Width	= 3.4 m
Bordes Width	= 3.4/2 = 1.7 m
Optrede (o)	= 0.18 m
Floor Height (Het)	= 4 m
Number of Stairs	= 4 / 0.18 = 22 piece
Antrede (A)	= 0.3 m
Stairs Width (Ltg)	= (0.5 × 22 – 1) × 0.3 = 3.033 m
Alpha	= 0.6 = 30.96°
Htg	= 0.15
Concrete Unit Weight	= 24 kN/ m^2
Tile Unit Weight	= 21 kN/ m^2

LOAD

Qtg load

Stairs Unit Weight	$= \frac{htg}{\cos\propto} \times$ concrete volume weight
	$= \frac{0.15}{\cos 30.96} \times 24 = 4.2$ kN/m^2
Single Stairs Unit Weight	$= \frac{1}{2} \times O \times$ concrete volume weight
	$= \frac{1}{2} \times 0.2 \times 24 = 2.4$ kN/ m^2
Tile Unit Weight	= 0.05 × tiles volume weight
	= 0.05 × 21 = 1.05 kN/m^2
Railing Weight	= 1 kN/m^2
Qtg load (Total)	= 4.2 kN/m^2 + 2.4 kN/ m^2 + 1 kN/m^2
	= 8.65 kN/m^2

Qbd load

Stairs Unit Weight	= htg × concrete volume weight
	= 0.15 × 24 = 3.6 kN/m^2
Tile unit weight	= 0.05 × tiles volume weight
	= 0.05 × 21 = 1.05 kN/m^2

Railing weight = 1 kN/m^2

Qbd load (Total) = 3.6 kN/m^2 + 1.05 kN/m^2 + 1 kN/m^2

= 5.65 kN/m^2

Live load = 4.79 kN/m^2

Loading

MDL = 23.07 kNm

MLL = 14.71 kNm

VDL = 24.35 kN

VLL = 15.265 kN

Combination :

MU1 = 1.4 x 23.07 = 32.3085 kNm

MU2 = 1.2 x 23.07 + 1.6 x 14.71 = 51.2430 kNm

VU1 = 1.4 x 24.35 = 34.0998 kN

VU2 = 1.2 x 24.35 + 1.6 x 15.265 = 53.6524 kN

Used :

Mur = 51.2430 kNm

Vur = 53.6524 kN

Stairs Reinforcement Design At Support Field

Mur = 0.5 kNm

Mux = 0.5 x 59,688 x 10^{-3} = 0.025 kNm

Design :

Main Reinforcement = D16 ; As = 201.1 mm^2

Shrinkage Reinforcement = P10 ; As = 78.54 mm^2

Fy main reinforcement = 560 Mpa

Fy shrinkage reinforcement = 390 Mpa

F’c = 25 Mpa

B = 1 m; htg = 150 mm

Concrete Cover = 20 mm ; $\beta 1$ = 0.9

ds = 150 – 20 – (16/2)/1000 = 0.122 m

Calculation

Rn $= \frac{0,256}{0,9 \times 1 \times 0,122^2} = 1.9126$ kN/m^2

ρ min $= 0.0018$

ρ need $= \frac{0,85 f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2Rn}{0,85\ f'c}}\right]$

$= \frac{0,85 \times 25}{560}\left[1 - \sqrt{1 - \frac{2 \times 1.9126}{0,85 \times 25}}\right] = 0.0035$

ρ max $= 0.75 \times 0.85 \times 0.9 \times \frac{25}{560} \times (\frac{600}{600+560})$

$= 0.01325$

As min $= 0.0018 \times 1 \times 1000 \times 0.122 \times 1000 = 219.6$ mm^2

As need $= 0.00358 \times 1 \times 1000 \times 150 = 537.7256$ mm^2

S $= \frac{0,25\pi d^2 b}{As}$

$= \frac{0,25 \times \pi \times 16^2 \times 1000}{537.7256} = 373.9118$ mm

Used D16 – 300

As use $= \frac{0,25\pi d^2 b}{Htg}$

$= \frac{0,25 \times \pi \times 16^2 \times 1000}{150} = 1340.4129$

As need $= 631.8250$

Shear Force Check

Vc $= \frac{1}{6}\sqrt{f'c} \times b \times d$

$= \frac{1}{6}\sqrt{25} \times 1000 \times 0.122 = 101,667$ kN

ØVc $= 0.75 \times 101.667$

$= 76.25$ kN > Vur = 53.6524 (SAFE)

Shrinkage Reinforcement

ρ min $= 0.0018$

htg = 150 mm

bw = 1 m

As min = 0.0018 x 1 x 1000 x 150 = 270 mm2

S $= \frac{0.25\pi d^2 p}{As\ min} = \frac{0.25\pi \times 10^2 \times 1000}{270} = 290.8882$ mm

Used = 200 mm

Reinforcement P10-200

As use $= \frac{0.25\pi d^2 p}{Htg} = \frac{0.25\pi \times 10^2 \times 1000}{150}$ = 523.6 mm2

As use > As min (OK)

STAIR REINFORCEMENT DESIGN AT FIELD AREA

Mur = 0.8 kNm

Mux = 0.8 x 51.2430 x 10^{-3} = 0.0409 kNm

Design :

Main Reinforcement = D16 ; As = 201.1 mm^2

Shrinkage Reinforcement = P10 ; As = 78.54 mm^2

Fy main reinforcement = 560 Mpa

Fy shrinkage reinforcement = 390 Mpa

f'c = 25 Mpa

B = 1 m; htg = 150 mm

Concrete Cover = 20 mm ; $\beta 1$ = 0.9

ds = 150 – 20 – (16/2)/1000 = 0.122 m

Calculation :

Rn $= \frac{0.0409}{0,9 \times 1 \times 0,122^2}$ = 3.0602 kN/m^2

ρ min = 0.0018

ρ need $= \frac{0,85 f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2Rn}{0,85\ f'c}}\right]$

$= \frac{0,85 \times 25}{560}\left[1 - \sqrt{1 - \frac{2 \times 1.9126}{0,85 \times 25}}\right]$ = 0.0059

ρ max $= 0.75 \times 0.85 \times 0.9 \times \frac{25}{560} \times (\frac{600}{600+560})$

= 0.01325

As min = 0.0018 × 1 × 1000 × 0.122 × 1000 = 219.6 mm^2

As need = 0.00358 × 1 × 1000 × 150 = 889.1726 mm^2

S $= \frac{0.25\pi d^2 b}{As}$

$= \frac{0,25 \times \pi \times 16^2 \times 1000}{889.1726}$ = 226.1225 mm

Used D16 – 300

As use $= \frac{0.25\pi d^2 b}{Htg}$

$= \frac{0.25 \times \pi \times 16^2 \times 1000}{150}$ = 2234.0214

As need = 889.1726

As use > As need (OK)

Shear Force Check

Vc $= \frac{1}{6}\sqrt{f'c} \times b \times d$

$= \frac{1}{6}\sqrt{25} \times 1000 \times 0.122$ = 101.6667 kN

ØVc = 0.75 × 101.667

= 76.25 kN > Vur = 53.6524 (SAFE)

Shrinkage Reinforcement

ρ min = 0.0018

htg = 150 mm

bw = 1 m

As min = 0.002 x 1 x 1000 x 150 = 300 mm2

S $= \frac{0.25\pi d^2 p}{As\ min} = \frac{0.25\pi \times 10^2 \times 1000}{300}$ = 261.7994 mm

Used = 200 mm

Reinforcement P10-200

As use $= \frac{0.25\pi d^2 p}{Htg} = \frac{0.25\pi \times 10^2 \times 1000}{150}$ = 523.6 mm2

As use > As min (OK)

2.12 Conclusion

1. Assallafiyyah Islamic Boarding School has a medium land site class (SD), this library is included in category IV and is included in KDS D.
2. Assallafiyyah Islamic Boarding School has a medium land site class (SD), this library is included in category IV and is included in KDS D.
3. In planning the roof structure, this Boarding School has a limas roof that uses a profile C 200×75×20 thickness 2.0 for curtain rods, C 200×75×20 thickness 2.0 for truss element design and uses bolted connections in both buildings.
4. This Assallafiyyah Islamic Boarding School structural columns with a size of 500×500 mm. In addition, this building has 14 types of main beams with a size of 400×200 mm (BI 1 Dorm), a size of 350×200 mm (BI 2 Dorm, BI 2 Edu, and BI 3 Edu), a size of 300×200 mm (BI 3 Dorm and BI 6 Edu), size 250×200 mm (BI 4 Dorm, BI 4 Edu, BI 7 Edu, and BI 8 Edu), size 200×200 mm (BI 5 Dorm, BI 5 Edu and BI 9 Edu). There are 8 types of joists in this building with a size of

250×200 mm (BA 1 Dorm, BA 3 Dorm, BA 1 Edu and BA 5 Edu), size 350×200 mm (BA 2 Dorm, BA 2 Edu and BA 3 Edu) and size 400×250 mm (BA 4 Edu).

5. In planning directional slabs, Assallafiyyah Islamic Boarding School has 6 types of slabs using two-way reinforcement and 16 types of slabs using one-way reinforcement.
6. The design of stairs in building one has main reinforcement D16-300, field main reinforcement D16-150, and shrinkage reinforcement P10-200. In the second building, the design of the stairs has D10-300 main support reinforcement, D10-100 field main reinforcement, and P8-150 shrinkage reinforcement.