BAB V

ANALISIS DAN DISAIN STRUKTUR

Dimensi awal elemen struktur untuk satu rangking wilayah gempa adalah sama. Untuk rangking wilayah gempa 3, yaitu R/W 3/3 lama dan R/W 3/4 baru, dimensi elemen strukturnya sebagai berikut :

Kolom : 900 X 700

Balok : 500 X 300

Balok Anak : 450 X 300

Sedangkan untuk rangking wilayah gempa 4, yaitu R/W 4/4 lama dan R/W 4/3 baru, dimensi elemen strukturnya sebagai berikut :

Kolom : 800 X 700

Balok : 400 X 300

Balok Anak : 300 X 300

5.1 Perhitungan Gaya Geser Dasar Horizontal

5.1.1 Berat Total Struktur (W_t)

a) Lantai 12 (Atap)

Beban Mati

Plat atap : 40 x 18 x 0,10 x 24 = 1728,0000 kN

Balok induk : (0,50 – 0,10) x 0,30 x [(40x4 + 18x9)] x 24 = 927,3600 kN

Balok anak	: (0,45 – 0,10) x 0,30 x (40 x 2) x 24	=	201,6000 kN
Kolom	: 0,90 x 0,70 x 2 x 36 x 24	=	1088,6400 kN
Dinding	: [(40x4) + (18x9)] x 2 x 0,6 x 2,5	=	966,0000 kN
Plafond	: 40 x 18 x (0,11 + 0,07)	=	129,6000 kN
	W_D	=	5041,2000 kN

Beban Hidup

q_L = 1 kN/m^2

Koefisien reduksi = 0,3

W_L = 0,3 x (40 x 18) x 1 = 216,0000 kN

Berat Total Lantai 12 = W_{12} = W_D + W_L

= 5041,2000 + 216,0000 = 5257,2000 kN

b) Lantai 11

Beban Mati

Plat atap	: 40 x 18 x 0,12 x 24	=	2073,6000 kN
Balok induk	: (0,50 – 0,12) x 0,30 x [(40x4 + 18x9)] x 24	=	880,9920 kN
Balok anak	: (0,45 – 0,12) x 0,30 x (40 x 2) x 24	=	190,0800 kN
Kolom	: 0,90 x 0,70 x 4 x 36 x 24	=	2177,2800 kN
Dinding	: [(40x4) + (18x9)] x 4 x 0,6 x 2,5	=	1932,0000 kN
Plafond	: 40 x 18 x (0,11 + 0,07)	=	129,6000 kN
Spesi	: 40 x 18 x 0,02 x 21	=	302,4000 kN
Pasir	: 40 x 18 x 0,01 x 16	=	115,2000 kN
Tegel	: 40 x 18 x 0,02 x 24	=	345,6000 kN
	W_D	=	8146,7520 kN

Beban Hidup

q_L = 2,5 kN/m^2

Koefisien reduksi = 0,3

W_L = 0,3 x (40 x 18) x 2,5 = 540,0000 kN

Berat Total Lantai 11 = W_{11} = W_D + W_L

= 8146,7520 + 540,0000 = 8686,7520 kN

Berat lantai 1 sampai lantai 10 sama dengan berat lantai 11 (Tipikal)

Berat Total Struktur :

$\mathbf{W_t} = W_1 + W_2 + W_3 + W_4 + W_5 + W_6 + W_7 + W_8 + W_9 + W_{10} + W_{11} + W_{12}$

= (11 x 8686,7520) + 5257,2000

= 100811,4720 kN

Tabel 5.1 Berat Total Struktur

Lantai ke-i	hi (m)	Wi (kN)	Wi.hi (kN.m)
12	48	5.257,2000	252.345,6000
11	44	8.686,7520	382.217,0880
10	40	8.686,7520	347.470,0800
9	36	8.686,7520	312.723,0720
8	32	8.686,7520	277.976,0640
7	28	8.686,7520	243.229,0560
6	24	8.686,7520	208.482,0480
5	20	8.686,7520	173.735,0400
4	16	8.686,7520	138.988,0320
3	12	8.686,7520	104.241,0240
2	8	8.686,7520	69.494,0160
1	4	8.686,7520	34.747,0080
Σ Total =		100.811,4720	2.545.648,1280

5.1.2 Waktu Getar Bangunan (T)

Tinggi bangunan : H = 4 x 12 = 48 m

$T = 0{,}06\ H^{3/4}$ .... (3.2.1)

$= 0{,}06\ (48)^{3/4} = 1{,}0942$ detik

5.1.3 Koefisien Gempa Dasar (C)

Dari spektrum respon, untuk nilai T = 1,0942 detik dan jenis tanah lunak maka akan didapat nilai C untuk masing-masing rangking wilayah gempa (R/W).

- R/W (3/3$_{\text{Code Lama}}$) → C = 0,0667
- R/W (3/4$_{\text{Code Baru}}$) → C = 0,7768
- R/W (4/4$_{\text{Code Lama}}$) → C = 0,0476
- R/W (4/3$_{\text{Code Baru}}$) → C = 0,6854

5.1.4 Faktor Keutamaan Gedung (I), Faktor Jenis Struktur (K) dan Faktor Reduksi Gempa (R)

Untuk gedung yang difungsikan sebagai hotel, maka diambil faktor keutamaan gedung I = 1,0 dan karena struktur merupakan struktur beton dengan daktilitas penuh maka diambil nilai faktor jenis struktur K = 1,0 sedangkan nilai faktor reduksi gempanya R = 8,5.

5.1.5 Gaya Geser Horizontal Akibat Gempa

a). *Code* Lama → V = CIK Wt .... (3.2.2)

b). *Code* Baru → $V = \frac{C_1 I}{R} W_t$ .... (3.2.15)

- R/W (3/3$_{\text{Code Lama}}$) → V = 0,0667.1.1.100811,4720 = 6724,1252 kN
- R/W (3/4$_{\text{Code Baru}}$) → $V = \frac{0,7768.1}{8,5} 100811,4720 = 9212,9825$ kN
- R/W (4/4$_{\text{Code Lama}}$) → V = 0,0476.1.1.94210,5120 = 4798,6261 kN
- R/W (4/3$_{\text{Code Baru}}$) → $V = \frac{0,6854.1}{8,5} 94210,5120 = 8128,9627$ kN

5.1.6 Distribusi Gaya Geser Horizontal Total Akibat Gempa ke Sepanjang Tinggi Gedung (Fi).

$$\left.\begin{aligned} H &= 48m \\ B &= 18M \end{aligned}\right\} \frac{H}{B} = \frac{48}{18} = 2{,}667 < 3$$

maka seluruh beban didistribusikan sebagai gaya horizontal dengan

persamaan: $$Fi = \frac{Wi.hi}{\sum Wi.hi} V \quad \text{...} \quad (3.2.3)$$

Contoh perhitungan :

Untuk contoh perhitungan diambil perhitungan gaya horizontal pada R/W 3/3 dan R/W 3/4 untuk portal E.

Tabel 5.2 Hitungan Gaya Horizontal R/W 3/3 Portal E

Lantai ke-i	hi (m)	Wi (kN)	Wi.hi (kN.m)	C	I	K	V (kN)	Fi (kN)
12	48	5.257,2000	252.345,6000	0,0667	1	1	6.724,1252	666,5506
11	44	8.686,7520	382.217,0880	0,0667	1	1	6.724,1252	1.009,5958
10	40	8.686,7520	347.470,0800	0,0667	1	1	6.724,1252	917,8143
9	36	8.686,7520	312.723,0720	0,0667	1	1	6.724,1252	826,0329
8	32	8.686,7520	277.976,0640	0,0667	1	1	6.724,1252	734,2515
7	28	8.686,7520	243.229,0560	0,0667	1	1	6.724,1252	642,4700
6	24	8.686,7520	208.482,0480	0,0667	1	1	6.724,1252	550,6886
5	20	8.686,7520	173.735,0400	0,0667	1	1	6.724,1252	458,9072
4	16	8.686,7520	138.988,0320	0,0667	1	1	6.724,1252	367,1257
3	12	8.686,7520	104.241,0240	0,0667	1	1	6.724,1252	275,3443
2	8	8.686,7520	69.494,0160	0,0667	1	1	6.724,1252	183,5629
1	4	8.686,7520	34.747,0080	0,0667	1	1	6.724,1252	91,7814
Σ Total =		100.811,4720	2.545.648,1280					6.724,1252

Tabel 5.3 Hitungan Gaya Horizontal R/W 3/4 Portal E

Lantai ke-i	hi (m)	Wi (kN)	Wi.hi (kN.m)	C	I	R	V (kN)	Fi (kN)
12	48	5.257,2000	252.345,6000	0,7768	1	8,5	9.212,9825	913,2667
11	44	8.686,7520	382.217,0880	0,7768	1	8,5	9.212,9825	1.383,2860
10	40	8.686,7520	347.470,0800	0,7768	1	8,5	9.212,9825	1.257,5327
9	36	8.686,7520	312.723,0720	0,7768	1	8,5	9.212,9825	1.131,7794
8	32	8.686,7520	277.976,0640	0,7768	1	8,5	9.212,9825	1.006,0262
7	28	8.686,7520	243.229,0560	0,7768	1	8,5	9.212,9825	880,2729
6	24	8.686,7520	208.482,0480	0,7768	1	8,5	9.212,9825	754,5196
5	20	8.686,7520	173.735,0400	0,7768	1	8,5	9.212,9825	628,7664
4	16	8.686,7520	138.988,0320	0,7768	1	8,5	9.212,9825	503,0131
3	12	8.686,7520	104.241,0240	0,7768	1	8,5	9.212,9825	377,2598
2	8	8.686,7520	69.494,0160	0,7768	1	8,5	9.212,9825	251,5065
1	4	8.686,7520	34.747,0080	0,7768	1	8,5	9.212,9825	125,7533
Σ Total =		100.811,4720	2.545.648,1280					9.212,9825

Nilai-nilai gaya geser horizontal untuk tiap Rangking Wilayah gempa (R/W) lebih rinci diperlihatkan pada lampiran.

5.1.7 Waktu Getar Struktur dengan Cara *T Rayleigh*

Waktu getar struktur yang sebenarnya dihitung berdasarkan besar simpangan akibat beban gempa pada struktur menggunakan persamaan :

$$T = 2.\pi\sqrt{\left(\frac{\sum Wi.\delta i^2}{g.\sum Fi.\delta i}\right)} \quad \text{.......} \quad (3.2.7)$$

Diambil contoh perhitungan pada portal E.

f'c = 25 Mpa = 25000 kN/m^2

Ec = $4700\sqrt{f'c}$ = $4700\sqrt{25}$ = 23500 Mpa = 2,35 x 10^7 kN/m^2

Momen Inersia Kolom, Ix

$$Ix = \frac{1}{12}.b.h^3 \quad \text{.......} \quad (3.2.6)$$

$$= \frac{1}{12}x0,70x0,90^3$$

$$= 0,0425 \text{ m}^4$$

Kekakuan Tingkat, k

Kekakuan tingkat untuk kolom luar dan kolom dalam tiap lantai sama, karena dimensi kolom dan tinggi tiap tingkat sama.

$$k = \frac{12.E.Ix}{h^3} = \frac{12x2{,}35x10^7x0{,}0425}{4^3} \qquad \text{...} \quad (3.2.5)$$

$$= 187375{,}7813 \text{ kN/m}$$

Kekakuan tingkat tipikal (untuk 4 kolom) = 4 x 187375,7813 kN/m

= 749503,1250 kN/m

Tabel 5.4 Kontrol Periode Getar menurut Rayleigh untuk R/W 3/3

Lantai ke-i	Fi (kN)	Gaya Geser (kN)	Kekakuan Tk. k (kN/m)	Simpangan Antar Tk. Δ (m)	Simpangan Tk δi (m)	Δ h	Wi (kN)	Wi.δi2 (kN.m2)	Fi.δi (kN.m)
12	666,5506	666,5506	749.503,1250	0,0009	0,0726	0,0002	5.257,2000	27,7360	48,4148
11	1.009,5958	1.676,1464	749.503,1250	0,0022	0,0717	0,0006	8.686,7520	44,7143	72,4339
10	917,8143	2.593,9607	749.503,1250	0,0035	0,0695	0,0009	8.686,7520	41,9702	63,7965
9	826,0329	3.419,9936	749.503,1250	0,0046	0,0660	0,0011	8.686,7520	37,8948	54,5580
8	734,2515	4.154,2451	749.503,1250	0,0055	0,0615	0,0014	8.686,7520	32,8397	45,1456
7	642,4700	4.796,7151	749.503,1250	0,0064	0,0559	0,0016	8.686,7520	27,1858	35,9414
6	550,6886	5.347,4037	749.503,1250	0,0071	0,0495	0,0018	8.686,7520	21,3214	27,2826
5	458,9072	5.806,3109	749.503,1250	0,0077	0,0424	0,0019	8.686,7520	15,6227	19,4614
4	367,1257	6.173,4366	749.503,1250	0,0082	0,0347	0,0021	8.686,7520	10,4363	12,7250
3	275,3443	6.448,7809	749.503,1250	0,0086	0,0264	0,0022	8.686,7520	6,0656	7,2758
2	183,5629	6.632,3438	749.503,1250	0,0088	0,0178	0,0022	8.686,7520	2,7586	3,2712
1	91,7814	6.724,1252	749.503,1250	0,0090	0,0090	0,0022	8.686,7520	0,6992	0,8234
							Σtotal =	269,2445	391,1296

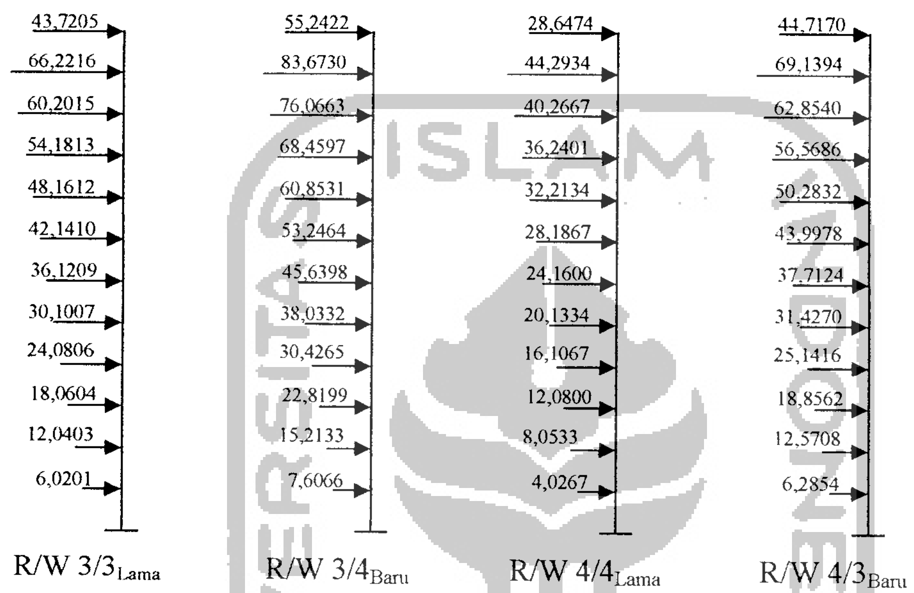
$$T = 2\pi\sqrt{\left(\frac{\sum Wi.\delta i^2}{g.\sum Fi.\delta i}\right)} = 2\pi\sqrt{\left(\frac{269{,}2445}{9{,}81x391{,}1296}\right)} \qquad \text{.....................................} \quad (3.2.7)$$

$$= 1{,}6689 \text{ detik}$$

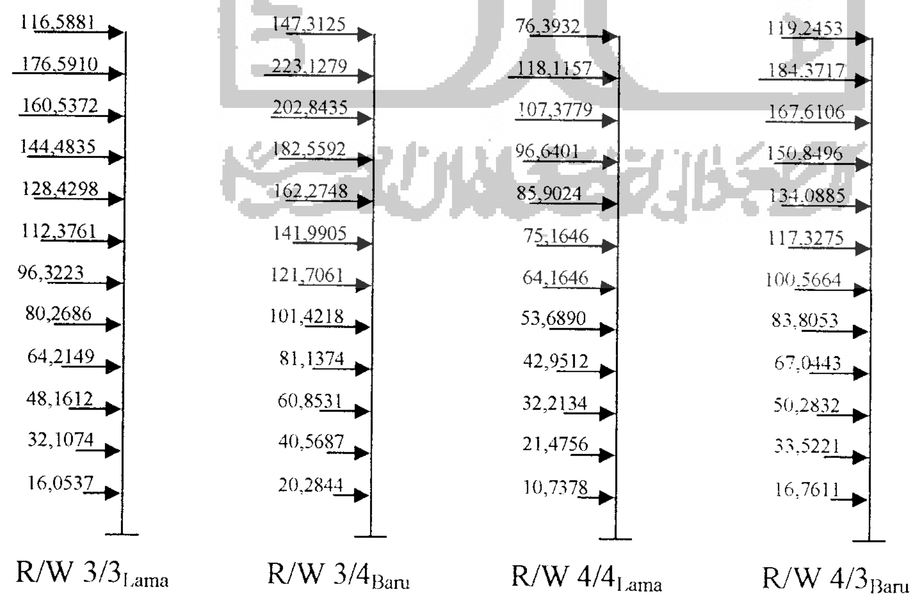
Nilai-nilai waktu getar Rayleigh T untuk masing-masing R/W ditampilkan dalam lampiran.

Selanjutnya setelah Waktu Getar T menurut Rayleigh didapat untuk masing-masing R/W, maka siklus diulangi lagi mulai dari menghitung koefisien gempa dasar C. Siklus ulang atau kontrol hitungan gaya horizontal dan waktu getar T Rayleigh iterasi II ditampilkan dalam lampiran.

Sehingga untuk masing-masing arah (arah X dan arah Y), nilai gaya horizontal harus dibagi dengan jumlah portal dikurangi satu (n-1). Nilai akhir dari gaya horizontal tiap portal pada masing-masing arah ditampilkan pada gambar berikut :

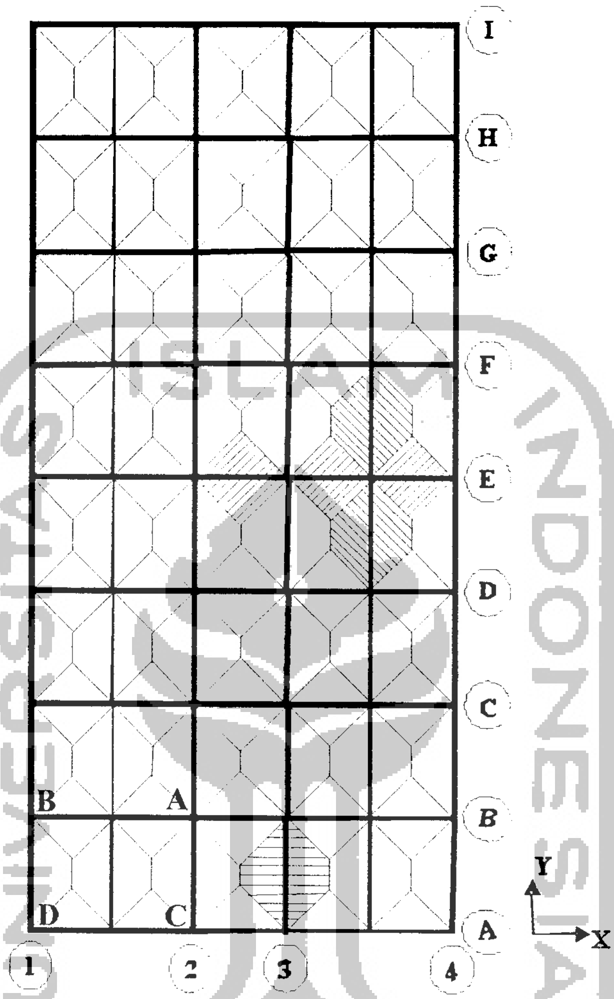


Gambar 5.1 Gaya Geser Horizontal Portal E untuk Masing-Masing R/W



Gambar 5.2 Gaya Geser Horizontal Portal 2 untuk Masing-Masing R/W

5.2 Perhitungan Beban Akibat Gaya Gravitasi

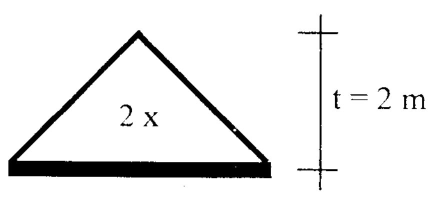


Gambar 5.3 Pembagian Beban Gravitasi

5.2.1 Perhitungan Beban Gravitasi untuk Portal arah X

Diambil contoh perhitungan pada portal E

A. Bentang Balok 4 m



a. Beban segitiga pada balok lantai atap (lantai 12)

➢ Beban mati untuk tiap m^1

- Plat	= 2 (0,1 x 24 x 2)	=	9,600 kN/m^1
- Plafon	= 2 (0,18 x 2)	=	0,72 kN/m^1
- Balok	= (0,5-0,1) x 0,30 x 24	=	2,88 kN/m^1
- L.Kedap Air	= 2 (0,02 x 21 x 2)	=	1,68 kN/m^1
		q_D =	14,88 kN/m^1

➢ Beban hidup untuk tiap m^1

q_L = 2 (2 x 1 KN/m^2)

= 4 kN/m^1

b. Beban segitiga pada balok lantai 1 s/d lantai 11

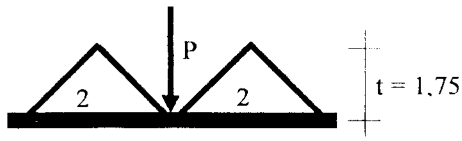
➢ Beban mati untuk tiap m^1

- Plat	= 2 (0,12 x 24 x 2)	=	11,52 kN/m^1
- Plafon	= 2 (0,18 x 2)	=	0,72 kN/m^1
- Balok	= (0,6-0,12) x 0,35 x 24	=	2,736 kN/m^1
- Dinding	= 4 x 2,5 x 0,6	=	6,00 kN/m^1
- Spesi	= 2 (0,02 x 21 x 2)	=	1,68 kN/m^1
- Pasir	= 2 (0,01 x 16 x 2)	=	0,64 kN/m^1
- Tegel	= 2 (0,02 x 24 x 2)	=	1,92 kN/m^1
	q_D	=	25,216 kN/m^1

➢ Beban hidup untuk tiap m^1

q_L = 2 (2 x 2,5 kN/m^2) = 10 kN/m^1

B. Bentang Balok 7 m



a. Beban segitiga pada balok lantai atap (lantai 12)

➢ Beban mati untuk tiap m^1

- Plat	= 2 (0,1 x 24 x 1,75)	= 8,40	kN/m^1
- Plafon	= 2 (0,18 x 1,75)	= 0,63	kN/m^1
- Balok	= (0,5-0,1) x 0,30 x 24	= 2,88	kN/m^1
- L.Kedap Air	= 2 (0,02 x 21 x 1,75)	= 1,47	kN/m^1
	q_D	= 13,38	kN/m^1

➢ Beban hidup untuk tiap m^1

q_L = 2 (1,75 x 1 kN/m^2)

= 3,5 kN/m^1

b. Beban Titik pada balok lantai atap (lantai 12)

- Plat	= 2 (0,1 x 24 x 1,75)	=	8,40	kN/m^1
- Plafon	= 2 (0,18 x 1,75)	=	0,63	kN/m^1
- Balok anak	= (0,45-0,1) x 0,30 x 24	=	2,52	kN/m^1
- L.Kedap Air	= 2 (0,02 x 21 x 1,75)	=	1,47	kN/m^1
	q_D	=	13,02	kN/m^1
P= 2 (13,02 x 2,5)		=	65,1	kN

c. Beban segitiga pada balok lantai 1 s/d lantai 11

➢ Beban mati untuk tiap m^1

- Plat = 2 (0,12 x 24 x 1,75) = 10,08 kN/m^1

- Plafon	= 2 (0,18 x 1,75)	=	0,63 kN/m^1
- Balok	= (0,5-0,12) x 0,30 x 24	=	2,736 kN/m^1
- Dinding	= 4 x 2,5 x 0,6	=	6,00 kN/m^1
- Spesi	= 2 (0,02 x 21 x 1,75)	=	1,47 kN/m^1
- Pasir	= 2 (0,01 x 16 x 1,75)	=	0,56 kN/m^1
- Tegel	= 2 (0,02 x 24 x 1,75)	=	1,68 kN/m^1
	q_D	=	23,156 kN/m^1

- Beban hidup untuk tiap m^1

  q_L = 2(1,75 x 2,5 KN/m^2) = 8,75 kN/m^1

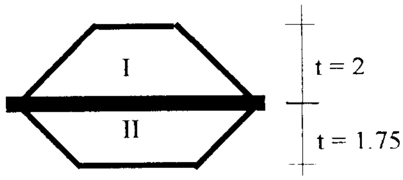
d. Beban Titik pada balok lantai 1 s/d lantai 11

- Plat	= 2 (0,12 x 24 x 1,75)	=	10,08 kN/m^1
- Plafon	= 2 (0,18 x 1,75)	=	0,63 kN/m^1
- Balok anak	= (0,45-0,12) x 0,30 x 24	=	2,376 kN/m^1
- Dinding	= 4 x 2,5 x 0,6	=	6,00 kN/m^1
- Spesi	= 2 (0,02 x 21 x 1,75)	=	1,47 kN/m^1
- Pasir	= 2 (0,01 x 16 x 1,75)	=	0,56 kN/m^1
- Tegel	= 2 (0,02 x 24 x 1,75)	=	1,68 kN/m^1
	q_D	=	22,796 kN/m^1

P = 2 (22,796 x 2,5) = 113,98 kN

5.2.1 Perhitungan Beban Gravitasi untuk Portal Arah Y

Diambil contoh perhitungan pada portal 2



Luasan Trapesium I

A. Beban trapesium balok lantai atap (lantai 12)

a. Beban mati untuk tiap m^1

- Plat	= 0,1 x 24 x 2	=	4,80 kN/m^1
- Plafon	= 0,18 x 2	=	0,36 kN/m^1
- Balok	= ((0,5-0,12) x 0,30 x 24)0,5	=	1,188 kN/m^1
- L.Kedap Air	= 0,02 x 21 x 2	=	0,84 kN/m^1
	q_D	=	7,692 kN/m^1

b. Beban hidup untuk tiap m^1

q_L = 2 x 1 kN/m^2 = 2,00 kN/m^1

B. Beban trapesium balok lantai 1 s/d lantai 11

a. Beban mati untuk tiap m^1

- Plat	= 0,12 x 24 x 2	=	5,76 kN/m^1
- Plafon	= 0,18 x 2	=	0,36 kN/m^1
- Balok	= ((0,5-0,12) x 0,30 x 24)0,5	=	1,188 kN/m^1
- Dinding	= 4 x 2,5 x 0,6 x 0,5	=	3,00 kN/m^1
- Spesi	= 0,02 x 21 x 2	=	0,84 kN/m^1
- Pasir	= 0,01 x 16 x 2	=	0,32 kN/m^1
- Tegel	= 0,02 x 24 x 2	=	0,96 kN/m^1
	q_D	=	12,788 kN/m^1

b. Beban hidup untuk tiap m^1

q_L = 2 x 2,5 kN/m^2 = 5,00 kN/m^1

Luasan Trapesium II

A. Beban trapesium balok lantai atap (lantai 12)

a. Beban mati untuk tiap m^1

- Plat	= 0,1 x 24 x 1,75	=	4,20 kN/m^1
- Plafon	= 0,18 x 1,75	=	0,315 kN/m^1
- Balok	= ((0,5-0,10) x 0,30 x 24) 0,5	=	1,44 kN/m^1
- L.Kedap Air	= 0,02 x 21 x 2	=	0,84 kN/m^1
	q_D	=	6,795 kN/m^1

b. Beban hidup untuk tiap m^1

q_L = 1,75 x 1 kN/m^2 = 1,75 kN/m^1

B. Beban trapesium balok lantai 1 s/d lantai 11

a. Beban mati untuk tiap m^1

- Plat	= 0,12 x 24 x 1,75	=	5,04 kN/m^1
- Plafon	= 0,18 x 1,75	=	0,315 kN/m^1
- Balok	= ((0,5-0,12) x 0,30 x 24)0,5	=	1,188 kN/m^1
- Dinding	= 4 x 2,5 x 0,6 x 0,5	=	3,00 kN/m^1
- Spesi	= 0,02 x 21 x 1,75	=	0,735 kN/m^1
- Pasir	= 0,01 x 16 x 1,75	=	0,28 kN/m^1
- Tegel	= 0,02 x 24 x 1,75	=	0,84 kN/m^1
	q_D	=	12,758 kN/m^1

b. Beban hidup untuk tiap m^1

q_L = 1,75 x 2,5 kN/m^2 = 4,375 kN/m^1

5.3 Perancangan Struktur Portal

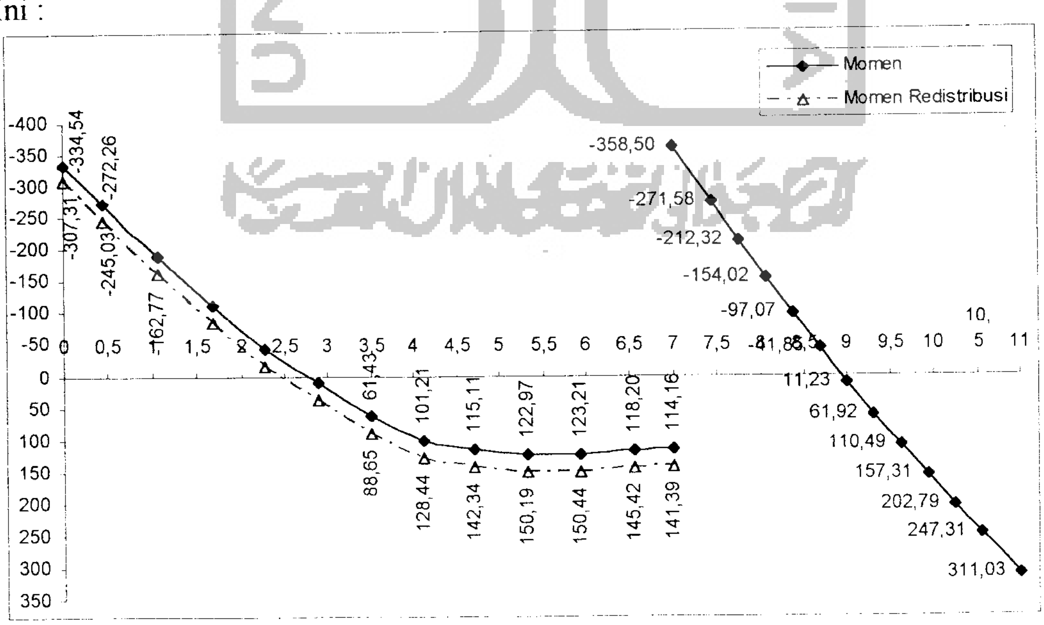
Semua data yang telah ada, seperti dimensi balok dan kolom, pembebanan beban mati, beban hidup, dan beban gempa serta kombinasi pembebanan kemudian dimasukkan ke program SAP2000 sebagai *input*. Hasil dari pengolahan data dengan program SAP2000 tersebut berupa momen, gaya aksial,dan gaya geser.

Sebagai contoh perhitungan diambil portal E bentang 7m lantai 3 R/W 3/3 lama. Hasil output SAP2000 ditampilkan dalam tabel 5.5.1 berikut ini :

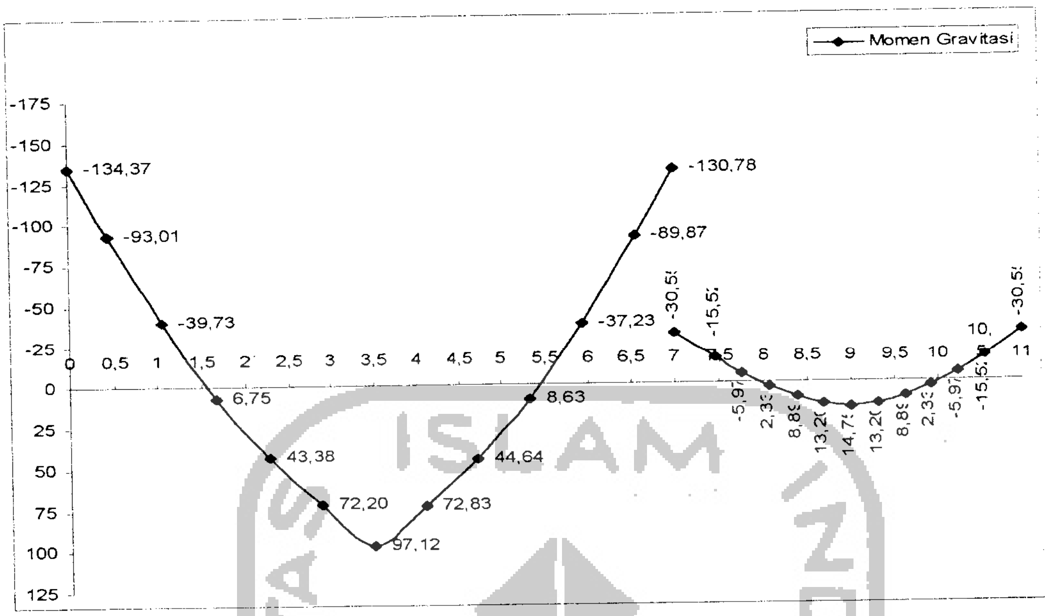
Tabel 5.5.1 Output SAP untuk Portal E Bentang 7m Lantai 3

Lantai	Frame	Stasion	Kombo1	Kombo2	Kombo3	Kombo4	Kombo5	M+	M-	M	%	Distribusi	M red	M perlu
3	51	0	-134,3745	112,9040	-334,5375	114,1602	-269,3611	114,1602	-334,5375	-334,5375	10	-27,2257	-307,3118	M - maks :
		0,45	-93,0070	118,1992	-272,2569	112,5053	-222,1714	118,1992	-272,2569	-272,2569	10	-27,2257	-245,0312	-272,2569
		1,06	-39,7335	123,2099	-189,9993	109,1368	-159,3283	123,2099	-189,9993	-189,9993	10	-27,2257	-162,7736	M + maks :
		1,67	6,7480	122,9678	-112,9945	103,0408	-99,2127	122,9678	-112,9945	-112,9945	10	-27,2257	-85,7688	123,2099
		2,28	43,3833	115,1109	-43,6045	92,9909	-43,0509	115,1109	-43,6045	-43,6045	10	-27,2257	-16,3788	p'/p :
		2,89	72,2043	101,2106	19,7421	79,8029	9,9727	101,2106	9,9727	9,9727	10	-27,2257	37,1984	-0,4526
		3,5	97,1167	84,2875	80,0659	65,0454	61,4269	84,2875	61,4269	61,4269	10	-27,2257	88,6525	========
		4,11	72,8314	24,4488	97,4741	13,8436	76,4367	97,4741	13,8436	101,2106	10	-27,2257	128,4363	M-red maks:
		4,72	44,6375	-38,4127	111,8595	-38,9278	89,8769	111,8595	-38,9278	115,1109	10	-27,2257	142,3366	-245,0312
		5,33	8,6292	-107,3176	120,2015	-94,8372	100,1792	120,2015	-107,3176	122,9678	10	-27,2257	150,1935	M+red maks:
		5,94	-37,2252	-183,8372	120,9287	-154,7005	106,5275	120,9287	-183,8372	123,2099	10	-27,2257	150,4356	150,4356
		6,55	-89,8717	-265,6097	116,4031	-217,2913	110,1483	116,4031	-265,6097	118,1992	10	-27,2257	145,4249	p'/p red :
		7	-130,7762	-327,5323	111,4660	-264,2950	111,9893	111,9893	-327,5323	114,1602	10	-27,2257	141,3859	-0,6139

Dari hasil momen tersebut dapat digambarkan seperti gambar-gambar di bawah ini :



Gambar 5.4.1 Momen Akibat Gempa Portal E Bentang 7m Lantai 3 yang Teredistribusi 10%



Gambar 5.4.2 Momen Akibat Berat Sendiri Portal E Bentang 7m Lantai 3

Dari tabel dan gambar di atas dapat diketahui momen perlu untuk disain tulangan lentur balok, antara lain :

M_u tumpuan negatif = 245,0312 kNm

M_u tumpuan positif = 150,4356 kNm

M_u lapangan = 97,1167 kNm

5.3.1 Disain Balok

Sebagai contoh perhitungan diambil pada balok portal E bentang 7m lantai 3.

5.3.1.1 Disain Tulangan Lentur Balok

1. Balok Tumpuan

   a. Tulangan Tumpuan Negatif

   Dari output SAP2000 teredistribusi (Tabel 5.5.1) didapat :

M_{perlu} = 245,0312 kNm

$$M_n = \frac{M_{perlu}}{\phi} = \frac{245{,}0312}{0{,}8} = 306{,}2890 \text{ kNm}$$

Data-data :

b = 300 mm h = 500 mm

d = 449 mm L = 7000 mm

fy = 400 MPa fc' = 25 MPa

Cek dimensi balok terhadap tulangan rangkap.

$$R_{maks} = 0{,}6375.f_c'.\beta_1.A.\left\{\frac{B-(0{,}375.\beta_1.A)}{B^2}\right\} \quad \text{..........................} \quad (3.3.4)$$

A = $(\varepsilon_c.E_s)$ = (0,003.200000) = 600

B = $((\varepsilon_c.E_s)+f_y)$ = ((0,003.200000)+400) = 1000

$$R_{maks} = 0{,}6375.25.0{,}85.600.\left\{\frac{1000-(0{,}375.0{,}85.600)}{1000^2}\right\}$$

= 6,5736

$M_{n,maks}$ = R_{maks} . $(b.d^2)$ .... (3.3.3)

= 6,5736 . (300.449^2) = 397574575,8363 Nmm

= 397,5746 kNm > M_n = 306,2890 kNm

ukuran balok terlalu besar, maka cek ulang ukurannya

$M_{n,maks}$ = R_{maks} . $(b.d^2)$

306,2890 . 10^6 = 6,5736 . $(b.d^2)$

$$(b.d^2) = \frac{306{,}2890.10^6}{6{,}5736} = 46593651{,}8293$$

ambil b = 0,5.d maka:

$(0{,}5.d^3)$ = 46593651,8293

$$d = \sqrt[3]{\frac{46593651,8293}{0,5}} = 453,3694 \text{ mm}$$

b = 0,5.d = 0,5 . 453,3694 = 226,6847 mm,

diambil b_{baru} = 250 mm

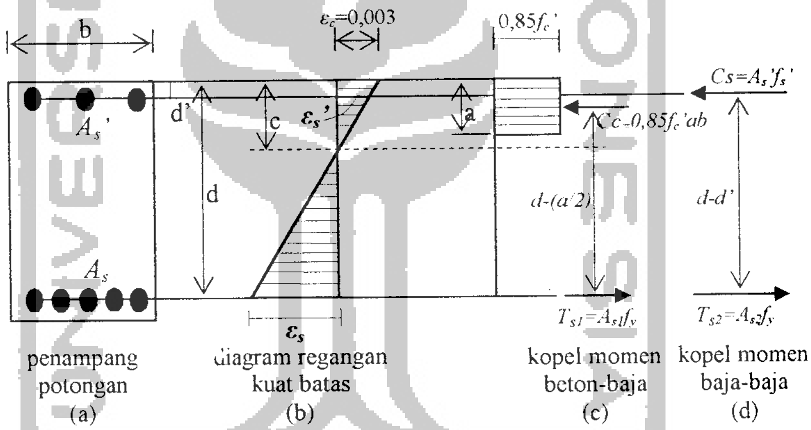
Tulangan tarik dianggap 2 lapis, d' = 75 mm

h = d + d' = 453,369 + 75 = 528,3694 mm,

diambil h_{baru} = 500 mm

d_{baru} = h - d' = 500 - 75 = 425 mm

balok didisain menggunakan tulangan rangkap:



Gambar 5.4.3 Diagram Regangan dan Keseimbangan Balok Bertulangan Rangkap

Kopel Beton Desak - Baja Tarik :

R_d = 0,55.R_{maks} = 0,55. 6,5736 = 3,6155 .... (3.3.9)

$M_{d1} = R_d \cdot (b.d^2)$ .... (3.3.10)

$= 3,6155. (250.425^2)$ = 163262042,6331 Nmm

= 163,2620 kNm

$$M = 0{,}85.fc'.a.b.\left(d - \frac{a}{2}\right) \quad \text{.......................} \quad (3.3.7)$$

$$163262042{,}6331 = 0{,}85.25.a.250.\left(425 - \frac{a}{2}\right)$$

$$163262042{,}6331 = 2257812{,}5a - 2656{,}25a^2$$

$$2656{,}25a^2 - 2257812{,}5a + 163262042{,}6331 = 0$$

$a = 79{,}8020$ mm

$Cc = 0{,}85.fc'.a.b$ .... (3.3.5)

$= 0{,}85 \ .\ 25 \ .\ 79{,}8020 \ .\ 250$

$= 423948{,}2514$ N

$Ts_1 = As_1 \ .\ fy$ .... (3.3.6)

$Cc = Ts$

$423948{,}2514 = As_1 \ .\ fy$

$$As_1 = \frac{423948{,}2514}{400}$$

$= 1059{,}8706 \text{ mm}^2$

Dipakai D22, AØ22 = $379{,}94 \text{ mm}^2$

$$n = \frac{1059{,}8706}{379{,}94} = 2{,}7896 \approx 3 \text{ tulangan}$$

Kopel Baja Desak – Baja Tarik :

$M_{d2} = Mn - M_{d1}$ .... (3.3.17)

$= 306{,}2890 - 163{,}2620$

$= 143{,}0270$ kNm

$= 143{,}0270 \ .\ 10^6$ Nmm

$M_{d2} = Cs . (d - d')$

$Cs = \dfrac{M_{d2}}{d - d'}$ .... (3.3.18)

$= \dfrac{143,0270 . 10^6}{425 - 75} = 408648,4853$ N

$Ts_2 = As_2 . fy$ .... (3.3.6)

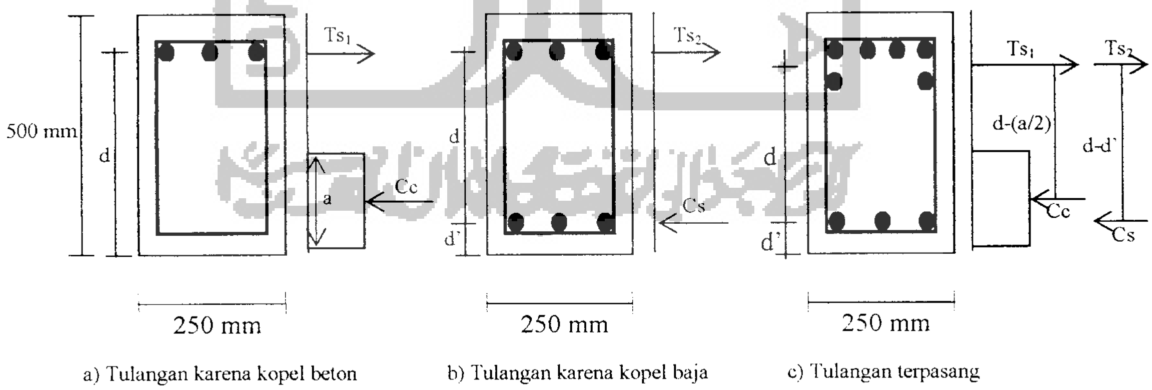
$Ts_2 = Cs = 408648,4853$ N

$As_2 = \dfrac{T_{s2}}{fy} = \dfrac{408648,4853}{400} = 1021,6212 \text{ mm}^2$

Dipakai D22, AØ22 = 379,94 mm^2

$n = \dfrac{1021,6212}{379,94} = 2,6889 \approx 3$ tulangan

Dari pasangan kopel beton – baja dan baja- baja, didapat tulangan tumpuan terpasang seperti diperlihatkan pada gambar di bawah ini :



Gambar 5.4.4 Penampang balok tumpuan

1. Mencari $M_{tersedia}$ - :

$$a_k = \frac{As.fy - As'.fy}{0{,}85.f'c.b} \quad \text{......} \quad (3.3.27)$$

$$= \frac{(2279{,}64.400) - (1139{,}82.400)}{0{,}85.25.250} = 85{,}8217 \text{ mm}$$

$$a = \frac{\beta_1.d'.Es.\varepsilon c}{Es.\varepsilon c - fy} \quad \text{......} \quad (3.3.28)$$

$$= \frac{0{,}85.75.200000.0{,}003}{(200000.0{,}003) - 400} = 191{,}25 \text{ mm}$$

Karena $a_k < a$, maka tulangan desak belum luluh.

$$Cc = 0{,}85.fc'.0{,}85c.b \quad \text{......} \quad (3.3.38)$$

$$= 0{,}85.25.0{,}85c.250 = 4515{,}6250c$$

$$Cs = As'.\frac{c - d'}{c}Es.\varepsilon_c \quad \text{......} \quad (3.3.37)$$

$$= 1139{,}82.\frac{c - 75}{c}200000.0{,}003 = 683892 - \frac{51291900}{c}$$

$$Ts = As.fy = 2279{,}64.400 = 911856 \text{ N} \quad \text{......} \quad (3.3.39)$$

Keseimbangan gaya-gaya dalam

$$Ts = Cc + Cs \quad \text{......} \quad (3.3.26)$$

$$911856 = 4515{,}6250c + 683892 - \frac{51291900}{c}$$

$$911856c = 4515{,}6250c^2 + 683892c - 51291900$$

$$4515{,}6250c^2 + 683892c - 911856c - 68389200 = 0$$

$$4515{,}6250c^2 - 227964c - 51291900 = 0$$

$$c = 134{,}7675 \text{ mm}$$

a = 0,85.c = 0,85. 134,7675 = 114,5524 mm

$$fs' = \frac{c-d'}{c} Es.\varepsilon c \quad \text{.......... (3.3.36)}$$

$$= \frac{134,7675-75}{134,7675} 200000.0,003 = 266,0916 \text{ Mpa}$$

$$M_1 = 0,85.fc'.a.b.\left(d - \frac{a}{2}\right) \quad \text{.......... (3.3.43)}$$

$$= 0,85.25.114,5524.250.\left(425 - \frac{114,5524}{2}\right)$$

= 223781812,3579 Nmm

$$M_2 = As'.fs'.(d-d') \quad \text{.......... (3.3.44)}$$

$$= 1139,82.266,0916.(425-75) = 106153782,5172 \text{ Nmm}$$

$M_{tersedia}$- = M_1 + M_2 .... (3.3.45)

= 223781812,3579 + 106153782,5172

= 329935594,8751 Nmm

= 329,9356 kNm > Mu = 245,0312 kNm

2. Mencari $M_{kapasitas}$ - :

$$a_k = \frac{As.fy.\phi - As'.fy}{0,85.f'c.b} \quad \text{.......... (3.3.27)}$$

$$= \frac{(2279,64.400.1,25)-(1139,82.400)}{0,85.25.250} = 128,7326 \text{ mm}$$

$$a = \frac{\beta_1.d'.Es.\varepsilon c}{Es.\varepsilon c - fy} \quad \text{.......... (3.3.28)}$$

$$= \frac{0,85.75.200000.0,003}{(200000.0,003)-400} = 191,25 \text{ mm}$$

Karena $a_k < a$, maka tulangan desak belum luluh.

$$Cc = 0{,}85.fc'.0{,}85c.b \quad \text{..........} \quad (3.3.38)$$

$$= 0{,}85.25.0{,}85c.250 = 4515{,}6250c$$

$$Cs = As'.\frac{c-d'}{c}Es.\varepsilon_c \quad \text{..........} \quad (3.3.37)$$

$$= 1139{,}82.\frac{c-75}{c}200000.0{,}003 = 683892 - \frac{51291900}{c}$$

$$Ts = As.fy.\phi = 2279{,}64.400.1{,}25 = 1139820 \text{ N} \quad \text{..........} \quad (3.3.39)$$

Keseimbangan gaya-gaya dalam

$$Ts = Cc + Cs \quad \text{..........} \quad (3.3.26)$$

$$1139820 = 4515{,}6250c + 683892 - \frac{51291900}{c}$$

$$1139820c = 4515{,}6250c^2 + 683892c - 51291900$$

$$4515{,}6250c^2 + 683892c - 1139820c - 51291900 = 0$$

$$4515{,}6250c^2 - 455928c - 51291900 = 0$$

c = 168,4127 mm

a = 0,85.c = 0,85. 168,4127 = 143,1508 mm

$$fs' = \frac{c-d'}{c}Es.\varepsilon c \quad \text{..........} \quad (3.3.36)$$

$$= \frac{168{,}4127-75}{168{,}4127}200000.0{,}003 = 332{,}7993 \text{ Mpa}$$

$$M_1 = 0{,}85.fc'.a.b\left(d - \frac{a}{2}\right) \quad \text{..........} \quad (3.3.43)$$

$$= 0{,}85.25.143{,}1508.250\left(425 - \frac{143{,}1508}{2}\right)$$

= 268775413,2360 Nmm

$M_2 = As'.fs'.(d - d') = 1139{,}82.332{,}7993.(425 - 75)$ .... (3.3.44)

$= 132765950{,}0301$ Nmm

$M_{kapasitas}$ - = $M_1 + M_2$ .... (3.3.45)

= 268775413,2360 + 132765950,0301

= 401541363,2661 Nmm

= 401,5414 kNm > Mu = 245,0312 kNm

b. Tulangan Tumpuan Positif

M_{perlu} = 150,4356 kNm

$$M_n = \frac{M_{perlu}}{\phi} = \frac{150{,}4356}{0{,}8} = 188{,}0445 \text{ kNm}$$

Dari hasil disain tulangan tumpuan negatif diperoleh jumlah tulangan untuk tumpuan positif sebagai berikut :

tulangan tarik, n = 3 tulangan

As = n . AØ22 = 3 . 379,94 = 1139,82 mm^2

tulangan tekan, n = 6 tulangan

As' = n . AØ22 = 6 . 379,94 = 2279,64 mm^2

1. Mencari $M_{tersedia}$ – :

$$a_k = \frac{As.fy - As'.fy}{0{,}85.f'c.b} \quad \text{... (3.3.27)}$$

$$= \frac{(1139{,}82.400) - (2279{,}64.400)}{0{,}85.25.250} = -85{,}8217 \text{ mm}$$

$$a = \frac{\beta_1.d'.Es.\varepsilon c}{Es.\varepsilon c - fy} \quad \text{.. (3.3.28)}$$

$$= \frac{0{,}85.75.200000.0{,}003}{(200000.0{,}003) - 400} = 191{,}25 \text{ mm}$$

Karena a_k < a, maka tulangan desak belum luluh.

$$Cc = 0{,}85.fc'.0{,}85c.b \quad \text{.......................} \quad (3.3.38)$$

$$= 0{,}85.25.0{,}85c.250 = 4515{,}6250c$$

$$Cs = As'.\frac{c-d'}{c}Es.\varepsilon_c \quad \text{.......................} \quad (3.3.37)$$

$$= 2279{,}64.\frac{c-75}{c}200000.0{,}003 = 1367784 - \frac{102583800}{c}$$

$$Ts = As.fy = 1139{,}82.400 = 455928 \text{ N} \quad \text{.......................} \quad (3.3.39)$$

Keseimbangan gaya-gaya dalam

$$Ts = Cc + Cs \quad \text{.......................} \quad (3.3.26)$$

$$455928 = 4515{,}6250c + 1367784 - \frac{102583800}{c}$$

$$455928c = 4515{,}6250c^2 + 1367784c - 102583800$$

$$4515{,}6250c^2 + 1367784c - 455928c - 102583800 = 0$$

$$4515{,}6250c^2 + 911856c - 102583800 = 0$$

$$c = 80{,}4494 \text{ mm}$$

$$a = 0{,}85.c = 0{,}85.\ 80{,}4494 = 68{,}3820 \text{ mm}$$

$$fs' = \frac{c-d'}{c}Es.\varepsilon c \quad \text{.......................} \quad (3.3.36)$$

$$= \frac{80{,}4494 - 75}{80{,}4494}200000.0{,}003 = 40{,}6419 \text{ Mpa}$$

$$M_1 = 0{,}85.fc'.a.b.\left(d - \frac{a}{2}\right) \quad \text{.......................} \quad (3.3.43)$$

$$= 0{,}85.25.68{,}3820\ .250.\left(425 - \frac{68{,}3820}{2}\right)$$

$$= 1419727591{,}9811 \text{ Nmm}$$

$M_2 = As'.fs'.(d-d') = 2279{,}64.40{,}6419.(425-75)$........ (3.3.44)

$= 32427106{,}3764$ Nmm

$M_{tersedia}{}^{+} = M_1 + M_2$ .... (3.3.45)

$= 1419727591{,}9811 + 32427106{,}3764$

$= 174399866{,}3575$ Nmm

$= 174{,}3999$ kNm $> Mu = 150{,}4356$ kNm

2. Mencari $M_{kapasitas}{}^{+}$:

$$a_k = \frac{As.fy.\phi - As'.fy}{0{,}85.f'c.b} \quad \text{...} \quad (3.3.27)$$

$$= \frac{(1139{,}82.400.1{,}25)-(2279{,}64.400)}{0{,}85.25.250} = -64{,}3663 \text{ mm}$$

$$a = \frac{\beta_1.d'.Es.\varepsilon c}{Es.\varepsilon c - fy} \quad \text{...} \quad (3.3.28)$$

$$= \frac{0{,}85.75.200000.0{,}003}{(200000.0{,}003)-400} = 191{,}25 \text{ mm}$$

Karena $a_k < a$, maka tulangan desak belum luluh.

$$Cc = 0{,}85.fc'.0{,}85c.b \quad \text{...} \quad (3.3.38)$$

$$= 0{,}85.25.0{,}85c.250 = 4515{,}6250c$$

$$Cs = As'.\frac{c-d'}{c}Es.\varepsilon_c \quad \text{...} \quad (3.3.37)$$

$$= = 1367784 - \frac{102583800}{c}$$

$$Ts = As.fy.\phi = 1139{,}82.400.1{,}25 = 569910 \text{ N} \text{.......................} (3.3.39)$$

Keseimbangan gaya-gaya dalam

Ts = Cc + Cs .... (3.3.26)

$$569910 = 4515{,}6250c + 1367784 - \frac{102583800}{c}$$

$$569910c = 4515{,}6250c^2 + 1367784c - 102583800$$

$$4515{,}6250c^2 + 1367784c - 569910c - 102583800 = 0$$

$$4515{,}6250c^2 + 797874c - 102583800 = 0$$

c = 86,3610 mm

a = 0,85.c = 0,85. 86,3610 = 73,4069 mm

$$fs' = \frac{c-d'}{c} Es.\varepsilon c \quad \text{...} \quad (3.3.36)$$

$$= \frac{86{,}3610 - 75}{86{,}3610} 200000.0{,}003 = 78{,}9317 \text{ Mpa}$$

$$M_1 = 0{,}85.fc'.a.b.\left(d - \frac{a}{2}\right) \quad \text{...} \quad (3.3.43)$$

$$= 0{,}85.25.73{,}4069\ .250.\left(425 - \frac{73{,}4069}{2}\right)$$

= 151425594,8129 Nmm

$$M_2 = As'.fs'.(d-d') = 2279{,}64.78{,}9317\ .(425-75) \quad \text{....} \quad (3.3.44)$$

= 62977569,8489 Nmm

$M_{kapasitas}{}^{+}$ = $M_1 + M_2$ .... (3.3.45)

= 151425594,8129 + 62977569,8489

= 214403164,6619 Nmm

= 214,4032 kNm > Mu = 150,4356 kNm

Dari hasil perhitungan didapatkan Mkap (+) sebesar 214,4032 kNm dan Mkap (-) sebesar 401,5414 kNm, sedangkan hasil perhitungan pada portal yang lain dapat dilihat pada lampiran 1 tabel 1.2.2.1 dan tabel 1.2.2.2

2. Balok Lapangan

Dari output SAP2000 teredistribusi (gambar 5.4.2) didapat :

M_{perlu} = 97,1167 kNm

$M_n = \dfrac{M_{perlu}}{\phi}$

$= \dfrac{97,1167}{0,8}$

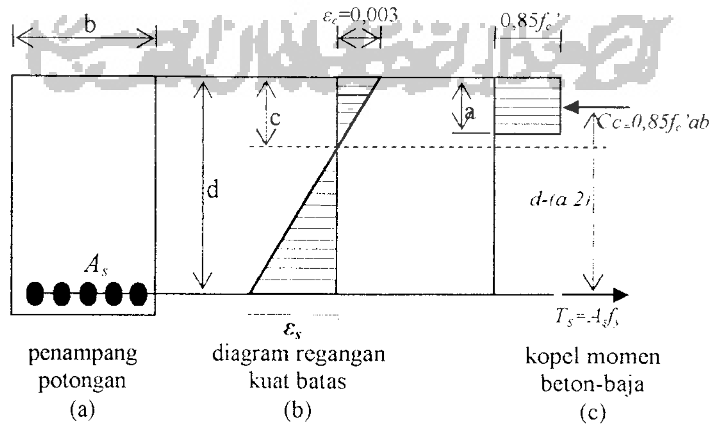
= 121,3958 kNm

Data-data :

b = 250 mm h = 500 mm

d = 425 mm L = 7000 mm

fy = 400 MPa fc' = 25 MPa



Gambar 5.4.5 Diagram Regangan Balok Bertulangan Sebelah

Kopel Beton Desak - Baja Tarik :

$$M = 0{,}85.fc'.a.b.\left(d - \frac{a}{2}\right) \quad \text{...} \quad (3.3.7)$$

$$121{,}3958.10^6 = 0{,}85.25.a.250.\left(425 - \frac{a}{2}\right)$$

$$121{,}3958.10^6 = 2257812{,}5a - 2656{,}25a^2$$

$$2656{,}25a^2 - 2257812{,}5a + 121{,}3958.10^6 = 0$$

$$a = 57{,}6813 \text{ mm}$$

$$Cc = 0{,}85.fc'.a.b \quad \text{...} \quad (3.3.5)$$

$$= 0{,}85 \,.\, 25 \,.\, 57{,}6813 \,.\, 250$$

$$= 306431{,}7979 \text{ N}$$

$$Cc = Ts = As \,.\, fy \quad \text{...} \quad (3.3.6)$$

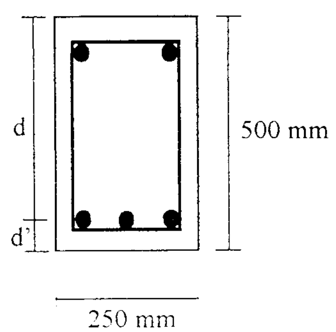
$$306431{,}7979 = As \,.\, 400$$

$$As = \frac{306431{,}7979}{400} = 766{,}0795 \text{ mm}^2$$

Dipakai D22, AØ22 = 379,94 mm^2

$$n = \frac{766{,}0795}{379{,}94} = 2{,}0163 \approx 3 \text{ tulangan}$$

Dari momen kopel beton – baja didapatkan tulangan lapangan seperti gambar di bawah ini :



Gambar 5.4.6 Penampang balok lapangan

1. Mencari $M_{tersedia}$ - :

$$Cc = 0{,}85.fc'.0{,}85c.b \quad \text{...} \quad (3.3.38)$$

$$= 0{,}85.25.0{,}85c.250 = 4515{,}6250c$$

$$Cs = As'.\frac{c-d'}{c}Es.\varepsilon_c \quad \text{...} \quad (3.3.37)$$

$$= 759{,}88.\frac{c-75}{c}200000.0{,}003 = 455928 - \frac{34194600}{c}$$

$$Ts = As.fy = 1139{,}82.400 = 455928 \quad \text{..............................} \quad (3.3.39)$$

Keseimbangan gaya-gaya dalam

$$Ts = Cc + Cs \quad \text{...} \quad (3.3.26)$$

$$455928 = 4515{,}6250c + 455928 - \frac{34194600}{c}$$

$$455928c = 4515{,}6250c^2 + 455928c - 34194600$$

$$4515{,}6250c^2 + 455928c - 455928c - 34194600 = 0$$

$$4515{,}6250c^2 - 34194600 = 0$$

$$c = 87{,}0202 \text{ mm}$$

$$a = 0{,}85.c = 0{,}85.\ 87{,}0202 = 73{,}9671 \text{ mm}$$

$$fs' = \frac{c-d'}{c}Es.\varepsilon c \quad \text{...} \quad (3.3.36)$$

$$= \frac{87{,}0202 - 75}{87{,}0202}200000.0{,}003 = 82{,}8784 \text{ Mpa}$$

$$M_1 = 0{,}85.fc'.a.b\left(d - \frac{a}{2}\right) \quad \text{...} \quad (3.3.43)$$

$$= 0{,}85.25.73{,}9671.250\left(425 - \frac{73{,}9671}{2}\right)$$

$$= 152471200{,}8125 \text{ Nmm}$$

$M_2 = As'.fs'.(d - d')$ .... (3.3.44)

$= 759{,}88 . 82{,}8784 .(425 - 75) = 22042171{,}6838$ Nmm

$M_{tersedia^-} = M_1 + M_2$ .... (3.3.45)

$= 152471200{,}8125 + 22042171{,}6838$

$= 174513372{,}4963$ Nmm

$= 174{,}5134$ kNm $>$ Mu $= 97{,}1167$ kNm

2. Mencari $M_{kapasitas^-}$:

$Cc = 0{,}85.fc'.0{,}85c.b$ .... (3.3.38)

$= 0{,}85.25.0{,}85c.250 = 4515{,}6250c$

$Cs = As'.\dfrac{c - d'}{c} Es.\varepsilon_c$ .... (3.3.37)

$= 759{,}88 . \dfrac{c - 75}{c} 200000.0{,}003 = 455928 - \dfrac{34194600}{c}$

$Ts = As.fy.\phi = 1139{,}82 .400.1{,}25 = 569910$ .... (3.3.39)

Keseimbangan gaya-gaya dalam

Ts = Cc + Cs .... (3.3.26)

$569910 = 4515{,}6250c + 455928 - \dfrac{34194600}{c}$

$569910c = 4515{,}6250c^2 + 455928c - 34194600$

$4515{,}6250c^2 + 455928c - 569910c - 34194600 = 0$

$4515{,}6250c^2 - 113982c - 34194600 = 0$

c = 100,5515 mm

a = 0,85.c = 0,85. 100,5515 = 85,4687 mm

$$fs' = \frac{c-d'}{c} Es.\varepsilon c \quad \text{...} \quad (3.3.36)$$

$$= \frac{100{,}5515-75}{100{,}5515} 200000.0{,}003 = 152{,}4679 \text{ Mpa}$$

$$M_1 = 0{,}85.fc'.a.b.\left(d - \frac{a}{2}\right) \quad \text{...} \quad (3.3.43)$$

$$= 0{,}85.25.85{,}4687.250.\left(425 - \frac{85{,}4687}{2}\right)$$

$$= 173568728{,}2607 \text{ Nmm}$$

$$M_2 = As'.fs'.(d-d') = 759{,}88.152{,}4679.(425-75) \text{ ....} \quad (3.3.44)$$

$$= 40550067{,}3777 \text{ Nmm}$$

$$M_{kapasitas} - = M_1 + M_2 \quad \text{...} \quad (3.3.45)$$

$$= 173568728{,}2607 + 40550067{,}3777$$

$$= 214118795{,}9984 \text{ Nmm}$$

$$= 214{,}1188 \text{ kNm} > Mu = 97{,}1167 \text{ kNm}$$

5.3.1.2 Desain Tulangan Geser Balok

Diambil contoh perhitungan pada lantai 3 portal E bentang 7 m R/W 3 – 3 lama.

Dari output SAP didapatkan :

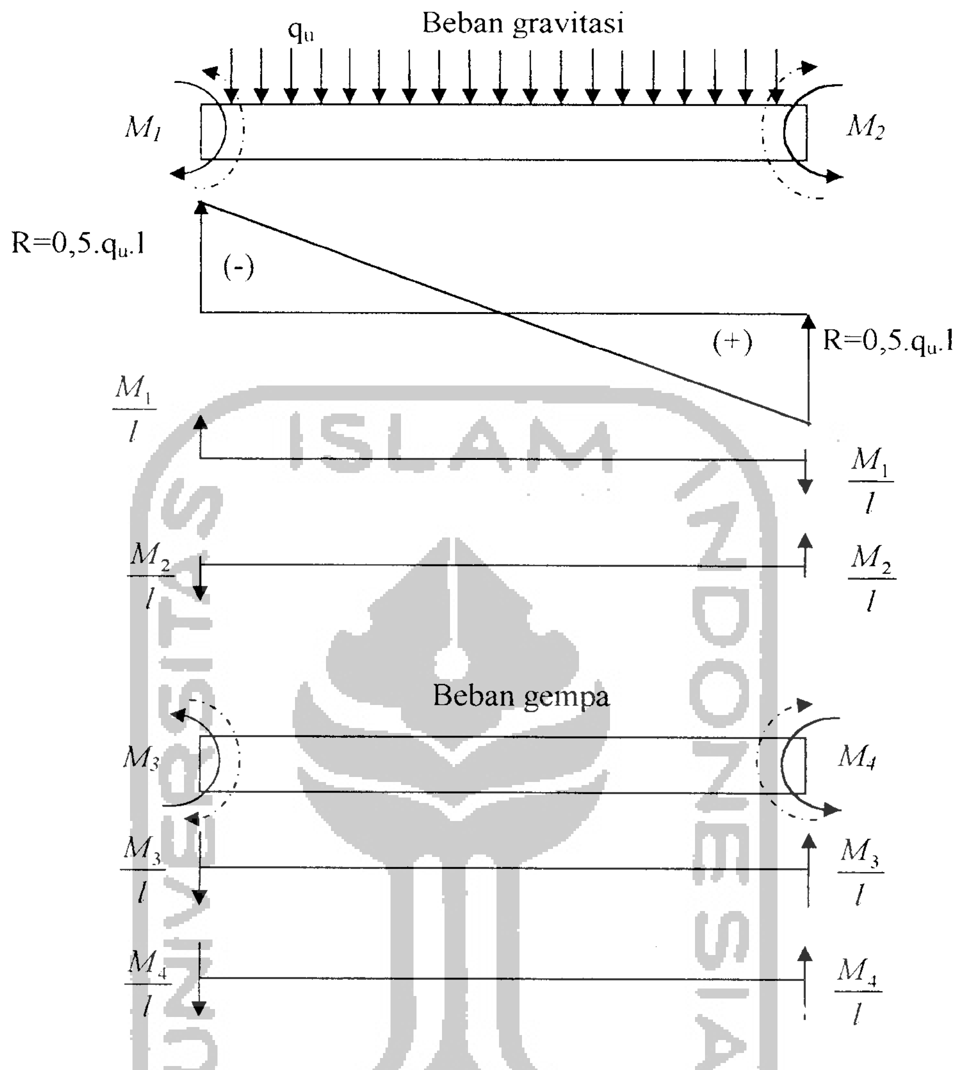
$V_D = 56{,}0157$ kN $\quad V_g = V_D + V_L = 71{,}1793$ kN

$V_L = 15{,}1636$ kN $\quad V_E = 60{,}3020$ kN

Dari hasil perhitungan momen kapasitas balok didapat :

$M_{kap-,b} = 401{,}5414$ kNm $\quad M_{kap+,b} = 214{,}4032$ kNm

$l_n = 6{,}1$ m $\quad d = 0{,}425$ m $\quad K = 1$



Gambar 5.5.1 Reaksi Balok

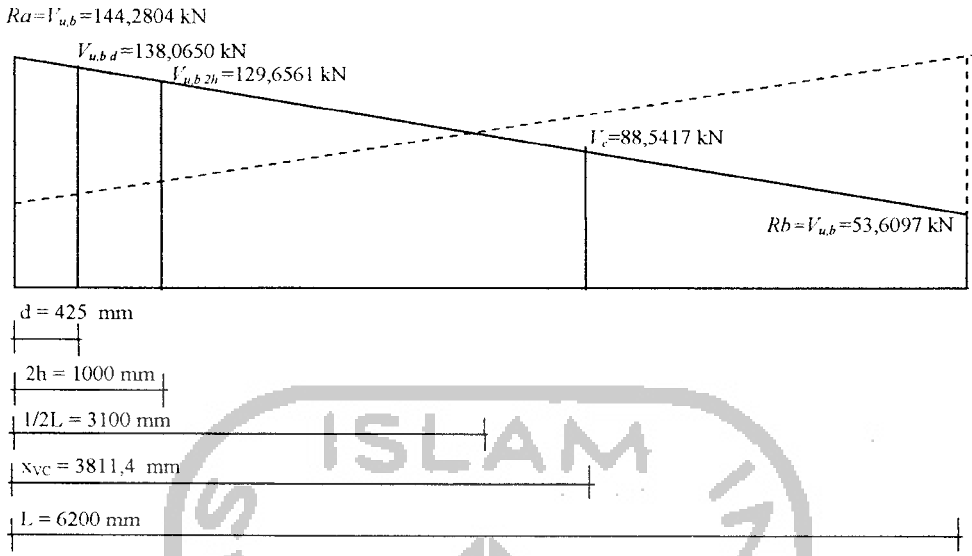
$$V_{u,b} = 0{,}7.\left(\frac{401{,}5414 + 214{,}4032}{6{,}1}\right) + 1{,}05(71{,}1793) \quad \ldots\ldots\ldots\ldots \quad (3.3.30)$$

$$= 144{,}2804 \text{ kN}$$

hasil tersebut tidak boleh melebihi

$$V_{u,b} = 1{,}05\left(56{,}0157 + 15{,}1636 + \frac{4}{1}.60{,}3020\right) \quad \ldots\ldots\ldots\ldots \quad (3.3.31)$$

$$= 328{,}0067 \text{ kN}$$



Gambar 5.5.2 Gaya Geser pada Penampang Kritis, Daerah Sendi Plastis, dan Luar Sendi Plastis

1). *Di daerah sendi plastis*

$V_{u,b\ pakai}$ = 138,0650 kN

$$Vs = \frac{V_u}{\phi} = \frac{138{,}0650}{0{,}6}$$

$$= 230{,}1084 \text{ kN},$$

dengan syarat spasi tidak boleh melebihi

* $\frac{d}{4} = \frac{425}{4} = 106{,}25$ mm

* $8.\varnothing_{pokok} = 8.22 = 176$ mm

* $24.\varnothing_{sengkang} = 24.10 = 240$ mm

* $$\frac{1600.f_{y\,sengkang}.As_{sengkang}}{As_{pokok}.f_{ypokok}} = \frac{1600.240.78{,}5}{379{,}94.400} = 198{,}3471 \text{ mm}$$

$$S = \frac{Av.fy.d}{Vs} \quad \text{...................................} \quad (3.3.36)$$

$$= \frac{\left(2.1/4.\pi.10^2\right)240.425}{230{,}1084.10^3} = 69{,}5933 \text{ mm} < 106{,}25 \text{ mm}$$

dipakai tulangan sengkang $\varnothing_{10}$ - $_{65}$

2). *Di luar sendi plastis*

Dengan perbandingan segitiga didapat $V_{u,b\ pakai}$ luar sendi plastis didapat :

$V_{u,b\ pakai}$ = 129,6561 kN

$$V_c = \frac{1}{6}\sqrt{fc}.b.d \quad \text{...................................} \quad (3.3.33)$$

$$= \frac{1}{6}\sqrt{25}.300.425 = 88{,}5417 \text{ mm}$$

$$\frac{V_{u,b} - V_{u.bpakai}}{d} = \frac{V_{u,b} - V_c}{x_{Vc}}$$

$$\frac{144{,}2804 - 138{,}0650}{425.10^{-3}} = \frac{144{,}2804 - 88{,}5417}{x_{Vc}}$$

$$x_{Vc} = \frac{55{,}7387.425.10^{-3}}{6{,}2154}$$

$= 3{,}8114$ m

$= 3811{,}4$ mm

Panjang daerah di luar sendi plastis = $x_{Vc} - 2h$

= 3811,4 - 2.500

= 3811,4 - 1000

= 2811,4 mm

$$V_s = \frac{V_{u,bpakai}}{\phi} - V_C \quad \text{.......} \quad (3.3.34)$$

$$= \frac{129{,}6561}{0{,}6} - 88{,}5417$$

$$= 127{,}5518 \text{ mm},$$

dengan syarat spasi tidak melebihi

* $\frac{1}{2}d = \frac{1}{2}.425 = 212{,}5$ mm

* 600 mm

$$S = \frac{Av.fy.d}{Vs} \quad \text{.......} \quad (3.3.36)$$

$$= \frac{(2.1/4.\pi.10^2)240.425}{127{,}5518.10^3}$$

$$= 125{,}5490 \text{ mm}$$

dipakai sengkang $\varnothing_{10-120}$

5.3.2 Desain Kolom

Sebagai contoh perhitungan diambil pada kolom A lantai 3 R/W 3-3 lama.

5.3.2.1 Desain Tulangan Lentur Kolom

a. Momen Rencana Kolom

Dari data perencanaan didapat:

ω_d = 1,3	h = 4 m	h_n = 3,5 m
$L_{x.ki}$ = 7 m	$L_{x.ka}$ = 4 m	

Dari hasil perhitungan balok bentang 7 m didapat :

M_{kap-} = 401,5414 kNm

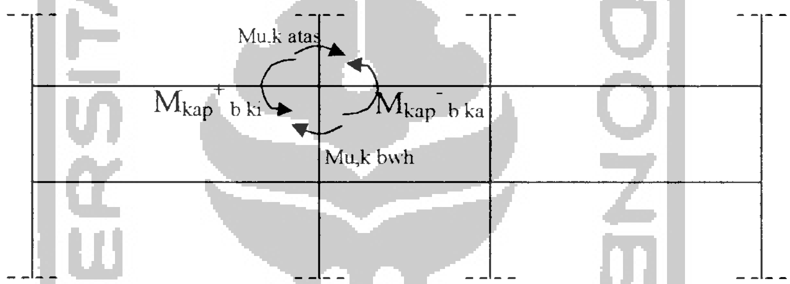
M_{kap+} = 214,4032 kNm

Sedangkan momen kapasitas balok bentang 4 m didapat :

M_{kap--} = 470,4221 kNm

M_{kap+} = 346,7628 kNm

Untuk hasil perhitungan M_{kap} dapat dilihat pada lampiran 1 tabel 1.2.2.1.



Gambar 5.6.1 Keseimbangan Momen Kolom

Persamaan yang digunakan untuk mencari α_k :

$$\alpha_{k\ atas} = \frac{M_{E,kti\_atas}}{M_{E,klti\_atas} + M_{E,klti+1\_bawah}} \quad \ldots\ldots\ldots\ldots \quad (3.4.2)$$

$$\alpha_{k\ bwh} = \frac{M_{E,kti\_bawah}}{M_{E,klti\_bawah} + M_{E,klti-1\_atas}} \quad \ldots\ldots\ldots\ldots \quad (3.4.3)$$

Dari SAP didapat :

Tabel 5.5 Momen Output SAP.

KOLOM	Arah X			Arah Y		
	MD,kx Atas/Bawah	ML,kx Atas/Bawah	ME,kx Atas/Bawah	MD,ky Atas/Bawah	ML,ky Atas/Bawah	ME,ky Atas/Bawah
	kNm	kNm	kNm	kNm	kNm	kNm
1	2	3	4	2	3	4
A						
Lantai 1	30,0241	4,2996	69,3864	0,0000	0,0000	39,2471
	14,0404	2,0106	569,9592	0,0000	0,0000	584,7308
Lantai 2	36,8359	5,1896	152,3174	0,0000	0,0000	170,4383
	40,6842	5,7862	416,7358	0,0000	0,0000	392,9021
Lantai 3	35,0267	4,8474	243,9489	0,0000	0,0000	250,3221
	34,1903	4,7507	333,3810	0,0000	0,0000	309,2948
Lantai 4	35,2702	4,8127	278,8766	0,0000	0,0000	275,4489
	35,5318	4,8765	283,9192	0,0000	0,0000	262,8273
Lantai 5	35,0483	4,7182	286,5348	0,0000	0,0000	278,7853
	35,0442	4,7387	245,3780	0,0000	0,0000	230,5533
Lantai 6	34,9623	4,6510	280,0516	0,0000	0,0000	270,7066
	35,0100	4,6772	209,6435	0,0000	0,0000	199,8778
Lantai 7	34,8384	4,5922	264,8541	0,0000	0,0000	255,7504
	34,8820	4,6112	172,9353	0,0000	0,0000	167,8125
Lantai 8	34,8505	4,5372	243,1180	0,0000	0,0000	235,3800
	34,8237	4,5576	134,0156	0,0000	0,0000	132,9274
Lantai 9	34,3966	4,5260	215,1428	0,0000	0,0000	209,6290
	34,6508	4,5224	92,9683	0,0000	0,0000	95,2303
Lantai 10	35,9860	4,3820	180,5683	0,0000	0,0000	178,0170
	35,0210	4,4603	51,6003	0,0000	0,0000	56,1432
Lantai 11	29,0270	4,8612	134,0571	0,0000	0,0000	134,6775
	33,1830	4,5694	13,6302	0,0000	0,0000	18,6808
Lantai 12	58,2805	2,7301	78,8969	0,0000	0,0000	82,2366
	40,7636	4,0010	7,4542	0,0000	0,0000	3,7237

Untuk lantai 3 :

$$\alpha_{k,x\ atas} = \frac{243,9489}{243,9489 + 283,9192} = 0,46 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.4.2)$$

$$\alpha_{k,x\ bwh} = \frac{333,3810}{333,3810 + 152,3174} = 0,69 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.4.3)$$

$$M_{u,k\ atas} = \frac{h_n}{h}.0,7.\omega_d.\alpha_{k.atas}\left\{\frac{l}{\ln}M^{+}_{kap\ .ki} + \frac{l}{\ln}M^{-}_{kap.ka}\right\} \ldots\ldots\ldots\ldots \quad (3.4.1)$$

$$= \frac{3,5}{4}.0,7.1,3.0,46\left\{\frac{7}{6,1}.214,4032 + \frac{4}{3,1}.470,4221\right\}$$

$$= 313,8980 \text{ kNm}$$

$$M_{u,kbawah} = \frac{h_n}{h}.0,7.\omega_d.\alpha_{k,bawah}\left\{\frac{l}{\ln}M^{+}{}_{kap\ .ki} + \frac{l}{\ln}M^{-}{}_{kap,ka}\right\}\text{.........} \quad (3.4.1)$$

$$= \frac{3,5}{4}.0,7.1,3.0,69\left\{\frac{7}{6,1}.214,4032 + \frac{4}{3,1}.470,4221\right\}$$

$$= 466,2183 \text{ kNm}$$

b. Momen Maksimum Kolom

$$M_{U,k} = 1,05\left[M_{L,k} + M_{D,k} + \frac{4}{K}\left(M_{E,k}\right)\right]\text{..} \quad (3.4.4)$$

$$M_{u,kx\ atas} = 1,05\left[4,8474 + 35,0267 + \frac{4}{1}243,9489\right] = 1066,4532 \text{ kNm}$$

$$M_{u,kx\ bwh} = 1,05\left[4,7507 + 34,1903 + \frac{4}{1}333,3810\right] = 1441,0881 \text{ kNm}$$

c. Momen Kolom Terpakai

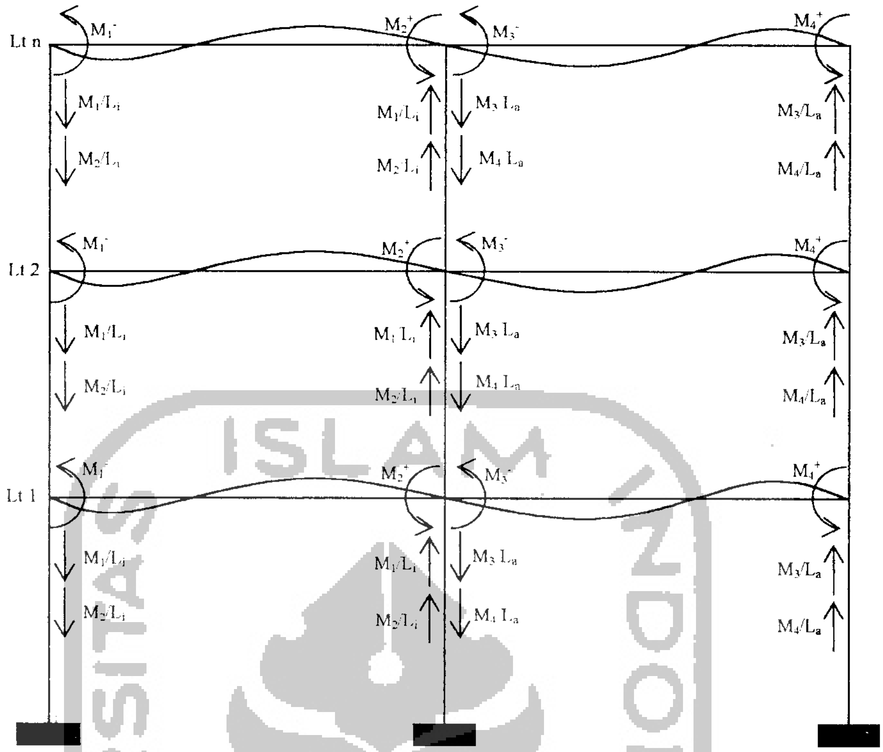
Momen kolom terpakai diambil nilai terkecil dari perhitungan pada momen rencana kolom dan momen maksimum kolom.

$M_{u,kx\ atas}$ = 313,8980 kNm $\qquad$ $M_{u,kx\ bwh}$ = 466,2183 kNm

d. Gaya Aksial Rencana Kolom

Dari output SAP lampiran 1 tabel 1.3.5 didapat :

$N_{L,kx}$ = 234,8772 kN	$N_{D,kx}$ = 699,1574 kN
$N_{L,ky}$ = 274,6505 kN	$N_{D,ky}$ = 1022,7940 kN
$N_{g,kx}$ = 934,0346 kN	$N_{g,ky}$ = 1297,4445 kN
$N_{E,kx}$ = 560,2989 kN	$N_{E,ky}$ = 0,0299 kN



Gambar 5.6.2 Gaya Aksial Kolom

Perhitungan kumulatif momen kapasitas pada lantai 3 dihitung dengan menjumlahkan momen kapasitas dari lantai 3 sampai dengan lantai 12.

$$\sum M_{kap} = M_{kapL12} + M_{kapL11} + M_{kaL10} + M_{kaL9} + M_{kapL8} + M_{kapL7} + M_{kapL6} + M_{kapL5} + M_{kapL4} + M_{kapL3} \quad \text{.............} \quad (3.4.6)$$

1. Untuk portal E (arah x):

$$\sum M_1^- = 147{,}7236 + 147{,}7007 + 147{,}7007 + 214{,}4099 + 214{,}4099 + 214{,}4099 + 214{,}4099 + 214{,}4032 + 214{,}4032 + 214{,}4032 + 214{,}4032$$

$$= 3403{,}0554 \text{ kNm}$$

dengan cara yang sama didapat :

$\sum M_2^+ = 1943{,}9674$ kN $\quad \sum M_3^- = 3743{,}0940$ kN

$\sum M_4^+ = 2672{,}3979$ kN

$$\sum M_{kap\,kiri} = \sum M_1^- + \sum M_2^+$$

$$= 3403{,}0554 \text{ kNm} + 1943{,}9674 \text{ kN}$$

$$= 5347{,}0228 \text{ kN}$$

$$\sum M_{kap\,ka} = \sum M_3^- + \sum M_4^+$$

$$= 3743{,}0940 \text{ kNm} + 2672{,}3979 \text{ kN}$$

$$= 6415{,}4919 \text{ kN}$$

n = Jumlah lantai diatas kolom yang ditinjau = 10

R_v = 1,1 – 0,025.n = 1,1 – 0,025.10 = 0,85

$$N_{u,k} = Rv.0{,}7\sum_{i=1}^{n}\left\{\frac{\sum M_{kap,ki}}{L_{ki}} + \frac{\sum M_{kap,ka}}{L_{ka}}\right\} + 1{,}05N_{g,k} \quad \ldots\ldots\ldots\ldots \quad (3.4.5)$$

$$N_{u,kx-atas} = Rv.0{,}7.\sum_{i=3}^{10}\left\{\frac{\sum M_{kap.x\,ki\,atas}}{L_{ki}} + \frac{\sum M_{kap.x\,ka\,atas}}{L_{ka}}\right\} + 1{,}05N_{g,kx}$$

$$= 0{,}85.0{,}7.\left\{\frac{5347{,}0228}{7} + \frac{6415{,}4919}{4}\right\} + 1{,}05.934{,}0346$$

$$= 2389{,}5377 \text{ kN}$$

$N_{u,kx-bwh}$ = 2478,8686 kN

Dari kedua nilai di atas ($N_{u,kx-atas}$ dan $N_{u,kx-bwh}$), diambil nilai terbesar sehingga $N_{u,kx}$ = 2478,8686 kN

2. Untuk portal 2 (arah y):

$\sum M_1^- \;=\; 214{,}1188 + 214{,}1188 + 174{,}5134 + 401{,}5414 + 401{,}5414 + 329{,}9356 + 470{,}4221 + 470{,}4221 + 470{,}4221 + 470{,}4221$

$= \; 3617{,}4578$ kNm

dengan cara yang sama didapat :

$\sum M_2^+ = 2473{,}4314$ kN $\quad \sum M_3^- = 3617{,}4578$ kN

$\sum M_4^+ = 2473{,}4314$ kN

$\sum M_{kap\,kiri} = \sum M_1^- + \sum M_2^+$

$= 3617{,}4578$ kNm + $2473{,}4314$ kN

$= 6090{,}8892$ kN

$\sum M_{kap\,ka} = \sum M_3^- + \sum M_4^+$

$= 3617{,}4578$ kNm + $2473{,}4314$ kN

$= 6090{,}8892$ kN

n = Jumlah lantai diatas kolom yang ditinjau = 10

R_v = 1,1 – 0,025.n = 1,1 – 0,025.10 = 0,85

$$N_{u.k} = Rv.0{,}7\sum_{i=1}^{n}\left\{\frac{\sum M_{kap.ki}}{L_{ki}} + \frac{\sum M_{kap,ka}}{L_{ka}}\right\} + 1{,}05N_{g.k} \quad \ldots\ldots\ldots \quad (3.4.5)$$

$$N_{u.ky-atas} = Rv.0{,}7.\sum_{i=3}^{10}\left\{\frac{\sum M_{kap.y\,ki\,atas}}{L_{ki}} + \frac{\sum M_{kap.y\,ka\,atas}}{L_{ka}}\right\} + 1{,}05N_{g.kx}$$

$$= 0{,}85.0{,}7.\left\{\frac{6090{,}8892}{5} + \frac{6090{,}8892}{5}\right\} + 1{,}05.1297{,}4445$$

$= 2811{,}9484$ kN

$N_{u,ky\text{-}bwh}$ = 2920,3199 kN

Dari kedua nilai di atas ($N_{u,ky\text{-}atas}$ dan $N_{u,ky\text{-}bwh}$), diambil nilai terbesar sehingga $N_{u,ky}$ = 2920,3199 kN

Dari gaya aksial rencana masing-masing arah, diambil nilai terbesar dari keduanya, yaitu $N_{u,k}$ = 2920,3199 kN.

e. Gaya Aksial Maksimum Kolom

$$N_{u,k} = 1{,}05\left(N_{g,k} + \frac{4}{K} N_{E,k}\right) \quad \text{.......................................} \quad (3.4.7)$$

$$Nu,_{kx} = 1{,}05\left(934{,}0346 + \frac{4}{1} 560{,}2989\right) = 3333{,}9917 \text{ kN}$$

$$Nu,_{ky} = 1{,}05\left(1297{,}4445 + \frac{4}{1} 0{,}0299\right) = 1362{,}4423 \text{ kN}$$

Dari gaya aksial maksimum masing-masing arah, diambil nilai terbesar dari keduanya, yaitu $N_{u,k}$ = 3333,9917 kN.

Gaya aksial terpakai merupakan nilai terkecil dari gaya aksial rencana dengan gaya aksial maksimum, yaitu $N_{u,k}$ = 2920,3199 kN.

f. Penulangan Kolom

Contoh perhitungan diambil pada kolom A lantai 3

Diketahui dari data SAP :

Pu = 2920,3199 kN

$$\text{Pn} = \frac{2920{,}3199}{0{,}65}$$

= 4492,7998kN

Mu = 466,2183 kNm

$$\text{Mn} = \frac{466{,}2183}{0{,}65}$$

= 717,2589 kNm

fc' = 25 MPa

fy = 400 MPa

b = 700 mm

h = 900 mm

$$e = \frac{Mn}{Pn} = \frac{717,2589}{4492,7998} = 0,1596 \text{ m} = 159,6 \text{ mm} \qquad \text{.............} \quad (3.4.15)$$

d' = 40 + 10 + 11 = 61 mm

d = h – d' = 900 – 61 = 839 mm

Perkiraan penulangan bruto = **3 %**

$$\rho = \rho' = \frac{As}{b.d} = 0,03 \qquad \text{..............................} \quad (3.4.8)$$

As = As' = 0,03 . 700 . 839 = 17619 mm^2

Digunakan tulangan baja Ø22, A = 379,94 mm^2

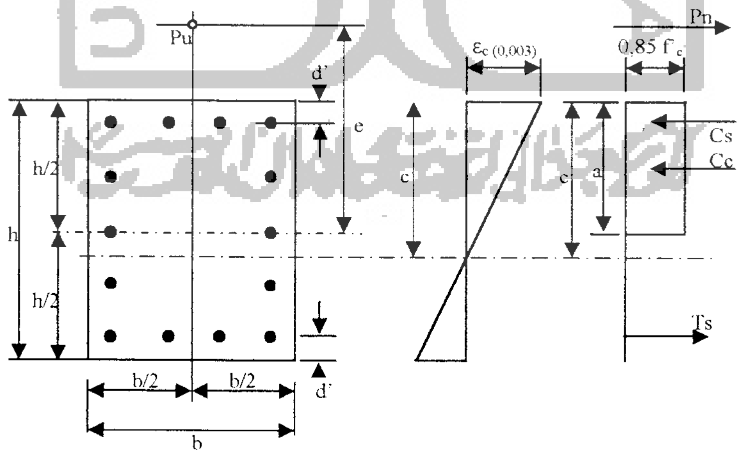
$$n = \frac{17619}{379,94} = 46,37 \approx 48 \text{ buah} \qquad \text{..............................} \quad (3.4.9)$$

Dicoba 24D22 pada masing-masing sisi penampang

As = 24 . 379,94 = 9118,56 mm^2

$$\rho = \frac{9118,56}{700.839} = 0,0155 \qquad \text{..............................} \quad (3.4.8)$$

Pemeriksaan Pu terhadap beban seimbang Pub



Gambar 5.6.3 Diagram Gaya dalam Kolom

$$c_b = \frac{600.839}{600 + 400} = 503,4 \text{ mm} \qquad \text{..............................} \quad (3.4.10)$$

$\beta_1 = 0{,}85$

$a_b = \beta_1 . c_b$ .... (3.4.11)

$= 0{,}85 \;.\; 503{,}4 = 427{,}89$ mm

$\varepsilon_{s'} = \dfrac{c_b - d'}{c_b} \varepsilon_c$ .... (3.4.12)

$= \dfrac{503{,}4 - 61}{503{,}4} .0{,}003 = 0{,}0026 > \dfrac{fy}{Es} = \dfrac{400}{200000} = 0{,}0020$

Tulangan desak telah luluh, sehingga :

fs' = fy

$ØPnb = Ø.0{,}85.fc'.a_b.b + As'.fs' - As.fs$ .... (3.4.13)

$= 0{,}65 \;.\; 0{,}85 \;.\; 25 .\; 427{,}89 \;.\; 700 + 9118{,}56.400 - 9118{,}56.400$

$= 4137161{,}438$ N

$= 4137{,}1614$ kN > Pu = 2920,3199 kN

dengan demikian kolom akan mengalami hancur dengan diawali luluhnya tulangan tarik.

Pemeriksaan kekuatan penampang

$\rho = 0{,}0155$

$m = \dfrac{fy}{0{,}85.fc'} = \dfrac{400}{0{,}85.25} = 18{,}8235$ .... (3.4.16)

$\dfrac{h - 2e}{2.d} = \dfrac{900 - 2.159{,}6}{2.839} = 0{,}3461$

$\left(1 - \dfrac{d'}{d}\right) = \left(1 - \dfrac{61}{839}\right) = 0{,}9273$

$Pn = 0{,}85.fc'.b.d\left[\left(\dfrac{h-2e}{2d}\right) + \sqrt{\left(\dfrac{h-2e}{2d}\right)^2 + 2.m.\rho\left(1 - \dfrac{d'}{d}\right)}\right]$ .... (3.4.14)

$$Pn = 0{,}85.25.700.839\,.\left[0{,}3461 + \sqrt{0{,}3461^2 + 2.18{,}8235.0{,}0155.0{,}9273}\right]$$

$= 14465115{,}636 \text{ N} = 14465{,}1156 \text{ kN}$

ØPn = 0,65. 14465,1156 = 9402,3251 kN .... (3.4.17)

0,1.Ag.fc' = 0,1.900.700.25 = 1575 kN

ØPn > 0,1.Ag.fc'

9402,3251 kN > 1575 kN

$$a = \frac{Pn}{0{,}85.fc'.b} \qquad \text{...} \quad (3.4.18)$$

$$= \frac{14465{,}1156.10^3}{0{,}85.25.700} = 972{,}4447 \text{ mm}$$

$$c = \frac{a}{\beta 1} = \frac{972{,}4447}{0{,}85} = 1144{,}0526 \text{ mm} \text{....................................} \quad (3.4.19)$$

$$fs' = \varepsilon_c Es\left(\frac{c-d'}{c}\right) = 0{,}003.200000\left(\frac{1144{,}0526 - 61}{1144{,}0526}\right) \text{..........} \quad (3.4.20)$$

$$= 568{,}0085 \text{ MPa} > fy = 400 \text{ MPa}$$

Tulangan tekan telah luluh, sesuai anggapan awal.

Pu = 2920,3199 kN < ØPn = 9402,3251 kN, maka perencanaan penampang kolom memenuhi persyaratan. Tetapi karena selisih antara ØPn dan Pn terlalu besar, maka ρ diperkecil.

Perkiraan penulangan bruto = **1 %**

$$\rho = \rho' = \frac{As}{b.d} = 0{,}01 \qquad \text{...} \quad (3.4.8)$$

As = As' = 0,01 . 700 . 839 = 5873 mm^2

Digunakan tulangan baja Ø22, A = 379,94 mm^2

$$n = \frac{5873}{379{,}94} = 15{,}46 \approx 16 \text{ buah} \quad \text{.......................................} \quad (3.4.9)$$

Dicoba 8D22 pada masing-masing sisi penampang

As = 8 . 379,94 = 3039,52 mm^2

$$\rho = \frac{3039{,}52}{700.839} = 0{,}0052 \quad \text{...} \quad (3.4.8)$$

Pemeriksaan Pu terhadap beban seimbang Pub

$$c_b = \frac{600.839}{600+400} = 503{,}4 \text{ mm} \quad \text{..} \quad (3.4.10)$$

$$\beta_1 = 0{,}85$$

$$a_b = \beta_1 . c_b \; a_b \quad \text{...} \quad (3.4.11)$$

$$= 0{,}85 \;.\; 503{,}4 = 427{,}89 \text{ mm}$$

$$\varepsilon s' = \frac{c_b - d'}{c_b} \varepsilon_c = \frac{503{,}4 - 61}{503{,}4} .0{,}003 \quad \text{..} \quad (3.4.12)$$

$$= 0{,}0026 > \frac{fy}{Es} = \frac{400}{200000} = 0{,}0020$$

fs' = fy

$$ØPnb = Ø.0{,}85.fc'.a_b.b + As'.fs' - As.fs \quad \text{..............................} \quad (3.4.13)$$

$$= 0{,}65 \;.\; 0{,}85 \;.\; 25.\; 427{,}89 \;.\; 700 + 3039{,}52.400 - 3039{,}52.400$$

$$= 4137{,}1614 \text{ kN} > Pu = 2920{,}3199 \text{ kN}$$

dengan demikian kolom akan mengalami hancur dengan diawali

luluhnya tulangan tarik.

Pemeriksaan kekuatan penampang

$\rho = 0{,}0052$

$$m = \frac{fy}{0{,}85.fc'} = \frac{400}{0{,}85.25} = 18{,}8235 \quad \text{.....................................} \quad (3.4.16)$$

$$\frac{h - 2e}{2.d} = \frac{900 - 2.159{,}6{,}0}{2.839} = 0{,}3461$$

$$\left(1 - \frac{d'}{d}\right) = \left(1 - \frac{61}{839}\right) = 0{,}9273$$

$$Pn = 0{,}85.fc'.b.d\left[\left(\frac{h-2e}{2d}\right) + \sqrt{\left(\frac{h-2e}{2d}\right)^2 + 2.m.\rho.\left(1 - \frac{d'}{d}\right)}\right] \text{....} \quad (3.4.14)$$

$$Pn = 0{,}85.25.700.839\left[0{,}3461 + \sqrt{0{,}3461^2 + 2.18{,}8235.0{,}0052.0{,}9273}\right]$$

$= 11170011{,}175$ N

$= 11170{,}0112$ kN

$\varnothing Pn = 0{,}65.\ 11170{,}0112 = 7260{,}5073$ kN .... (3.4.17)

$0{,}1.Ag.fc' = 0{,}1.900.700.25 = 1575$ kN

$\varnothing Pn > 0{,}1.Ag.fc'$

7260,5073 kN > 1575 kN

$$a = \frac{Pn}{0{,}85.fc'.b} \quad \text{...} \quad (3.4.18)$$

$$= \frac{11170{,}0112.10^3}{0{,}85.25.700} = 750{,}9251 \text{ mm}$$

$$c = \frac{a}{\beta 1} = \frac{750{,}9251}{0{,}85} \quad \text{...} \quad (3.4.19)$$

$= 883{,}4413$ mm

$$fs' = \varepsilon_c Es\left(\frac{c-d'}{c}\right) \quad \text{...} \quad (3.4.20)$$

$$= 0{,}003.200000\left(\frac{883{,}4413-61}{883{,}4413}\right) = 558{,}5711 \text{ MPa} > fy = 400 \text{ MPa}$$

Tulangan tekan telah luluh, sesuai anggapan awal.

Pu = 2920,3199 kN < ØPn = 7260,5073 kN, maka perencanaan penampang kolom memenuhi persyaratan. Karena rasio penulangan terhadap beton ρ harus berada dalam daerah batas nilai 0,01 ≤ ρ ≤ 0,08, maka dipakai ρ = 0,01.

Analisis Kolom

Dari perhitungan kolom didapatkan data-data :

fy = 400 MPa	fc' = 25 MPa
b = 700 mm	h = 900 mm
d' = 40 + 10 + 11 = 61 mm	
d = h – d' = 900 – 61 = 839 mm	
ρ = 1 %	Ag = 630000 mm^2

$\rho = \frac{Ast}{b.d}$ → Ast = ρ.b.d = 0,01.700.839 = 5873 mm^2

y = 0,5.h = 0,5.900 = 450 mm^2

As = As' = 8.379,94 = 3039,52 mm^2

- Kondisi Pmaks (Titik A)

Pmaks = 0,8{0,85.fc'(Ag – Ast) + fy.Ast} .... (3.4.21)

= 0,8{0,85.25(630000 – 5873) – 400. 5873}

= 12489519 N = 12489,519 kN

- Kondisi Seimbang (Titik C)

$$c_b = \frac{\varepsilon_s . E_s}{\varepsilon_s . E_s + fy} . d$$

$$= \frac{0{,}003.200000}{0{,}003.200000 + 400} .839 = 503{,}4 \text{ m} \quad \text{(3.4.28)}$$

a = 0,85.c_b = 0,85. 503,4 = 427,89 mm .... (3.4.29)

$$fs' = \frac{c_b - d'}{c_b} Es.\varepsilon_c = \frac{503{,}4 - 61}{503{,}4} .600 \quad \text{(3.4.30)}$$

= 527,2944 MPa > fy = 400 MPa

fs' = fy = 400 MPa

fs = fy = 400 MPa

Cc = 0,85.fc'.b.a .... (3.4.31)

= 0,85.25.700. 427,89

= 6364,8637 kN

Cs = As'(fs' – 0,85.fc') .... (3.4.32)

= 3039,52.(400 – 0,85.25)

= 1151,2182 kN

Ts = As.fs .... (3.4.33)

= 3039,52.400

= 1251,808 kN

$$Pn \;=\; Cc + Cs - Ts \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \;(3.4.34)$$

$$= 6364{,}8637 + 1151{,}2182 - 1251{,}808$$

$$= 6264{,}2739 \text{ kN}$$

$$Mn \;=\; Cc\left(y - \frac{a}{2}\right) + Cs(y - d') + Ts(d - y) \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots \;(3.4.35)$$

$$= 6364{,}8637\left(450 - \frac{427{,}89}{2}\right) + 1151{,}2182\,(450 - 61)$$

$$+1251{,}808\,(839\text{-}450)$$

$$= 2437{,}2351 \text{ kNm}$$

- Kondisi Lentur Murni (Titik E)

$$fs' = \frac{c - d'}{c} Es.\varepsilon_c \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \;(3.4.22)$$

$$Cc + Cs - Ts = 0 \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \;(3.4.23)$$

$$\{(0{,}85.fc').(0{,}85c).b\} + (fs'.As' - 0{,}85.fc'.As') - fy.As = 0$$

$$\{(0{,}85.25).(0{,}85c).700\} + \left\{\left(\frac{600.(c - 61)}{c}\right).3039{,}52 - 0{,}85.25.3039{,}52\right\}$$

$$- 400.\ 3039{,}52 = 0$$

$$12643{,}75c + \frac{1823712c}{c} - \frac{111246432}{c} - 64589{,}8 - 1215808 = 0$$

$$12643{,}75c + 543314{,}2 - \frac{111246432}{c} = 0$$

$$12643{,}75c^2 + 543314{,}2c - 111246432 = 0$$

$$c = 74{,}7442 \text{ mm}$$

$$fs' = 0{,}003.200000.\frac{74{,}7442 - 61}{74{,}7442} = 110{,}3299 \text{ MPa} \ldots\ldots\ldots\ldots (3.4.22)$$

Cc = 0,85.fc'.0,85c.b .... (3.4.24)

= 0,85.25.0,85. 74,7442.700

= 945,047 kN

Cs = fs'.As' = 110,3299. 3039,52 .... (3.4.25)

= 335,3499 kN

Ts = fy.As = 400. 3039,52 .... (3.4.26)

= 1215,808 kN

$$Mn = Cc\left(y - \frac{0,85c}{2}\right) + Cs(y\text{-}d') + Ts(d\text{-}y) \quad \text{.................. (3.4.27)}$$

$$= 945,047\left(450 - \frac{0,85.74,7442}{2}\right) + \{335,3499\ (450\text{-}61)\} +$$

{1215,808 (839-450)}

= 998,6509 kNm

- Kondisi Patah Desak (TitikB)

c > cb .... (3.4.36)

misal: c = 600 mm > cb = 503,4 mm

a = 0,85.600 = 510 mm .... (3.4.37)

$$fs' = \frac{c - d'}{c}.\varepsilon_c.Es = \frac{600 - 61}{600}.600 \quad \text{.................................... (3.4.38)}$$

= 539 MPa > fy = 400 MPa

$$fs = \frac{d - c}{c}.\varepsilon_c.Es = \frac{839 - 600}{600}.0,003.200000 \quad \text{.................. (3.4.39)}$$

= 239 MPa < fy

fs = 239 MPa

Cc = 0,85.fc'.a.b .... (3.4.24)

= 0,85.25.510.700 = 7586,250 kN

$Cs = As'.(fs'-0,85.fc')$ .... (3.4.25)

$= 3039,52.(400-0,85.25)$

$= 1151,2182$ kN

$Ts = As.fs = 3039,52.239 = 726,4453$ kN .... (3.4.26)

$Pn = Cc + Cs - Ts$ .... (3.4.34)

$= 7586,250 + 1151,2182 - 726,4453$

$= 8011,0229$ kN

$$Mn = Cc\left(y - \frac{a}{2}\right) + Cs(y - d') + Ts(d - y) \quad \text{......................... (3.4.35)}$$

$$= 7586,250.\left(450 - \frac{510}{2}\right) + 1151,2182\ (450\text{-}61) +$$

$$726,4453\ (839\text{-}450)$$

$= 2209,7298$ kNm

- Kondisi Patah Tarik (Titik D)

$c < cb$ .... (3.4.40)

Misal $c = 0,5.cb = 0,5.503,4 = 251,7$ mm ≈ 300 mm

$a = 0,85.c = 0,85.300 = 255$ mm .... (3.4.41)

$$fs' = \frac{c - d'}{c}.\varepsilon_c.Es \quad \text{... (3.4.42)}$$

$$= \frac{300 - 61}{300}.600$$

$= 453,6$ MPa $> fy = 400$ MPa

$fs' = fy = 400$ MPa

$$fs = \frac{d-c}{c}.\varepsilon_c.Es \quad \text{...} \quad (3.4.43)$$

$$= \frac{839-300}{300}.0{,}003.200000$$

$= 1078$ MPa $>$ fy $= 400$ MPa

fs = fy = 400 MPa

Cc = 0,85.fc'.a.b = 0,85.25.255.700 .... (3.4.31)

= 3793,125 kN

Cs = As'.(fs'-0,85fc') .... (3.4.32)

= 3039,52.(400-0,85.25)

= 1151,2182 kN

Ts = As.fy .... (3.4.33)

= 3039,52.400

= 1215,808 kN

Pn = Cc + Cs – Ts .... (3.4.34)

= 3793,125 + 1151,2182 – 1215,808

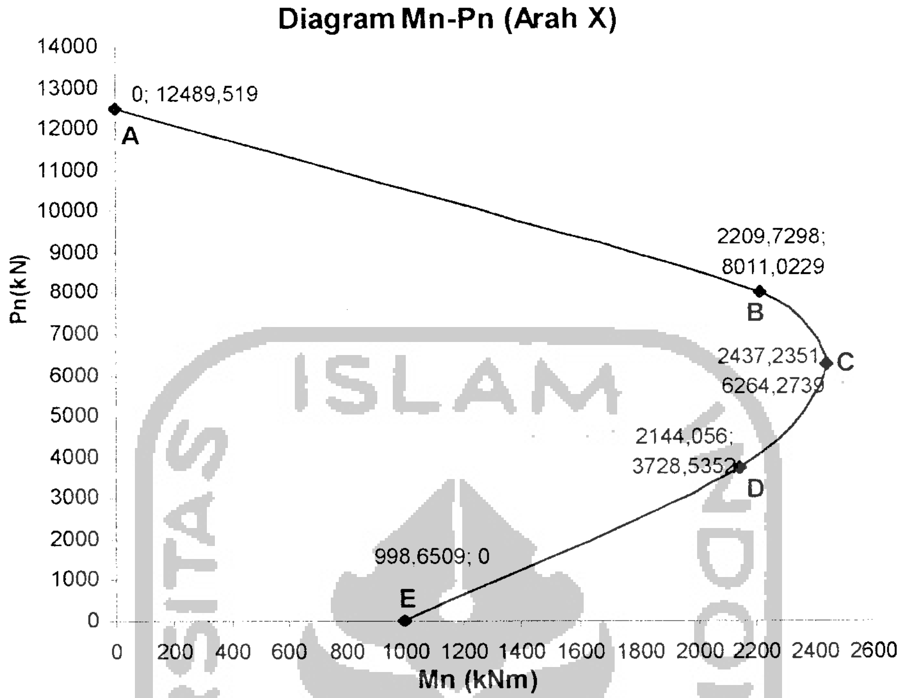
= 3728,5352 kN

$$Mn = Cc\left(y - \frac{a}{2}\right) + Cs(y - d') + Ts(d - y) \quad \text{......................} \quad (3.4.35)$$

$$Mn = 3793{,}125.\left(450 - \frac{255}{2}\right) + 1151{,}2182\ (450\text{-}61) +$$

1215,808 (839-450)

= 2144,056 kNm



Gambar 5.7 Diagram Mn-Pn

5.3.2.2 Desain Tulangan Geser Kolom

a. Gaya geser rencana kolom

Diambil contoh perhitungan pada kolom A

h_n = 3500 mm = 3,5 m

Dari perhitungan $M_{u,k}$ di atas didapatkan :

$M_{u,kx\ atas}$ = 313,8980 kNm

$M_{u,kx\ bwh}$ = 466,2183 kNm

$$V_{u,k-x} = \frac{M_{ukx-atas} + M_{ukx-bawah}}{h_n} \qquad (3.4.44)$$

$$= \frac{313,8980 + 466,2183}{3,5}$$

= 222,8904 kN

$M_{u,ky\ atas}$ = 369,0852 kNm

$M_{u,ky\ bwh}$ = 487,8028 kNm

$$V_{u,k-y} = \frac{M_{uky-atas} + M_{uky-bawah}}{h_n}$$

$$= \frac{369{,}0852 + 487{,}8028}{3{,}5}$$

$$= 244{,}8251 \text{ kN}$$

b. Gaya geser maksimum kolom

Dari output SAP lampiran 1 tabel 1.3.8 didapatkan :

$V_{D,k-x}$ = 17,3042 kN $V_{D,k-y}$ = 0 kN

$V_{L,k-x}$ = 2,3995 kN $V_{L,k-y}$ = 0 kN

$V_{E,k-x}$ = 144,3325 kN $V_{E,k-y}$ = 139,9042 kN

$$V_{u,kx} = 1{,}05\left(V_{Dk} + V_{Lk} + \frac{4}{K}V_{Ek}\right) \quad \text{...} \quad (3.4.45)$$

$$= 1{,}05\left(17{,}3042 + 2{,}3995 + \frac{4}{1}144{,}3325\right)$$

$$= 626{,}8854 \text{ kN}$$

$$V_{u,ky} = 1{,}05\left(V_{Dk} + V_{Lk} + \frac{4}{k}V_{Ek}\right) \quad \text{...} \quad (3.4.45)$$

$$= 1{,}05\left(0 + 0 + \frac{4}{1}139{,}9042\right)$$

$$= 587{,}5976 \text{ kN}$$

Untuk masing-masing arah nilai gaya geser rencana tidak boleh melebihi gaya geser maksimum, kemudian dari kedua nilai tersebut diambil nilai terbesar, jadi gaya geser terpakai $V_{u,k}$ = 244,8251 kN.

c. Penulangan geser kolom di daerah sendi plastis

Dari perhitungan gaya aksial kolom dan gaya geser kolom didapat :

$N_{u,k}$ = 2920,3199 kN $\quad$ $V_{u,k}$ = 244,8251 kN

b = 900 mm $\quad$ d = 660 mm

Ag = 630000 mm^2

$$Vs = \frac{Vu}{\phi} = \frac{244,8251}{0,6} = 408,0418 \text{ kN} \quad \text{.......... (3.4.47)}$$

$$S = \frac{Av.fy.d}{Vs} = \frac{2.78,5.300.660}{408,0418.10^3} = 76,1834 \text{ mm} \quad \text{.......... (3.4.46)}$$

dipakai $P_{10\text{-}75}$

d. Desain penulangan geser kolom luar daerah sendi plastis

$$Vc = \left[1 + \frac{Nu,k}{14.Ag}\left(\frac{1}{6}\sqrt{f'c}\right)b.d\right] \quad \text{.......... (3.4.48)}$$

$$= \left[1 + \frac{2920,3199.10^3}{14.630000}\left(\frac{1}{6}\sqrt{25}\right)900.660\right]10^{-3}$$

$$= 534,6952 \text{ kN}$$

$$Vs = \frac{Vu}{\phi} - Vc = \frac{159,1363}{0,6} - 534,6952 = 269,4679 \text{ kN} \quad \text{.......... (3.4.49)}$$

$$S = \frac{Av.fy.d}{Vs} = \frac{2.78,5.300.660}{269,4679.10^3} = 115,3607 \text{ mm} \quad \text{.......... (3.4.46)}$$

dipakai $P_{10\text{-}100}$

5.3.3 Pendetailan

Sebagai contoh diambil portal E lantai 3

5.3.3.1 Balok

a. Penentuan jarak bersih antar tulangan sejajar yang diletakkan selapis harus lebih besar sama dengan 25 mm.

$$jbt = \frac{250 - 2.30 - 2.10 - 4.22}{3} = 27,3 \text{ mm} > 25\text{mm}$$

Tulangan atas terpakai 6D-22 diletakkan dalam 2 lapis tulangan dan tulangan bawah terpakai 4D-22 diletakkan dalam satu lapis tulangan dengan jarak bersih antar tulangan 54 mm.

b. Panjang penanaman kait sengkang tertutup untuk ø10 diambil sebesar 6.db = 6.10 = 60 mm

c. Rasio lebar terhadap tinggi tidak oleh kurang dari 0,3

$$\frac{b}{h} = \frac{250}{500} = 0,5 > 0,3 \quad \text{aman}$$

d. Pada sembarang penampang struktur lentur jumlah tulangan atas dan bawah lebih besar sama dengan :

$$\frac{1,4.bw.d}{fy} = \frac{1,4.250.449}{400} = 392,875 \text{ mm}^2.$$

$$6.A\text{ø}22 = 6.\tfrac{1}{4}.3,14.22 = 6.379,94 = 2279,64 \text{ mm}^2 > 392,875 \text{ mm}^2$$

$$3.A\text{ø}22 = 3.\tfrac{1}{4}.3,14.22 = 3.379,94 = 1139,82 \text{ mm}^2 > 392,875 \text{ mm}^2$$

e. Sengkang tertutup harus dipasang sepanjang 2 kali tinggi komponen struktur diukur dari muka komponen struktur pendukung pada kedua ujung komponen struktur lentur.

5.3.3.2 Kolom

a. Dimensi penampang terpendek yang diukur pada satu garis lurus yang melalui titik berat penampang tidak boleh kurang dari 300mm.

b. Rasio dimensi penampang terpendek terhadap dimensi yang tegak lurus tidak boleh kurang dari 0,4 .

   Pada kolom A lantai 3, b/h = 700/900 = 0,7778

c. Rasio tinggi kolom terhadap dimensi penampang kolom terpendek tidak lebih dari 25.

d. Untuk kolom yang mengalami momen bolak-balik, rasio tidak boleh kurang dari 16.

   tinggi kolom/lebar kolom = 4000/700 = 5,71 < 16.

e. Rasio tulangan tidak boleh kurang dari 0,01 dan tidak melebihi 0,06.

f. Tulangan tranversal harus dipasang dengan spasi tidak melebihi :

   * ¼ dimensi komponen terkecil = ¼.700 = 175 mm,
   * 8 kali diameter tulangan longitudinal = 8.22 = 176 mm,
   * 100 mm

5.3.4 Desain Panel Pertemuan Balok Kolom

Data-data (dapat dilihat lebih lengkap pada lampiran 1 tabel 1.4.1):

Kolom : 900 x 700		Balok : 250 x 500	
h_c	= 700 cm = 0,7 m	b_j	= 500 cm = 0,5 m
L_{ki}	= 5 m	L_{ka}	= 5 m
$L_{ki'}$	= 4,3 m	$L_{ka'}$	= 4,3 m

Mkap,b ki = 346,7628 kNm   Mkap,b ka = 470,4221 kNm

Zki = 0,3798 m   Zka = 0,3569 m

hk,a = 4 m   hk,b = 4 m

$$C_{ki} = T_{ki} = 0{,}7.\frac{M_{kap,b\_ki}}{Z_{ki}} = 0{,}7.\frac{346{,}7682}{0{,}3798} = 639{,}0707 \text{ kN} \quad \ldots\ldots\ldots\ldots \quad (3.5.2)$$

$$T_{ka} = C_{ka} = 0{,}7.\frac{M_{kap,b\_ka}}{Z_{ka}} = 0{,}7.\frac{470{,}4221}{0{,}3569} = 922{,}6427 \text{ kN} \quad \ldots\ldots\ldots\ldots \quad (3.5.3)$$

$$V_{kol} = \frac{0{,}7\left(\frac{L_{ki}}{L_{ki}'}.M_{kap,b\_ki} + \frac{L_{ka}}{L_{ka}'}.M_{kap,b\_ka}\right)}{\frac{1}{2}\left(h_{k,a} + h_{k,b}\right)} \quad \ldots\ldots\ldots\ldots \quad (3.5.4)$$

$$= \frac{0{,}7\left(\frac{5}{4{,}3}.346{,}7628 + \frac{5}{4{,}3}.470{,}4221\right)}{\frac{1}{2}(4+4)} = 166{,}2876 \text{ kN}$$

$$V_{jh} = C_{ki} + T_{ka} - V_{kol} \quad \ldots\ldots\ldots\ldots \quad (3.5.1)$$

$$= 639{,}0707 + 922{,}6427 - 166{,}2876$$

$$= 1395{,}4257 \text{ kN}$$

$$V_{jv} = \frac{h_c}{b_j}V_{jh} \quad \ldots\ldots\ldots\ldots \quad (3.5.9)$$

$$= \frac{0{,}7}{0{,}5}1395{,}4257$$

$$= 1953{,}5960 \text{ kN}$$

Kontrol tegangan geser horizontal minimal :

$$V_{jh} = \frac{V_{jh}}{b_j.h_c} < 1{,}5.f'c \quad \ldots\ldots\ldots\ldots \quad (3.5.5)$$

$$= \frac{1395{,}4257.10^3}{500.700} < 1{,}5.25$$

$$= 3{,}9868 \text{ N/mm}^2 < 7{,}5 \text{ N/mm}^2 \qquad \text{OK.}$$

Penulangan geser horizontal

$N_{u,k}$ = 2920,3199 kN

Ag = 900 x 700 = 630000 mm^2

$$V_{ch} = \frac{2}{3}\sqrt{\left(\frac{N_{u,k}}{Ag}\right)} - 0,1.fc'.b_j.h_c \quad \text{.........} \quad (3.5.6)$$

$$= \frac{2}{3}\sqrt{\left(\frac{2920,3199.10^3}{630000}\right)} - 0,1.25.500.700 = 340,9724 \text{ kN}$$

V_{sh} = V_{jh} - V_{ch} .... (3.5.7)

= 1395,4257 - 340,9724 = 1054,4533 kN

$$A_{sh} = \frac{V_{sh}}{fy} \quad \text{.........} \quad (3.5.8)$$

$$= \frac{1054,4533.10^3}{300} = 3514,8442 \text{ mm}^2$$

$Ø_{sengkang}$ = 12 mm

A_s rangkap = 4.(0,25.3,14.12^2) = 452,16 mm^2

$$\text{Jumlah sengkang} = \frac{A_{sh}}{A_s rangkap}$$

$$= \frac{3514,8442}{452,16} = 7,7735 \approx 8 \text{ buah}$$

$$\text{Jarak sengkang} = \frac{b_j}{Jml\_sengkang + 1}$$

$$= \frac{500}{8+1} = 55,5556 \text{ mm}$$

Jarak sengkang maksimal = jarak sengkang pada sendi plastis kolom = 65 mm

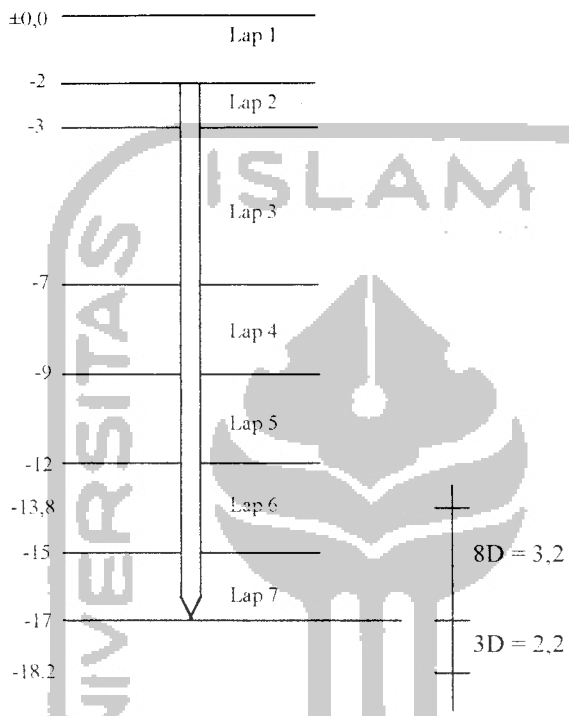
Jarak sengkang terpasang = 65 mm

$$\text{Jumlah sengkang terpasang} = \left(\frac{500}{65}\right) - 1 = 6,6923 \approx 7 \text{ buah}$$

5.3.5 Desain Pondasi

5.3.5.1 Perhitungan Kapasitas Tiang Tunggal

Kedalaman tiang dan data penyelidikan tanah



Gambar 5.8 Kedalaman Tiang Pancang

Tabel 5.6 Nilai CPT dan N-SPT

Depth	N-Value	Qc (kg/cm^2)	JHL (kg/cm^2)
-2	2	6	-
-3	2	3	-
-4	2	4	-
-5	3	8	-
-6	3	10	-
-7	4	28	-
-8	4	10	-
-9	13	20	-
-10	13	24	-
-11	12	20	-
-12	21	76	-
-13	50	90	-
-13,8	82	94	-

-14	90	90	-
-15	100	136	-
-16	100	172	-
-17	100	180	1680
-18	100	200	-
-18,2	100	230	-

Data-data :

Diameter tiang pancang = 0,3 m

Safety Faktor (SF) = 2,5

a. Perhitungan daya dukung tiang pancang tunggal cara *N-SPT*

Tahanan ujung (Qp)

$$Qp = Ap.qp \leq Ap.(400.\bar{N}) \quad \ldots\ldots\ldots\ldots \quad (3.6.2)$$

$$Ap = 0{,}3.0{,}3 = 0{,}09 \text{ m}^2$$

$$\bar{N} = \frac{82+90+100+100+100+100+100}{7} = 96$$

$$qp = 40.\bar{N}.\frac{Lb}{D} \quad \ldots\ldots\ldots\ldots \quad (3.6.3)$$

$$= 40.96.\frac{15}{0{,}3}$$

$$= 192000 \text{ kN}$$

$$Qp = 0{,}09.192000 \leq 0{,}09(400.96)$$

$$= 17280 \leq 3456$$

maka diambil Qp yang kecil = 3456 kN

Tahanan selimut (Qs)

$$Qs = fav.As \quad \ldots\ldots\ldots\ldots \quad (3.6.4)$$

$$As = (4.0{,}3.15) = 18 \text{ m}^2$$

$$\bar{N}_s = \frac{(2+2+2+2+3+3+4+4+13+13+12+21+50+82+90+100+100+100)}{17}$$

$= 35{,}3529$

$\text{fav} = 2.\ \bar{N}_s$ .... (3.6.5)

$= 2.35{,}3529$

$= 70{,}7059\ \text{kN/m}^2$

$\text{Qs} = \text{fav.As}$

$= 70{,}7059.18$

$= 1272{,}7062\ \text{kN}$

$\text{Qu} = \text{Qp} + \text{Qs}$ .... (3.6.17)

$= 3456 + 1272{,}7062$

$= 4728{,}7062\ \text{kN}$

$\text{Qa} = \frac{Qu}{SF} = \frac{4728{,}7062}{2{,}5}$ .... (3.6.18)

$= 1891{,}4824\ \text{kN}$

$= 192{,}8117\ \text{ton}$

b. Perhitungan daya dukung tiang pancang tunggal cara CPT

Tahanan ujung (Qp)

$$\text{Qc} = \frac{(94+90+136+172+180+200+230)}{7}$$

$= 157{,}4286\ \text{kg/cm}^2$

$\text{Qp} = \text{Ap.qc}$ .... (3.6.6)

$= (30.30).157.4286$

$= 141685{,}74\ \text{kg}$

$= 141{,}68574\ \text{ton}$

Tahanan selimut (Qs)

$Qs = As.JHP$ .... (3.6.7)

$= (4.30).1680$

$= 201600$ kg

$= 201,6$ ton

$Qu = Qp + Qs$ .... (3.6.17)

$= 141,68574 + 201,6$

$= 343,2857$ ton

$Qa = \dfrac{Qu}{SF} = \dfrac{343,2857}{2,5}$ .... (3.6.18)

$= 137,3143$ ton

c. Perhitungan daya dukung tiang pancang tunggal cara laboratorium

Data tanah :

Lap 1	**Lap 2**	**Lap 3**
Gs = 2,537	Gs = 2,645	Gs = 2,619
$\gamma b = 1,846$ t/m³	$\gamma b = 2$ t/m³	$\gamma b = 1,821$ t/m³
$\gamma k = 1,401$ t/m³	cu = 0,4587 kg/cm²	cu = 0,4077 kg/cm²
cu = 0,101 kg/cm²	Ø = 2°	Ø = 2°
Ø = 2°		

Lap 4	**Lap 5**	**Lap 6**
Gs = 2,67	Gs = 2,612	Gs = 2,608
$\gamma b = 2,11$ t/m³	$\gamma b = 2,811$ t/m³	$\gamma b = 1,917$ t/m³
cu = 0,5097 kg/cm²	cu = 0,8359 kg/cm²	cu = 1,9878 kg/cm²
Ø = 2°	Ø = 2°	Ø = 2°

Lap 7

Gs = 2,601

γb = 2,18 t/m³

cu = 2,5484 kg/cm²

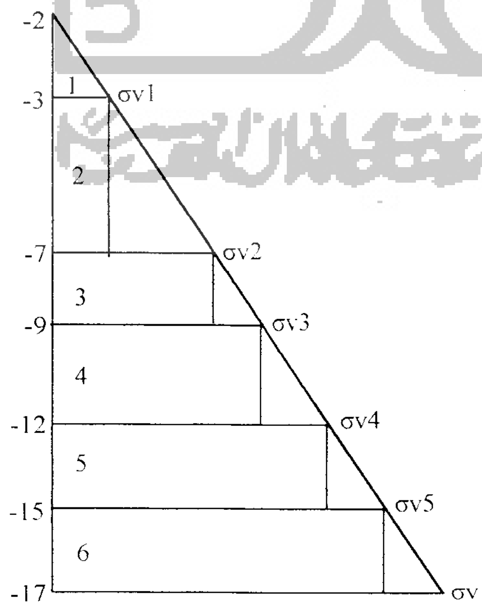
Ø = 2°

Metode α

Tabel 5.7 Perhitungan Tahanan Selimut (Qs) dengan Metode α

Depth	As = Ap.ΔL	Cu	α	Qs = As.α.Cu
2 – 3	(4 x 0,3) x 1 = 1,2	45	1	54
3 -7	(4 x 0,3) x 4 = 4,8	40	1	192
7 – 9	(4 x 0,3) x 2 = 2,4	50	1	120
9 -12	(4 x 0,3) x 3 = 3,6	82	0,65	191,88
12 – 15	(4 x 0,3) x 3 = 3,6	195	0,28	196,56
15 -17	(4 x 0,3) x 2 = 2,4	250	0,24	144
			∑Qs	898,44 kN
				91,5841 ton

Metode λ



Gambar 5.9 Tegangan efektif tanah

$\sigma v1 = h1.\gamma b1 \quad = 1.2 \quad = 2\ t/m^2$ .... (3.6.10)

$\sigma v2 = \sigma v1 + (h2.\gamma b2) = 2 + (4.1{,}821) = 9{,}284\ t/m^2$

$\sigma v3 = \sigma v2 + (h3.\gamma b3) = 9{,}284 + (2.2{,}11) = 13{,}504\ t/m^2$

$\sigma v4 = \sigma v3 + (h4.\gamma b4) = 13{,}504 + (3.1{,}811) = 18{,}937\ t/m^2$

$\sigma v5 = \sigma v4 + (h5.\gamma b5) = 18{,}937 + (3.1{,}917) = 24{,}688\ t/m^2$

$\sigma v6 = \sigma v5 + (h6.\gamma b6) = 24{,}688 + (2.2{,}18) = 29{,}048\ t/m^2$

Luas:

$A1 = \frac{1}{2}.h1.\ \sigma v1 \quad = \frac{1}{2}.1.2 \quad = 1\ t/m$ .... (3.6.9)

$A2 = \frac{1}{2}.h2.\ (\sigma v1 + \sigma v2) = \frac{1}{2}.4.(2 + 9{,}284) = 22{,}568\ t/m$

$A3 = \frac{1}{2}.h3.\ (\sigma v2 + \sigma v3) = \frac{1}{2}.2.(\ 9{,}284 + 13{,}504) = 22{,}788\ t/m$

$A4 = \frac{1}{2}.h4.\ (\sigma v3 + \sigma v4) = \frac{1}{2}.3.(\ 13{,}504 + 18{,}937) = 48{,}6615\ t/m$

$A5 = \frac{1}{2}.h5.\ (\sigma v4 + \sigma v5) = \frac{1}{2}.3.(\ 18{,}937 + 24{,}688) = 65{,}4375\ t/m$

$A6 = \frac{1}{2}.h6.\ (\sigma v5 + \sigma v6) = \frac{1}{2}.2.(\ 24{,}688 + 29{,}048) = 53{,}7360\ t/m$

$$\bar{\sigma} = \frac{\sum Ai}{L} = \frac{214{,}1910}{15} = 14{,}2794\ t/m \quad \text{.......................} \quad (3.6.11)$$

$$\bar{c}_u = \frac{\sum (c_u.i.hi)}{L} \quad \text{..} \quad (3.6.12)$$

$$= \frac{(45.1) + (40.4) + (50.2) + (82.3) + (195.3) + (250.2)}{15}$$

$= 109{,}0667\ kN/m^2 = 11{,}1179\ ton/m^2$

Dari tabel Braja M. Das untuk kedalaman 15 m, didapat $\lambda = 0{,}21$

$$fs = \lambda.(\bar{\sigma} + (2.\bar{c}_u)) \quad \text{..} \quad (3.6.14)$$

$= 0{,}21(14{,}2794 + (2.11{,}1179))$

$= 7{,}6682\ ton/m^2$

Qs = Ap.L.fs .... (3.6.13)

= (4.0,3).15.7,6682

= 138,0277 ton

Metode β

Tabel 5.8 Perhitungan Tahanan Selimut (Qs) dengan Metode β

Depth	Ø	(1-sin Ø)	tan Ø	Σv(t/m²)	fi(ton)
2 – 3	2	0,9651	0,0349	1	0,0337
3 - 7	2,5	0,9564	0,0437	5,5620	0,2358
7 – 9	3	0,9477	0,0524	11,3940	0,5658
9 - 12	2,5	0,9564	0,0437	16,2205	0,6779
12 – 15	5,5	0,9042	0,0963	21,8125	1,8993
15 - 17	5,5	0,9042	0,0963	26,8680	2,3395

Qs = p $\sum$ fi.Li .... (3.6.15)

= (4.0,3) x [(0,0337.1) + (0,2358.4) + (0,5658.2) + (0,6779.3)

+ (1,8993.3) + (2,3395.2)]

= 17,4229 ton

= 170,9188 kN

Dari ketiga metode dapat diperoleh Qs sebagai berikut :

Qsα = 91,584 ton

Qsλ = 138,0275 ton

Qsβ = 17,4229 ton

Dari ketiga Qs diambil 2 yang hampir sama :

$$\bar{Qs} = \frac{Qs\alpha + Qs\lambda}{2} = \frac{91,5841 + 138,0275}{2} = 114,8058 \text{ ton}$$

$$= 1126,2447 \text{ kN}$$

$Qp = Ap.Nc^*.Cu$ .... (3.6.1)

$= (0,3)^2.9.300$

$= 243$ kN

$= 24,7706$ ton

$Qu = Qp + Qs$ .... (3.6.17)

$= 24,7706 + 114,8058$

$= 139,5764$ ton

$$Qa = \frac{Qu}{SF} = \frac{139,5764}{2,5} = 54,1792 \text{ ton} \quad \text{.....................} \quad (3.6.18)$$

Qa dapat disimpulkan:

Qa N-SPT = 192,8117 ton

Qa CPT = 137,3143 ton

Qa Lab = 54,1792 ton

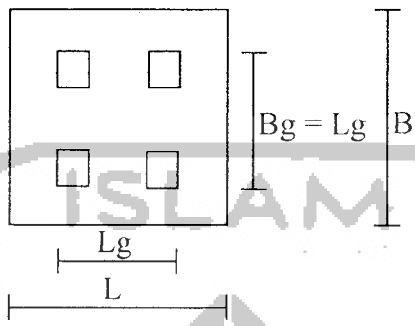
Kesimpulan : Dari hasil perhitungan di atas, hasil *Qa* lab sangat kecil dibandingkan *Qa N-SPT* dan *Qa CPT* . Nilai *Qa* diambil nilai terkecil yaitu 137,3143 ton.

Beban rencana 1 kolom adalah Qg = 3369,9132 kN = 343,5182 ton. dengan Qijin = 137,3143 ton/tiang, maka dalam 1 kelompok tiang diperlukan tiang sebanyak $\frac{Qg}{Qijin} = \frac{343,5182 \text{ ton}}{137,3143 \; ton/tiang}$

$= 2,5017$ tiang ≈ 4 tiang.

5.3.5.2 Perhitungan kapasitas kelompok tiang

Jumlah tiang yang digunakan dicoba sesuai dengan hasil perhitungan sebelumnya yaitu 4 buah dengan m = n = 2.



Gambar 5.10 Dimensi Pile Cap dengan 4 buah Tiang Pancang

D = 0,3 m

S = 3,5.D = 3,5.0,3 = 1,05 m

$$Bg = Lg = (m-1)s + 2(D/2) \quad \ldots\ldots (3.6.19)$$

$$= (2-1).1,05 + 2\left(0,3/2\right) = 1,35 \text{ m}$$

Jarak tepi tiang pancang terluar dengan tepi poer dipakai :

2D = 2.0,3 = 0,6 m

$$L = B = Lg + 2(2.D) = 1,05 + 2(2.0,3) = 2,55 \text{ m} \quad \ldots\ldots (3.6.20)$$

Jumlah kapasitas dukung individual tiang

$$Qu_i = m.n.(9.Ap.Cu + \sum \alpha.Ap.Cu.L) \quad \ldots\ldots (3.6.21)$$

$$= 2.2(9.0,3^2.250 + 898,44)$$

$$= 4403,7600 \text{ kN}$$

$$= 448,9052 \text{ ton}$$

Kapasitas kelompok tiang berdasar blok

$$Qu_b = Lg.Bg.Cu.Nc^* + \sum 2.(Lg + Bg).Cu.L \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6.22)$$

$$\frac{L}{Bg} = \frac{15}{1{,}35} = 11{,}111$$

$$\frac{Lg}{Bg} = \frac{1{,}35}{1{,}35} = 1$$

dari nilai tersebut di atas didapat Nc* = 9

Qu_b = 1,35.1,35.250.9 + {2[(1,35+1,35).45.1] + [(1,35+1,35).40.4] +

[(1,35+1,35).50.2] + [(1,35+1,35).82.3] + [(1,35+1,35).95.3] +

[(1,35+1,35).250.2]}

= 12935,0250 kN

= 1318,5550 ton

Qu_i = 448,9052 ton < Qu_b = 1318,5550 ton

Dipakai Qu = 448,9052 ton

Kontrol :

Beban kolom diambil beban kolom yang terbesar, yaitu:

P = 3369,9132 kN

= 343,5182 ton

$$Qall = \frac{Qu}{SF} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6.23)$$

$$= \frac{448{,}9052}{2{,}5}$$

= 179,5621 ton

$Qall$ = 179,5621 ton < P = 343,5182 ton

Karena $Qall < P$ maka blok tiang pancang tidak mampu menahan beban yang bekerja sehingga tidak aman, untuk itu jumlah tiang pancang diperbanyak.

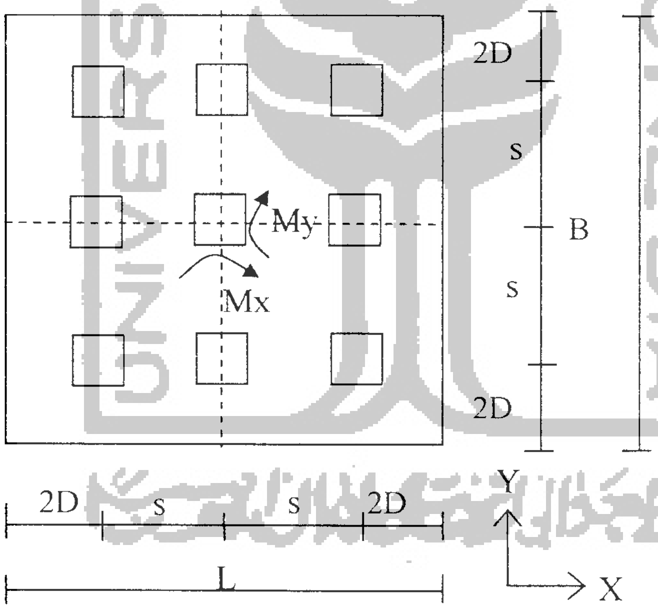
Dicoba jumlah tiang pancang 9 buah dengan m = n = 3

$$Bg = Lg = (m-1)s + 2(D/2) \quad \ldots\ldots\ldots\ldots \quad (3.6.19)$$

$$= (3-1).1{,}05 + 2\left(\frac{0{,}3}{2}\right) = 2{,}4 \text{ m}$$

$$L = B = Lg + 2(2.D) \quad \ldots\ldots\ldots\ldots \quad (3.6.20)$$

$$= 2{,}4 + 2(2.0{,}3) = 3{,}6 \text{ m}$$



Gambar 5.11 Dimensi Pile Cap dengan 9 Buah Tiang Pancang

Jumlah kapasitas dukung individual tiang

$$Qu_t = m.n.(9.Ap.Cu + \sum \alpha.Ap.Cu.L) \quad \ldots\ldots\ldots\ldots \quad (3.6.21)$$

$$= 3.3.(9.0{,}3^2.250 + 898{,}44)$$

$$= 9908{,}46 \text{ kN} = 1010{,}0367 \text{ ton}$$

Kapasitas kelompok tiang berdasar blok

$$Qu_b = Lg.Bg.Cu.Nc^* + \sum 2.(Lg + Bg).Cu.L \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6.22)$$

$$\frac{L}{Bg} = \frac{15}{2,4} = 6,25$$

$$\frac{Lg}{Bg} = \frac{2,4}{2,4} = 1$$

dari nilai tersebut diatas didapat Nc* = 9

Qu_b = 2,4.2,4.250.9 + {2[(2,4+2,4).45.1] + [(2,4+2,4).40.4] +

[(2,4+2,4).50.2] + [(2,4+2,4).82.3] + [(2,4+2,4).95.3] +

[(2,4+2,4).250.2]}

= 28665,6000 kN

= 2922,0795 ton

Qu_i = 1010,0367 ton < Qu_b = 2922,0795 ton

Dipakai Qu = 1010,0367 ton

Kontrol :

$$Qall = \frac{Qu}{SF} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6.23)$$

$$= \frac{1010,0367}{2,5}$$

= 404,0147 ton

$Qall$ = 404,0147 ton > P = 343,5182 ton (aman)

5.3.5.3 Perencanaan Pile Cap

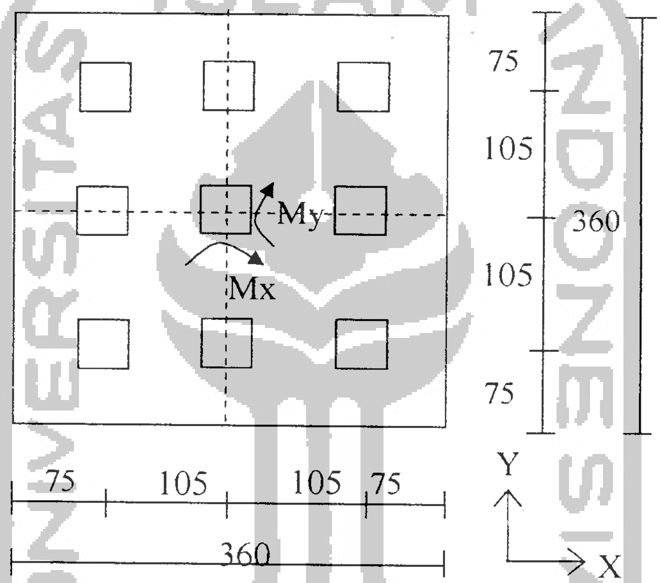
Dalam perencanaan pondasi pile cap dipergunakan untuk menyatukan kelompok tiang pancang yang bekerja pada suatu kolom.

Kolom yang dipakai dalam analisis ini adalah kolom dengan beban bekerja paling besar. Besarnya gaya momen pada kolom diambil dari lampiran I tabel 1.3.3 dan gaya aksial kolom diambil dari lampiran I tabel 1.3.5, sebesar :

$P_{u,k}$ = 3369,9132 kN

$M_{u,kY}$ = 487,8028 kN

$M_{u,kX}$ = 466,2183 kN



Gambar 5.12 Konfigurasi Kelompok Tiang Pancang

$\sum X^2 = 6.1{,}05^2 = 6{,}6150\ m^2$ .... (3.6.24)

$\sum Y^2 = 6.1{,}05^2 = 6{,}6150\ m^2$ .... (3.6.25)

Beban yang bekerja pada satu tiang

$$P\max = \frac{\sum P}{n} \pm \frac{Mu,kX}{\sum X^2} \pm \frac{Mu,kY}{\sum Y^2}$$ .... (3.6.26)

Σ P = Pu,k + Pile Cap + Tanah Urug .... (3.6.27)

$= 3369{,}9132 + (3{,}6.3{,}6.1{,}2.24) + \{[(3{,}6.3{,}6) - (0{,}9.0{,}7)]0{,}83.18{,}093\}$

= 3833,4463 kN

$$P_{maks} = \frac{3833{,}4463}{9} + \frac{466{,}2183}{6{,}6150} + \frac{487{,}8028}{6{,}6150}$$

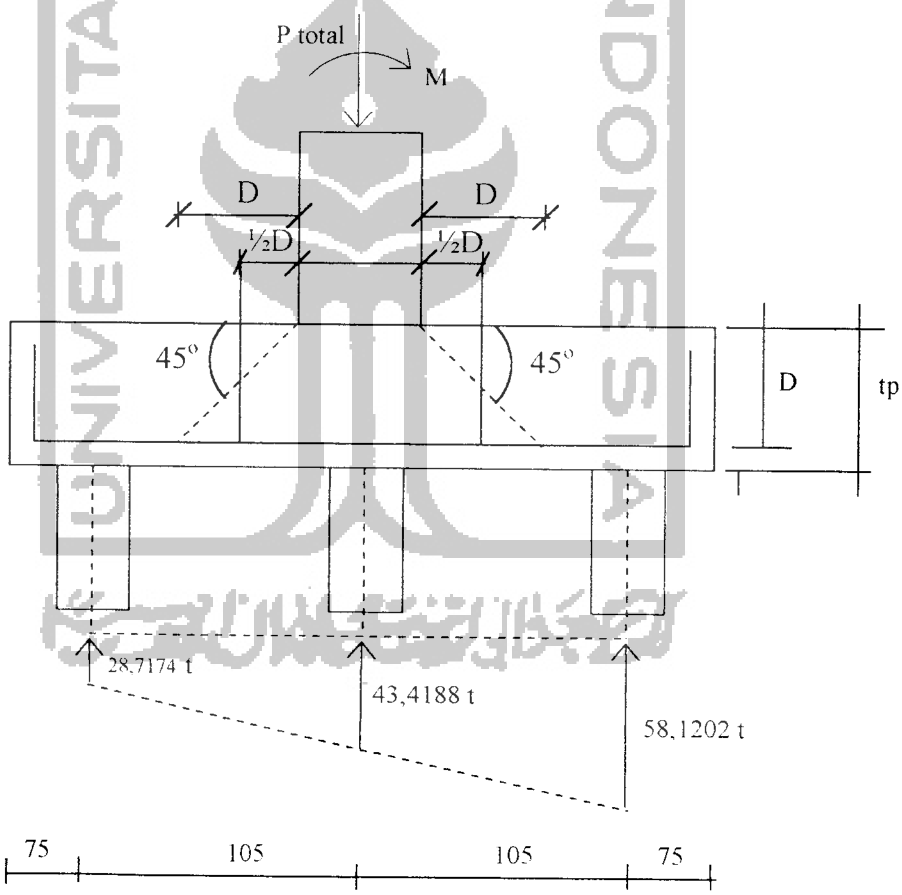
$= 570{,}1594$ kN

$= 58{,}1202$ ton

$$P_{min} = \frac{3833{,}4463}{9} - \frac{466{,}2183}{6{,}6150} - \frac{487{,}8028}{6{,}6150}$$

$= 281{,}7176$ kN

$= 28{,}7174$ ton



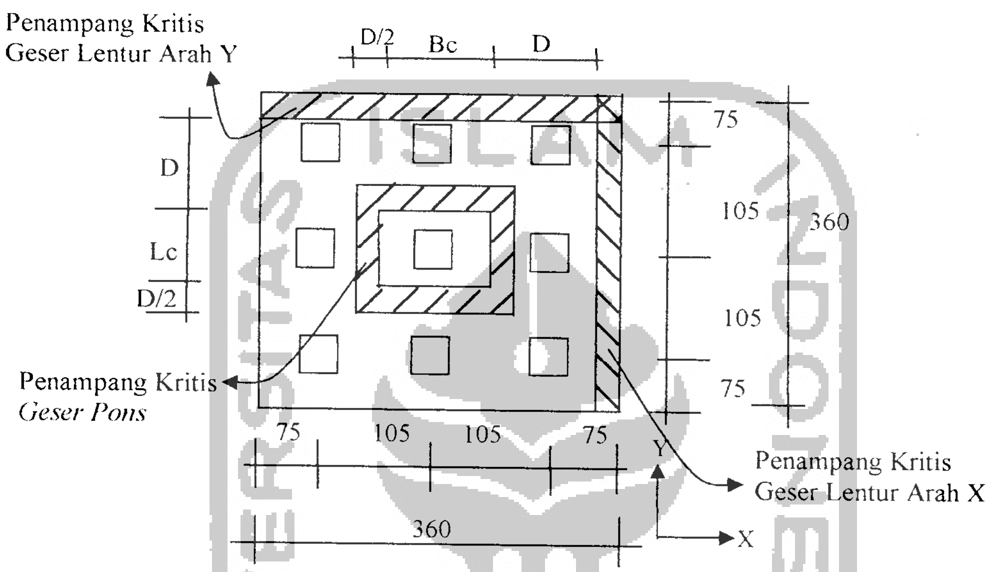
Gambar 5.13 Reaksi Tiang Pancang Akibat Beban Aksial dan Momen

Perencanaan tebal pile cap

Dicoba tebal pile cap, tp = 1200 mm

$D = tp - pb - \Phi\ tul$ .... (3.6.28)

$= 1200 - 75 - 22 = 1103$ mm



Gambar 5.14 Penampang Kritis Pile Cap Akibat Geser

Kontrol geser satu arah (geser lentur)

Arah X dan Y

$V_u = n.Pmax$ .... (3.6.29)

$= 3.\ 570,1594$

$= 1710,4781$ kN

$= 174,3607$ ton

$Vc = \frac{1}{6}\sqrt{fc'}.B.D$ .... (3.6.30)

$= \left(\frac{1}{6}\sqrt{25}.3300.1103\right)10^{-3} = 3033,2500$ kN

$= 309,1998$ ton

$\varnothing Vc = 0{,}6.\ 309{,}1998$ .... (3.6.31)

$= 185{,}5199 \text{ ton} > Vu = 174{,}3607 \text{ ton}$

Kontrol Geser Dua Arah (Geser Pons)

Geser pons akibat beban kolom :

$$V_c = \left(1+\frac{2}{\beta_0}\right)\left(\frac{\sqrt{fc'}}{6}\right)b_o D \leq 0{,}33.\sqrt{fc'}b_o D$$ .... (3.6.33)

$$\beta_o = \frac{hc}{bc}$$ .... (3.6.34)

$$= \frac{0{,}9}{0{,}7} = 1{,}2857$$

$$b_o = 2((h_c + D) + (b_c + D))$$ .... (3.6.35)

$$= 2((900 + 1103) + (700 + 1103))$$

$$= 7612 \text{ mm}$$

$$V_c = \left(1+\frac{2}{1{,}2857}\right)\left(\frac{\sqrt{25}}{6}\right)7612.1103.10^{-3}$$

$$= 17880{,}4470 \text{ kN}$$

$$= 1822{,}6755 \text{ ton}$$

$$V_c = 0{,}33.\sqrt{25}.7612.1106$$

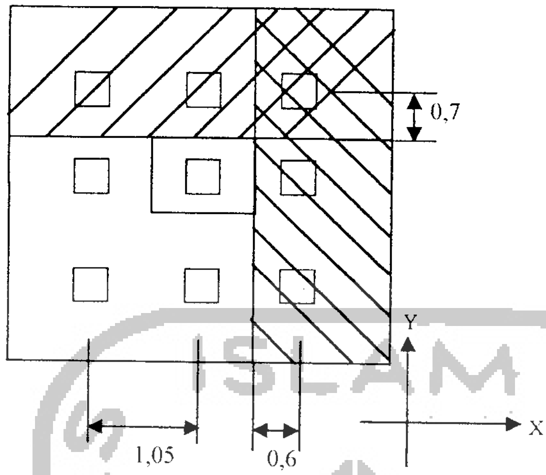
$$= 13993{,}3933 \text{ kN}$$

$= 1426{,}4417$ ton (menentukan)

$\phi.V_c = 0{,}6.\ 1426{,}4417$ .... (3.6.31)

$= 855{,}8650$ ton

Penulangan Lentur Pile Cap



Gambar 5.15 Penampang Kritis Pile Cap Akibat Momen

Momen Lentur pada Arah-X :

$M_{u,X}$ = 570,1594.0,6

= 342,0956 kNm

= 34,8721 tonm

d = 1200-75-22 = 1103 mm ; fc' = 25 MPa ; fy = 400 MPa .... (3.6.37)

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{400} = 0,0035 \quad \text{.......... (3.6.38)}$$

$$\rho_b = \frac{0,85.fc'}{fy}\beta\left(\frac{600}{600+fy}\right) \quad \text{.......... (3.6.39)}$$

$$= \frac{0,85.25}{400}0,85\left(\frac{600}{600+400}\right) = 0,02709$$

$$\rho_{max} = 0,75.\rho_b = 0,75.0,02709 = 0,02032 \quad \text{.......... (3.6.40)}$$

$$R_n = \frac{Mu/\phi}{b.d^2} = \frac{342,0956.10^6/0,8}{1000.1103^2} = 0,3515 \text{ MPa} \quad \text{.......... (3.6.41)}$$

$$m = \frac{fy}{0{,}85.fc'} = \frac{400}{0{,}85.25} = 18{,}8235 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6.42)$$

$$\rho_{\text{perlu}} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.R_n.m}{fy}}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6.43)$$

$$= \frac{1}{18{,}8235}\left(1 - \sqrt{1 - \frac{2.0{,}3515.18{,}8235}{400}}\right) = 0{,}0009 < \rho_{\min}$$

1,33 ρ perlu = 1,33.0,0009 = 0,0012

digunakan $\rho = 0{,}0012$

$$As_{perlu} = \rho.b.d \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6.44)$$

$$= 0{,}0012.1000.1103 = 1299{,}9024 \text{ mm}^2$$

digunakan tulangan D_{22}, $A\Phi_{22} = 379{,}94 \text{ mm}^2$

jarak antar tulangan :

$$S \leq \frac{A_{\phi 22}.1000}{As_{perlu}} = \frac{379{,}94.1000}{1299{,}9024} = 292{,}2835 \text{ m} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6.45)$$

digunakan D22-290

$$As_{ada} = \frac{A_{\phi 22}.b}{S} = \frac{379{,}94.1000}{290} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6.46)$$

$$= 1310{,}1379 \text{ mm}^2 > As_{perlu} = 1299{,}9024 \text{ mm}^2$$

cek kapasitas lentur arah X:

$$a = \frac{As_{ada}.fy}{0{,}85.fc'.b} = \frac{1310{,}1379.400}{0{,}85.25.1000} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6.47)$$

$$= 24{,}6614 \text{ mm}$$

$$Mn = As_{ada} . fy . \left(d - \frac{a}{2} \right) \quad \text{..} \quad (3.6.48)$$

$$= 1310{,}1379.400 \left(1103 - \frac{24{,}6614}{2} \right) = 571{,}5709 \text{ kNm}$$

ϕMn = 0,8. 571,5709 kNm

$$= 457{,}2567 \text{ kNm} \geq M_{u,X} = 342{,}0956 \text{ kNm} \quad \text{..........................} \quad (3.6.49)$$

Momen Lentur Arah Y:

$M_{u,Y}$ = 570,1594.0,7

= 399,1115 kNm

= 40,6841 tonm

d = 1200-75-22 = 1103 mm ; fc' = 25 MPa ; fy = 400 MPa................ (3.6.37)

$$R_n = \frac{Mu/\phi}{b.d^2} = \frac{399{,}1115.10^6 / 0{,}8}{1000.1103^2} = 0{,}4101 \text{ MPa} \quad \text{.....................} \quad (3.6.41)$$

$$\rho_{perlu} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.R_n.m}{fy}}\right) \quad \text{..} \quad (3.6.43)$$

$$= \frac{1}{18{,}8235}\left(1 - \sqrt{1 - \frac{2.0{,}4101.18{,}8235}{400}}\right) = 0{,}0010 < \rho_{min} = 0{,}0035$$

1,33 ρ perlu = 1,33.0,0010 = 0,0014

digunakan $\rho = 0{,}0014$

$$As_{perlu} = \rho.b.d \quad \text{..} \quad (3.6.44)$$

= 0,0014.1000.1103

= 1518,7027 mm^2

digunakan tulangan D_{22}, $A\Phi_{22} = 379{,}94$ mm^2

jarak antar tulangan :

$$S \le \frac{A_{\phi 22} \cdot 1000}{As_{perlu}} = \frac{379{,}94.1000}{1518{,}7027} = 250{,}1741 \text{ mm} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6.45)$$

digunakan D22-250

$$As_{ada} = \frac{A_{\phi 22} \cdot b}{S} = \frac{379{,}94.1000}{250} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6.46)$$

$$= 1519{,}7600 \text{ mm}^2 > As_{perlu} = 1518{,}7027 \text{ mm}^2$$

cek kapasitas lentur arah Y:

$$a = \frac{As_{ada} \cdot fy}{0{,}85.fc'.b} = \frac{1519{,}7600.400}{0{,}85.25.1000} = 28{,}6072 \text{ mm} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6.47)$$

$$Mn = As_{ada} \cdot fy\left(d - \frac{a}{2}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6.48)$$

$$= 1519{,}7600.400\left(1103 - \frac{28{,}6072}{2}\right) = 661{,}8229 \text{ kNm}$$

$$\phi Mn = 0{,}8.\ 661{,}8229$$

$$= 529{,}4583 \text{ kNm} \ge M_{u,Y} = 399{,}1115 \text{ kNm} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6.49)$$