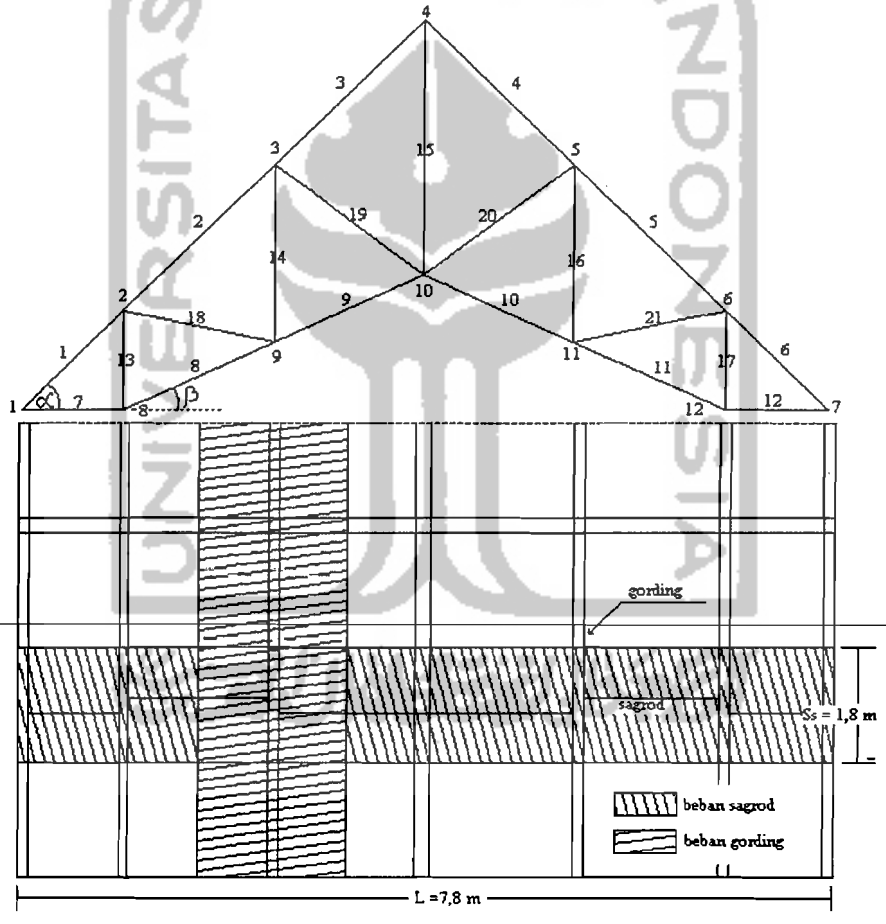
BAB IV

PERHITUNGAN KONSTRUKSI

4.1 Perencanaan Atap

Pada perencanaan ulang ini digunakan 2 macam rangka atap (kuda-kuda) yang direncanakan dengan menggunakan profil baja, di bawah ini diberikan gambar kuda-kuda 1 (KK-1) beserta perencanaannya, sedangkan untuk KK2, KK3, dan KK4 dapat dilihat pada lampiran.



Gambar 4.1 Pembebanan gording (KK-1)

4.1.1 Perencanaan Gording KK-1

1. Data-data

   Jarak antar kuda-kuda = 3.6 m

   Mutu baja profil fy = 2400 kg/cm²

   Kuat tarik fu = 3700 kg/cm²

   Mutu baut non full drat AISC A_{325x}

   fy = 2050 kg/cm²

   fu = 8250 kg/cm²

   Direncanakan untuk bangunan di darat.

2. Panjang batang

   Berikut ini contoh perencanaan kuda-kuda KK-1 dengan bentang 7,8 m.

   Adapun panjang tiap batang dapat dilihat pada tabel 4.1

   Tabel 4.1 Panjang batang rangka atap KK-1

No	No Btg	Nama Btg	Panjang Btg (m)
1	1	Batang atas	1.414
2	2	Batang atas	2.051
3	3	Batang atas	2.051
4	4	Batang atas	2.051
5	5	Batang atas	2.051
6	6	Batang atas	1.414
7	7	Batang bawah	1.000
8	8	Batang bawah	1.600
9	9	Batang bawah	1.600

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Dengan:

qc1 = nilai rata-rata qc dari ujung tiang sampai 8d diatas ujung tiang

qc2 = nilai rata-rata qc dari ujung tiang sampai 4d dibawah ujung tiang

$Qb = qb.Ab$ ..... **...(3.169)**

$fs = \frac{qb}{200}\ (kg/cm^2)$ ..... **(3.170)**

$Qs = As.fs(ton)$ ..... **(3.171)**

$Qu = Qb+Qs\ (ton)$

$Qa = \frac{Qb}{3} + \frac{Qs}{1,5}$

Dimana: Qb = tahanan ujung ultimit tiang

fs = tahanan gesek pada kulit tiang

Qs = tahanan gesek ultimit tiang

Qu = kapasitas ultimit tiang

Qa = kapasitas ijin tiang bor

3.6.2 Perencanaan Pondasi Tiang Bor

Langkah-langkah perencanaan pondasi tiang bor adalah sebagai berikut:

1. **Penentuan jenis tiang bor**

   Semua tiang harus direncanakan sebagai tiang pendek sesuai dengan ketentuan yang ada, kecuali apabila panjang tiang atau sumuran yang sesungguhnya lebih besar daripada panjang penunjangan yang diperlukan menurut perhitungan.

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2. Tiang pendek dengan ujung atas ditahan terhadap perputaran sudut

- **Menentukan tegangan tanah lateral izin (R)**

  Apabila tidak ditentukan dari hasil penyelidikan tanah, maka tegangan tanah lateral izin dapat ditentukan berdasarkan tabel berikut ini:

Jenis tanah	Tegangan Tanah Lateral Izin kg/cm^2/m' kedalaman
Kerikil bergradasi baik dan padat	6500
Lempung keras padat	6500
Pasir kasar padat	5500
Pasir kasar dan halus padat	5000
Lempung setengah keras	5000
Pasir halus padat	4000
Lanau	3500
Lempung pasiran	3500
Campuran pasir dan lanau padat	3500
Lempung lunak	1500
Campuran pasir organic lunak atau lepas dan lanau atau lumpur	0

- **Menentukan panjang penunjangan**

  Panjang penunjangan (L) yang diperlukan tiang atau sumuran untuk menyalurkan momen luar Mo dan beban horizontal Ho akibat beban kerja dari ujung atas tiang atau sumuran ke tanah sekillingnya tanpa dilampauinya tegangan tanah lateral yang diizinkan, dapat ditentukan dengan gambar dibawah ini. Untuk tampang persegi maka dikalikan dengan 1,75.

  Apabila panjang tiang sesungguhnya lebih besar dari pada panjang penunjangan maka tiang direncanakan sebagai tiang panjang.

Lanjutan tabel 4.1 Panjang batang rangka atap KK-1

10	10	Batang bawah	1.600
11	11	Batang bawah	1.600
12	12	Batang bawah	1.000
13	13	Batang vertikal	1.000
14	14	Batang vertikal	1.774
15	15	Batang vertikal	2.548
16	16	Batang vertikal	1.774
17	17	Batang vertikal	1.000
18	18	Batang diagonal	1.486
19	19	Batang diagonal	1.819
20	20	Batang diagonal	1.819
21	21	Batang diagonal	1.486

Pembebanan gording

Dicoba profil 100 X 50 X 20 X 3,2 (*Ligth Lip Channel*)

A = 7,007 cm^4

W = 5,5 kg/m'

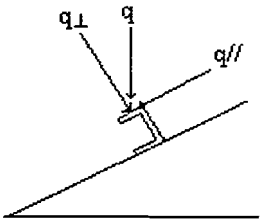
Ix =107 cm^4

Iy = 24,5 cm^4

Sx = 21,3 cm^3

Sy = 7,81 cm^3

Jarak antar gording maks = 2.051 m'



gambar 4.2 Distribusi beban pada gording

A Beban tetap

-Berat penutup atap = 50 × 2,051 = 102,55 kg/m'

-Beban gording = 5,5 kg/m'

q = 108,05kg/m'

$q\perp = q \cos \alpha = 108{,}05 \cos 45° = 76{,}403$ kg/m'

$q// = q \sin \alpha = 108{,}05 \sin 45° = 76{,}403$ kg/m'

B Beban Hidup

Sesuai dengan PPI beban hidup yang diakibatkan oleh orang bekerja sebesar 100 kg.

$p\perp = p \cos \alpha = 100 \cos 45° = 70{,}711$ kg

$p// = p \sin \alpha = 100 \sin 45° = 70{,}711$ kg

C Beban angin

Pada daerah daratan w = 25 kg/cm²

a. Angin tekan (wt) untuk $\alpha < 65°$

diketahui $\alpha = 45°$

$C_1 = 0{,}02\alpha - 0{,}4 = 0{,}02.45° - 0{,}4 = 0{,}5$

Wt = C_1.w.jarak gording = 0,5.25.2,051 =25,6375 kg/m'

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b. Angin hisap (wh)

$C_2 = -0,4$

Wh = C_2.w.jarak gording = -0,4.25.2,051 = -20,51 kg/m'

D Perhitungan momen

1. Akibat beban tetap

q⊥ = 76,403 kg/m, maka $M\perp_{bt}$ = 1/8.q⊥.b²

= 1/8.76,403.3,6²

= 123,77286 kgm' = 12377,286 kgcm'

q // = 76,403 kg/m, maka $M//_{bt}$ = 1/8. q//.Ss²

= 1/8.76,403.1,8²

= 30,943215 kgm' =3094,3215 kgcm'

2.Akibat beban hidup

P⊥ = 78,711 kg, maka $M\perp_{bh}$ = 1/4.P(.b

= 1/4.70,711.3,6

= 63,6399 kgm' = 6363,99 kgcm'

P// = 78,711 kg, maka M//bh = 1/4. P//.Ss

= 1/4.70,711. 1,8

=31,81995 kgm' = 3181,995 kgcm'

3. Akibat beban angin

Mba = 1/8. w. b²

= 1/8.25,6375.3,6²

= 41,53275 kgm' = 4153,275 kgcm'

M⊥ =12377,286 +6363,99 = 18741,276 kgcm'

M⊥=12377,286 +4153,275 = 16530,561 kgcm'

M// =3094,322+3181,995 = 6276.317 kgcm'

E Dimensi Gording

Kontrol Tegangan

$$\frac{fbx}{0,66.fy}+\frac{fby}{0,75.fy}<1$$

$$fbx=\frac{M\perp}{Sx}=\frac{18741,276}{21,3}=879,8721127\ kg/cm^2$$

$$fby=\frac{M//}{Sx}=\frac{6276,317}{7,81}=803,6257362\ kg/cm^2$$

maka, $\frac{879,872}{0,66.2400}+\frac{803,626}{0,75.2400}=1,001933\sim 1.0$ ok!

Kontrol Lendutan

$$\delta\perp=\frac{5}{384}\frac{q\perp.b^4}{E.Ix}+\frac{1}{48}\frac{p\perp.b^3}{E.Ix}=\frac{5}{384}\frac{0,76403.360^4}{2,1.10^6.107}+\frac{1}{48}\frac{0,70711.360^3}{2,1.10^6.107}$$

=0,74669 cm

$$\delta//=\frac{5}{384}\frac{q//.Ss^4}{E.Iy}+\frac{1}{48}\frac{p//.Ss^3}{E.Iy}=\frac{5}{384}\frac{0,76403.180^4}{2,1.10^6.24,5}+\frac{1}{48}\frac{0,70711.180^3}{2,1.10^6.24,5}$$

= 0,20465 cm

cek → $\sqrt{(\delta\perp)^2+(\delta//)^2}=\sqrt{0{,}74669^2+0{,}20465^2}=0{,}77423<1$ → OK

Profil *Light Lip Channel* 100 x 50 x 20 x 3,2 dapat digunakan.

4.1.2 Perencanaan Sagrod dan tierod

a. Beban Sagrod

-berat penutup atap x (½L/cosα) = 50x(½.7,8/cos 45°) = 275,772 kg/m'

-jml gording satu sisi miring x berat gording = 4 x 5,5 = 22,00 kg/m'

→ maka q = 275,772+22,00 = 297,772 kg/m'

-beban hidup akibat pekerja = 100 kg

P// = q.sin α . Ss + p.sin α =297,772.sin 45°.1,8 + 100 sin 45°

= 281,267 kg

b. Dimensi Sagrod

$$A\ sagrod = \frac{P//}{0{,}33.Fu} = \frac{281{,}267}{0{,}33.3700}$$

$$= 0{,}230\ cm^2$$

$$D = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4.0{,}230}{\pi}} = 0{,}542\ cm$$

Maka D sagrod = 0,542 + 0,3 = 0,842 cm → 1 cm

c. Dimensi Tierod

Beban tierod T = P//. cos α =281,267.cos 45° =198,886 kg

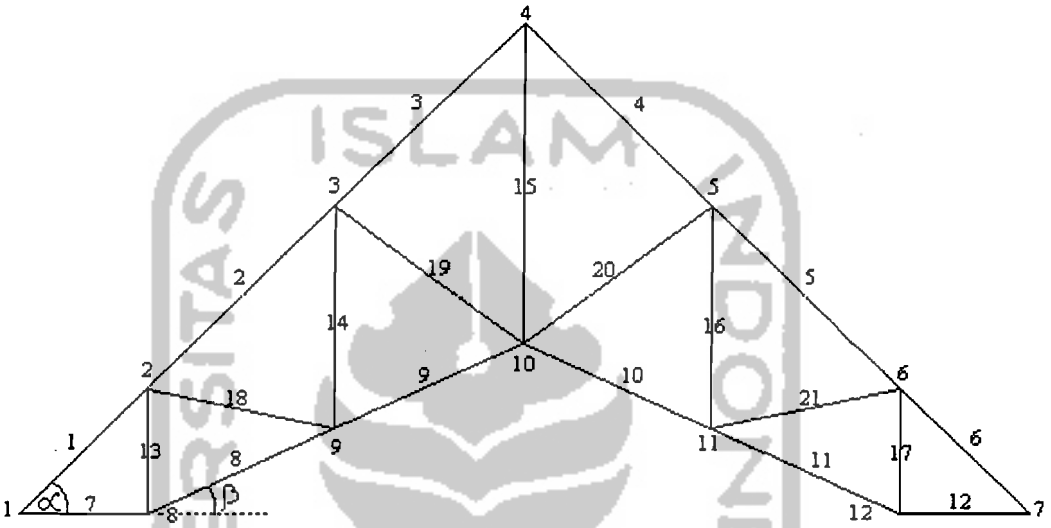
$$A\ tierod = \frac{T}{0{,}33.Fu} = \frac{198{,}886}{0{,}33.3700}$$

$$= 0{,}162888\ cm^2$$

$$D = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4.0{,}163}{\pi}} = 0{,}456 \text{ cm}$$

Maka D tierod = 0,456+0,3 = 0,756 cm → 1 cm

4.1.3 Perencanaan Kuda-Kuda KK-1



Gambar 4.3 Rencana kuda-kuda KK-1

L = 7,8 m α = 45° β = 25°

- Pembebanan

Tabel 4.2 Profil dan berat rencana kuda-kuda KK-1

Batang	Profil(mm)	Berat Profil(kg/m)	Panjang(m)	Berat(kg)
B.Atas	2L 50x50x5	7,54	11,032	83,18128
B.Bawah	2L 50x50x5	7,54	8,4	63,336
B.Vertikal	2L 50x50x5	7,54	8,096	61,04384
B.Diagonal	2L 50x50x5	7,54	6,61	49,8394

Wtot= 257,40052 kg

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- Berat total kuda –kuda = 105,828 kg
- Berat baut = 20%.berat total kuda-kuda

  = 51,4801 kg
- Berat akhir (Σ) = berat total kuda-kuda + berat baut

  = 257,4005 + 51,4801

  = 308,8806 kg
- Panjang rangka kuda-kuda (L)= 7,8 m
- $\frac{\sum}{L} = \frac{308,8806}{7,8} = 39,6001$ kg/m

a. Beban Tetap

-berat gording	= 5,5 kg/m'
-berat eternit + penggantung	= 18 kg/m'
-berat penutup atap	= 50 kg/m'
-beban hidup	= 20 kg/m'
-taksiran berat kuda-kuda	= 16,281 kg/m'

b. Beban masing-masing join

beban mati

→P1=P7

Berat gording = 5,5x3,6 = 19,800 kg

Berat penutup atap = 50x3,6x0,5.1,414 = 127,26 kg

P1 = P7 = 147,06 kg

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→P2=P6

Berat gording = 5,5x3,6 = 19,800 kg

Berat penutup atap = 50x3,6x(0,5.1,414+0,5.2,051) = 311,85 kg

P2 = P6 = 331,65 kg

→P3=P5

Berat gording = 5,5x3,6 = 19,800 kg

Berat penutup atap = 50x3,6x(0,5. 2,051 +0,5.2,051) = 369,18 kg

P3 = P5 = 388,98 kg

→P4

Berat gording = 2x5,5x3,6 = 32,472 kg

Berat penutup atap = 50x3,6x(0,5. 2,051 +0,5.2,051) = 369,18 kg

P4 = 408,78 kg

→P1'=P7'

Berat eternit + plafond = 18x3,6x0,5.1 = 32,4 kg

Berat taksiran kuda-kuda = 39,6001x0,5x1 = 19,800 kg

P1 = P7 = 52,200 kg

→P8=P12

Berat eternit + plafond = 18x3,6x(0,5.1+0,5.1,600) = 84,24 kg

Berat taksiran kuda-kuda = 39,6001x(0,5.1+0,5.1,600)= 51,480 kg

P8 = P12 = 135,720 kg

→P9=P11

Berat eternit + plafond = 18x3,6x(0,5.1,6+0,5.1,6) = 103,68 kg

Berat taksiran kuda-kuda = 39,6001x (0,5.1,6+0,5.1,6)= 63,360 kg

P9 = P11 = 167,040 kg

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→P10

Berat eternit + plafond = 8x3,6x(0,5.1,6+0,5.1,6) =103,68 kg

Berat taksiran kuda-kuda = 39,6001 x(0,5.1,6+0,5.1,6)= 63,360 kg

P10 = 167,040 kg

b.Beban hidup

P = 100 kg (beban pekerja)

Ph2= Ph3= Ph4= Ph5 = 100 kg

c.Beban angin

Muatan angin di darat = 25 kg/m^2

Koefisien angin menurut Peraturan Pembebanan Indonesia untuk Gedung 1983, untuk $\alpha < 65°$

Tekan = $C_1 = 0,02 . \alpha - 0,4$

$= 0,02 . 45 - 0,4 = 0,5$

Tarik = $C_2 = -0,4$

Beban angin yang bekerja

$W_t = C_1 \times w = 0,5 . 25 = 12,5$ kg/m^2

$W_h = C_2 \times w = -0,4 . 25 = 10$ kg/m^2

o Angin tekan

Wt_1 = 12,5.3,6.0,5.1,414 = 31,815 kg

Wt_2 = 12,5.3,6(0,5.1,414+0,5.2,051) = 77,963 kg

Wt_3 = 12,5.3,6(0,5.2,051+0,5.2,051) = 92,295 kg

Wt_4 = 12,5.3,6(0,5.2,051+0,5.2,051) = 92,295 kg

o Angin Hisap

Wh_1 = -10.3,6.0,5.1,414 = -25,452 kg

Wh_2 = -10.3,6(0,5.1,414+0,5.2,051) = -62,370 kg

Wh_3 = -10.3,6(0,5.2,051+0,5.2,051) = -73,836 kg

Wh_4 = -10.3,6(0,5.2,051+0,5.2,051) = -73,836 kg

4.1.4 Perencanaan Dimensi Batang

Sebagai contoh perhitungan dicoba batang no 13 dengan data-data sebagai berikut :

- Gaya tekan = 1696 kg
- Panjang batang = 1 m =100 cm
- Fy = 2400 kg/cm^2
- Fu = 3700 kg/cm^2
- E = 2,1x10^6 Mpa
- K = 1 (sendi-sendi)
- Syarat batang tekan

$\frac{L}{r} \leq 200$ $\quad r\,min = \frac{KL}{200} = \frac{100}{200} = 0,5$

dicoba Profil 2L 40X40X6

A = 4,48 cm^2 Atotal = 8,96 cm^2

r = 1,19 cm ≥ rmin = 0,5 cm → dipakai r = 1,19 cm

W = 3,52 kg/m

Ix=Iy = 6,33 cm^4

ix=ix = 1,19cm

e = 1,2

x = e + (0,5.tp) = 1,2 +(0,5.1)

= 1,7 cm

Ix gab = 2.Ix

= 2 . 6,33 = 12,66 cm^4

Iy gab = Ix gab+2A.x

= 12,66 + 2.4,48.1,7

= 27,892 cm^4

$$ix\ gab = \sqrt{\frac{Ixgab}{2A}} = \sqrt{\frac{12,66}{2.4,48}}$$

= 1,18867

$$iy\ gab = \sqrt{\frac{Iygab}{2A}} = \sqrt{\frac{27,892}{2.4,48}}$$

= 1,76435

dipakai r = 1,19 cm

syarat

$$\frac{KL}{r} \le Cc \text{ sehingga } \sqrt{\frac{2.\pi^2.E}{Fy}} = \frac{6400}{\sqrt{Fy}}$$

- $\frac{KL}{r} = \frac{1.100}{1,19} = 84,0336$
- $\sqrt{\frac{2.\pi^2.E}{Fy}} = \frac{6400}{\sqrt{Fy}} = 130,6395$
- $\frac{KL}{r} < Cc$, maka digunakan rumus

$$Fs = \frac{5}{3} + \frac{3.(KL/r)}{8.Cc} - \frac{1.(KL/r)^2}{8.Cc^2} = \frac{5}{3} + \frac{3.(84{,}0336)}{8.130{,}6395} - \frac{1.(84{,}0336)^3}{8.130{,}6395^3}$$

$= 1{,}8746$

$$Fa = \frac{fy}{Fs}\left(1 - 0{,}5\left(\frac{Kl/r}{Cc}\right)^2\right) = \frac{2400}{1{,}8746}\left(1 - 0{,}5\left(\frac{84{,}0336}{130{,}6395}\right)^2\right)$$

$= 1015{,}3965 \text{ kg/cm}^2$

Kontrol Kapasitas

P = Fa.Atotal > Pterjadi

= 1015,3965.8,96

= 9097,9527 kg >1696 kg → OK!!!

Profil 2L 40x40x6 dapat digunakan.

Untuk perhitungan dimensi batang yang lain dapat dilihat pada lampiran.

Tabel 4.3 Rekapitulasi perhitungan tegangan batang pada KK-1

no	l(m)	Gaya batang (kg)		Tegangan Terjadi (kg/cm^2)		Tegangan Ijin (kg/cm2)		Ket
		1,25b. tetap	b.smtr	1,25b.tetap	b.smtr	1,25b.tetap	b.smtr	
1	1.414	382.5745	323.29	89.302054	75.4645	1440	1440	OK
2	2.051	-1226.4994	-1001.15	161.80731	132.0778	370.17205	370.172505	OK
3	2.051	-1145.8482	-944.38	151.16731	124.588939	370.17205	370.172505	OK
4	2.051	-1145.8482	-1027.47	151.16731	135.55013	370.17205	370.17205	OK
5	2.051	-1226.4994	-1022.47	161.81	135.55013	370.17205	370.17205	OK
6	1.414	382.5745	317.51	89.302	73.996265	1440	1440	OK
7	1	-272.9027	-253.67	36.00299	33.4656	1020.4378	1020.4678	OK
8	1.6	-313.4411	-537.61	41.351069	70.9248	608.26801	608.26801	OK
9	1.6	941.3341	1031.61	219.73252	240.80532	1440	1440	OK

Lanjutan tabel 4.3 Rekapitulasi perhitungan tagangan batang pada KK-1

10	1.6	941.3341	853.87	219.73	199.31606	1440	1440	OK
11	1.6	-313.4411	-282.12	41.351069	37.203166	608.268	608.268	OK
12	1	-272.9027	-248.45	36.002995	32.777	1020.4378	1020.4378	OK
13	1	-1960.7791	-1617.06	258.67798	213.33245	1020.4378	1020.4378	OK
14	1.774	-569.3464	-449.15	75.111662	59.254617	497.03665	497.03665	OK
15	2.548	1076.369	978.62	251.25327	228.43604	1440	1440	OK
16	1.774	-569.3464	-497.33	75.111662	59.254617	497.03665	497.03665	OK
17	1	-1960.7791	-1507.11	198.82718	213.33245	1020.4378	1020.4378	OK
18	1.486	1167.341	907.75	272.48856	211.89309	1440	1440	OK
19	1.819	-53.8916	-118.02	7.1097	15.567	470.10207	470.10207	OK
20	1.819	-53.8916	-179.89	7.1097	23.73219	470.10207	470.10207	OK
21	1.486	1167.341	980.04	272.48856	228.76751	1440	1440	OK

4.1.5 Perencanaan Pelat Kuda-Kuda

P = 1969,7847 kg

fc' = 25 Mpa

= 250 kg/cm^2

$$\text{Aperlu} = \frac{P}{0{,}33.f'c}$$

$$= \frac{1969{,}7847}{0{,}33.250}$$

= 23,8762 cm^2

Diambil ukuran pelat = 15x20 = 300 cm^2 > Aperlu

$$Q = \frac{P}{BxL}$$

$$= \frac{1969,7847}{15x20}$$

$$= 6,5659 \text{ cm}^2$$

$$x = \frac{20-(5+1+5)}{2} = 4,5 \text{ cm}$$

$M = {}^1/_2.q.x^2 = {}^1/_2.\ 6,5659.4,5^2 = 66,4802$ kg.cm

Syarat

$$0,6.Fy = \frac{M}{{}^1/_6.1.tp^2}$$

$$tp = \sqrt{\frac{10.M}{Fy}} = \sqrt{\frac{10.66,4802}{2400}} = 0,5263 \text{ cm}$$

sehingga digunakan pelat dengan tebal 1,0 cm

dimensi pelat yang digunakan 15x20x1 cm.

4.1.6 Perencanaan Dukungan Arah Lateral

Diketahui

Lb = jarak antar gording = 2,051 m

Lc = jarak antar kuda-kuda = 3,6 m

$$L = \sqrt{Lb^2 + Lc^2}$$

$$= \sqrt{2.051^2 + 3,6^2}$$

$$= 4,14326 \text{ m}$$

Syarat : ${}^L/_r \leq 300$ sehingga

$$rmin \geq \frac{L}{300} = \frac{4{,}14326}{300}$$

$$= 1{,}38109 \text{ cm}$$

Keterangan

- $L \leq 3$ m → dipakai baja tulangan ∅ 12 mm
- $L \geq 3$ m → dipakai baja tulangan ∅ 19 mm
- $3 \text{ m} < L \leq 5$ m → dipakai baja tulangan ∅ 16 mm

→ Sehingga dipakai baja tulangan ∅ 16 mm > r min = 1,381 cm

4.1.7 Perencanaan Sambungan

Perhitungan sambungan dilakukan pada setengah bentang pada tiap joint, dengan tebal pelat sambungan = 1 cm.

Mutu pelat Fy = 2800 kg/cm^2 dan Fu = 4200 kg/cm^2.

Mutu baut non full drat dari AISC A_{325x}

Fy = 2050 kg/cm^2 Fu = 8250 kg/cm^2

Tegangan tumpu yang terjadi pada pelat

Fa tumpu = 1,2xFu pelat = 1,2x4200 = 5040 kg/cm^2

Diameter baut yang dipakai 1/2= 1,27 cm.

P tumpu = tp x Ø baut x Fa tumpu x n

= 1 x 1,27 x 5040 x n

= 6400,8.n kg

P geser = ¼.π.D^2.0,33.Fu baut.2n

= ¼ x 3,14 x (1,27)^2 x 0,33.8250 x 2 x n

= 6897,556.n kg

maka, $n = \frac{P}{6400,8}$ buah

Jarak penggunaan baut 1/2"

- Jarak baut ke tepi minimal 1,2.D

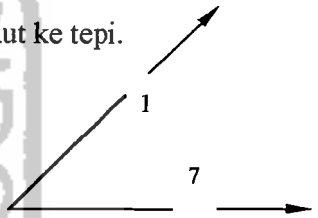
  Diambil 1,5.D = 1,5.1,27 = 1,905 cm dipakai 2 cm

- Jarak antar baut 2D s/d 7D

  Diambil 3.D = 3.1,27 = 3,81 cm dipakai 4 cm

Profil yang digunakan adalah 2L 40x40x5 dan 2L 40x40x6 dengan lebar masing-masing profil adalah 4 cm, dan memenuhi persyaratan jarak baut ke tepi.

1. Joint (1) dan joint (7)



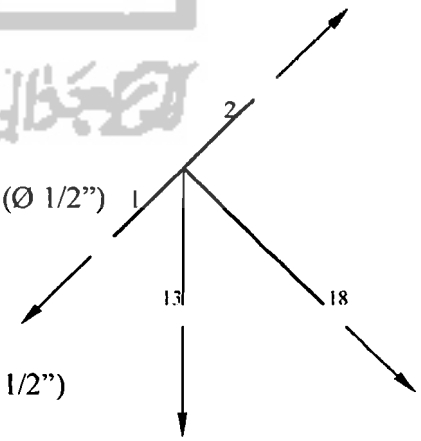
- Batang (1) = batang (6) = 381,032 kg

  $n = \frac{381,032}{6400,8} = 0,05953 \sim$ dipakai 2 buah baut (Ø 1/2")

- Batang (7) = batang (12) = 271,43 kg

  $n = \frac{271,43}{6400,8} = 0,04241 \sim$ dipakai 2 buah baut (Ø 1/2")

2. Joint (2) dan joint (6)



- Batang (1) = batang (6) = 381,032 kg

  $n = \frac{381,032}{6400,8} = 0,05953 \sim$ dipakai 2 buah baut (Ø 1/2")

- Batang (2) = batang (5) = 1063,2 kg

  $n = \frac{1063,2}{6400,8} = 0,1661 \sim$ dipakai 2 buah baut (Ø 1/2")

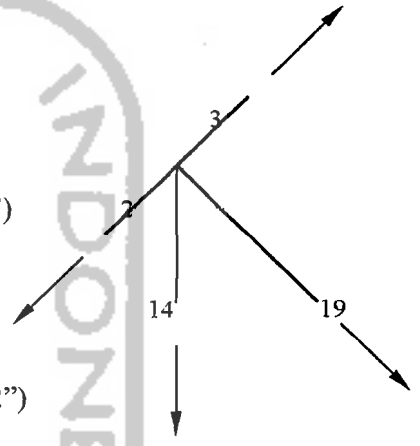
- Batang (13) = batang (17) = 1696 kg

$$n = \frac{1696}{6400,8} = 0,26497 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$

- Batang (18) = batang (21) = 1055,93 kg

$$n = \frac{1055,93}{6400,8} = 0,16497 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$

3. Joint (3) dan joint (5)



- Batang (2) = batang (5) = 1063,2 kg

$$n = \frac{1063,2}{6400,8} = 0,1661 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$

- Batang (3) = batang (4) = 1042,2 kg

$$n = \frac{1042,2}{6400,8} = 0,16282 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$

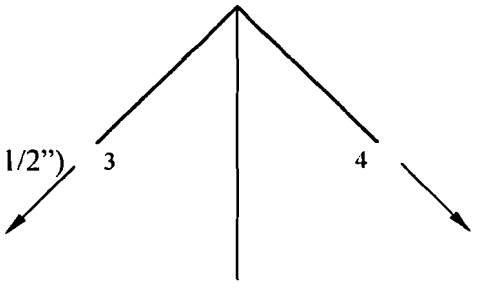
- Batang (14) = batang (16) = 493,63 kg

$$n = \frac{493,63}{6400,8} = 0,07712 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$

- Batang (19) = batang (20) = 87,64 kg

$$n = \frac{87,64}{6400,8} = 0,01369 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$

4. Joint (4)



- Batang (3) = batang (4) = 1042,2 kg

$$n = \frac{1042,2}{6400,8} = 0,16282 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$

- Batang (15) = 929,896 kg

$$n = \frac{929,896}{6400,8} = 0,14528 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$

5. Joint (8) dan joint (12)

- Batang (7) = batang (12) = 271,43 kg

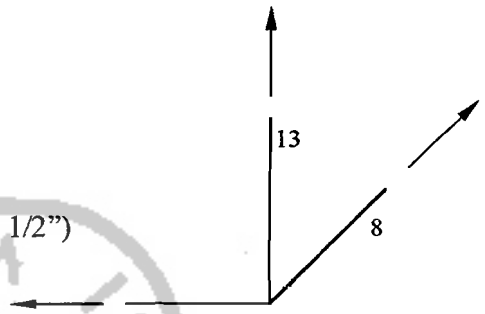
$$n = \frac{271,43}{6400,8} = 0,04241 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$

- Batang (8) = batang (11) = 535,11 kg

$$n = \frac{535,11}{6400,8} = 0,0836 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$

- Batang (13) = batang (17) = 1696 kg

$$n = \frac{1696}{6400,8} = 0,26497 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$



6. Joint (9) dan joint (11)

- Batang (8) = batang (11) = 310,12 kg

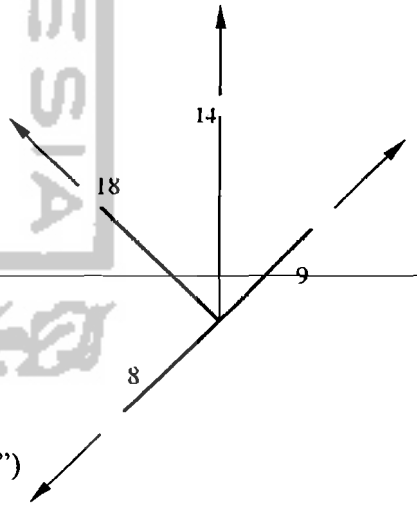
$$n = \frac{310,12}{6400,8} = 0,0836 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$

- Batang (9) = batang (10) = 824,919 kg

$$n = \frac{824,919}{6400,8} = 0,12888 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$

- Batang (18) = batang (21) = 1055,93 kg

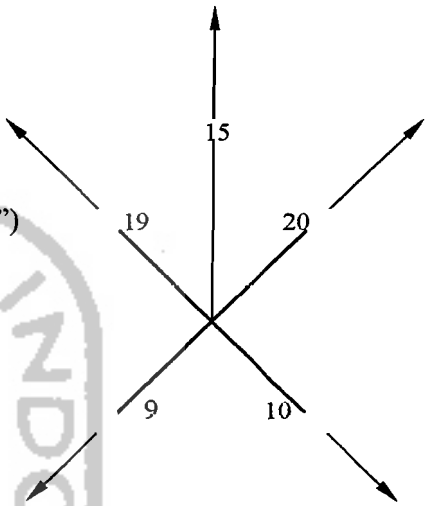
$$n = \frac{1055,93}{6400,8} = 0,16497 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$



- Batang (14) = batang (16) = 493,63 kg

$$n = \frac{493,63}{6400,8} = 0,07712 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$

7. Joint (10)



- Batang (9) = batang (10) = 824,919 kg

$$n = \frac{824,919}{6400,8} = 0,12888 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$

- Batang (19) = batang (20) = 87,64 kg

$$n = \frac{87,64}{6400,8} = 0,01369 \sim \text{dipakai 2 buah baut (Ø 1/2")}$$

- Batang (15) = 929,896 kg

$$n = \frac{929,896}{6400,8} = 0,14528 \sim \text{dipakai 3 buah baut (Ø 1/2")}$$

Tabel 4.4 Rekapitulasi perhitungan jumlah baut kuda-kuda KK-1

Join	Btg	Profil	Ø (cm)	Gaya Btg (Kg)	Jml Baut
1	1	2L 40x40x5	1.27	382.5745	3
	7	2L 40x40x5	1.27	-272.9027	3
2	1	2L 40x40x5	1.27	382.5745	3
	13	2L 40x40x6	1.27	-1960.779	3
	18	2L 40x40x5	1.27	1167.341	3
	2	2L 40x40x5	1.27	-1226.499	3
3	2	2L 40x40x5	1.27	-1226.499	3
	14	2L 40x40x5	1.27	-569.3464	3
	19	2L 40x40x5	1.27	-118.02	3
	3	2L 40x40x5	1.27	-1145.848	3
4	3	2L 40x40x5	1.27	-1145.848	3
	15	2L 40x40x5	1.27	1076.369	3
	4	2L 40x40x5	1.27	-1145.848	3
5	4	2L 40x40x5	1.27	-1145.848	3
	20	2L 40x40x5	1.27	-179.341	3
	16	2L 40x40x5	1.27	-569.3464	3
	5	2L 40x40x5	1.27	-1226.499	3

Lanjutan tabel 4.4

6	5	2L 40x40x5	1.27	-1246.499	3
	21	2L 40x40x5	1.27	1164.341	3
	17	2L 40x40x6	1.27	-1960.779	3
	6	2L 40x40x5	1.27	381.0317	3
7	6	2L 40x40x5	1.27	381.0317	3
	17	2L 40x40x6	1.27	-1960.779	3
	12	2L 40x40x5	1.27	-272.9027	3
8	7	2L 40x40x5	1.27	-272.1907	3
	13	2L 40x40x6	1.27	-1960.779	3
	8	2L 40x40x5	1.27	-537.61	3
9	8	2L 40x40x5	1.27	-537.61	3
	18	2L 40x40x5	1.27	1167.341	3
	14	2L 40x40x5	1.27	-569.3464	3
	9	2L 40x40x5	1.27	1031.51	3
10	9	2L 40x40x5	1.27	1031.51	3
	19	2L 40x40x5	1.27	-118.02	3
	15	2L 40x40x5	1.27	1076.369	3
	20	2L 40x40x5	1.27	-179.89	3
	10	2L 40x40x5	1.27	941.3341	3
11	10	2L 40x40x5	1.27	941.3341	3
	16	2L 40x40x5	1.27	-569.3464	3
	21	2L 40x40x5	1.27	1167.341	3
	11	2L 40x40x5	1.27	-313.4411	3
12	11	2L 40x40x5	1.27	-313.4411	3
	17	2L 40x40x6	1.27	-1960.779	3
	12	2L 40x40x5	1.27	-272.9027	3

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4.2 Perencanaan Pelat Lantai

4.2.1 Pembebanan Pelat Lantai Datar

- Fungsi bangunan : gelagar rumah sakit (ql) = 3 KN/m^2
- Spesifikasi bahan : mutu beton (f'c) = 25 Mpa

  mutu baja (fy) = 240 Mpa
- Perhitungan beban

  Tebal pelat lantai (h) = 120 cm

  - Beban mati (qD)

    Berat pelat beton = 0,12.24 = 2,88 KN/m^2

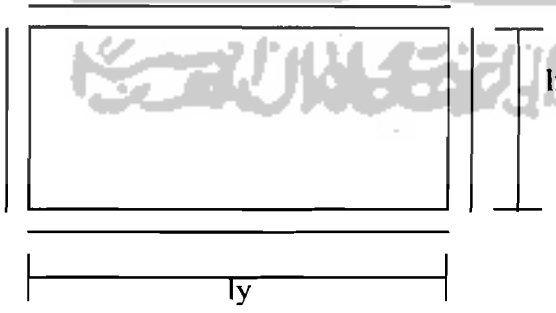
    Berat spesi (3cm) = 0,03.24 = 0,72 KN/m^2

    qD = 3,6 KN/m^2
  - Beban hidup (qL) = 3 KN/m^2

    Qu = 1,2.qD + 1,6.qL

    = 1,2.3,6 + 1,6.3

    = 9,12 KN/m^2
- Menghitung distribusi momen



Ly = 2500 mm dan Lx = 2400 mm

Ly/Lx =1,0417

Mu/ϕ = 2,0172/0,8 =2,52149 kN/m

$$Rn = \frac{Mu/\phi}{bd^2} = \frac{2,52149.10^6}{1000.95^2} = 0,27939 \text{ Mpa}$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.25} = 11,29412$$

$$\rho \text{ perlu} = \frac{1}{m}\left[1-\sqrt{1-\frac{2m.Rn}{fy}}\right] = \frac{1}{11,29412}\left[1-\sqrt{1-\frac{2.11,29412.0,27939}{240}}\right] =$$

$$= 0,001172$$

$$\rho b = \frac{0,85.f'c..\beta}{fy}\left[\frac{600}{600+fy}\right] = \frac{0,85.25.0,85}{240}\left[\frac{600}{600+240}\right] =$$

$$= 0,05376$$

ρmaks = 0,75. ρb =0,75.0,05376 = 0,04032

$$\rho min = \frac{1,4}{fy} = \frac{1,4}{240} = 0,00583$$

1,33. ρperlu = 1,33.0,001172 = 0,00156 < ρmin = 0,00583

ρ terpakai = 1,33. ρperlu = 0,00156

As perlu = ρ terpakai.b.d = 0,00156.1000.95 = 148,06722 mm^2

Dipakai tulangan pokok Ø 10 mm dengan A1Ø = 78,53982 mm^2

$$\text{Jarak tulangan : } S \leq \frac{A1Ø}{As\,perlu} = \frac{78,53982.1000}{148,06722} = 530,43351 \text{ mm}$$

Dipakai jarak (S) =200mm

$$As\ ada = \frac{A1Ø.1000}{S\,terpakai} = \frac{78,53982.1000}{200} = 392,69908\ mm^2 > As\ perlu$$

KONTROL KAPASITAS MOMEN (Mn)

$$a = \frac{As\,ada.fy}{0,85.f'c.b} = \frac{392,69908.240}{0,85.25.1000} = 4,43519\ mm^2$$

Mn = As.fy.(d-a/2) = 392,69908.240(95-4,43519 /2)

= 8,74454 kNm ≥ Mu/ϕ = 2,0172 kN/m →OK!

→dipakai tulangan Lx – Tx $P_{10\text{-}200}$

b. Perencanaan tulangan Ty

H = 120 mm

Pb = 20 mm

Øtul = 10 mm

d = h-pb-1/2. Øtul

= 120-20-1/2.10 = 95 mm

Mu Ty = 1,91214 kN/m

Mu/ϕ = 1,91214/0,8 =2,39017 kN/m

$$Rn = \frac{Mu/\phi}{bd^2} = \frac{2,39017.10^6}{1000.95^2} = 0,26484\ Mpa$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.25} = 11,29412$$

$$\rho\ perlu = \frac{1}{m}\left[1-\sqrt{1-\frac{2m.Rn}{fy}}\right] = \frac{1}{11,26412}\left[1-\sqrt{1-\frac{2.11,29412..0,26484}{240}}\right] =$$

= 0,001110

$$\rho b = \frac{0,85.f'c..\beta}{fy}\left[\frac{600}{600+fy}\right] = \frac{0,85.25.0,85}{240}\left[\frac{600}{600+240}\right] =$$

= 0,05376

93

ρmaks = 0,75. ρb =0,75.0,05376 = 0,040318

$$\rho min = \frac{1,4}{fy} = \frac{1,4}{240} = 0,00583$$

1,33. ρperlu = 1,33.0,001110 = 0,0014769 < ρmin = 0,00583

ρ terpakai = 1,33. ρperlu = 0,0014769

As perlu = ρ terpakai.b.d = 0,0014769.1000.95 = 140,3064 mm^2

Dipakai tulangan pokok Ø 10 mm dengan A1Ø = 78,53982 mm^2

$$\text{Jarak tulangan : } S \leq \frac{A1Ø}{As\,perlu} = \frac{78,53982.1000}{140,3064} = 559,7736 \text{ mm}$$

Dipakai jarak (S) =200 mm

$$\text{As ada} = \frac{A1Ø.1000}{S\,terpakai} = \frac{78,53982.1000}{200} = 392,69908 \text{ mm}^2 > \text{As perlu}$$

KONTROL KAPASITAS MOMEN (Mn)

$$a = \frac{As\,ada.fy}{0,85.f'c.b} = \frac{392,69908.240}{0,85.25.1000} = 4,43519 \text{ mm}$$

Mn = As.fy.(d-a/2) = 392,69908.240(95-4,435189/2)

= 8,74454 kNm ≥ Mu/ϕ = 2,39017 kN/m →OK!

→dipakai tulangan Ty P_{10-200}

c. Perencanaan tulangan Ly

H = 120 mm

Pb = 20 mm

Øtul = 10 mm

d = h-pb-1,5 Øtul

= 120-20-1,5.10 = 85 mm

Mu Ly = 1,91214 kN/m

Mu/ϕ = 1,91214/0,8 =2,39017 kN/m

$$Rn = \frac{Mu/\phi}{bd^2} = \frac{2,39017.10^6}{1000.85^2} = 0,33082 \text{ Mpa}$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.25} = 11,29412$$

$$\rho \text{ perlu} = \frac{1}{m}\left[1-\sqrt{1-\frac{2m.Rn}{fy}}\right] = \frac{1}{11,29412}\left[1-\sqrt{1-\frac{2.11,29412..0,33082}{240}}\right] =$$

$$= 0,001389$$

$$\rho b = \frac{0,85.f'c..\beta}{fy}\left[\frac{600}{600+fy}\right] = \frac{0,85.25.0,85}{240}\left[\frac{600}{600+240}\right] =$$

$$= 0,05376$$

ρmaks = 0,75. ρb =0,75.0,05376 = 0,04032

$$\rho min = \frac{1,4}{fy} = \frac{1,4}{240} = 0,00583$$

1,33. ρperlu = 1,33.0,001389 = 0,001848 < ρmin = 0,00583

ρ terpakai = 1,33. ρperlu = 0,001848

As perlu = ρ terpakai.b.d = 0,001848.10000.85 = 157,06192 mm^2

Dipakai tulangan pokok Ø 10mm dengan A1Ø = 78,53982 mm^2

$$\text{Jarak tulangan : } S \leq \frac{A1Ø}{As\,perlu} = \frac{78,53982.1000}{157,06192} = 500,05639 \text{ mm}$$

Dipakai jarak (S) = 200 mm

$$As\ ada = \frac{A1Ø.1000}{S\,terpakai} = \frac{78,53982.1000}{200} = 392,69908 \text{ mm}^2 > As\ perlu$$

KONTROL KAPASITAS MOMEN (Mn)

$$a = \frac{As\,ada.fy}{0,85.f'c.b} = \frac{392,69908.240}{0,85.25.1000} = 4,43519 \text{ mm}$$

Mn = As.fy.(d-a/2) = 392,69908.240(85-4,43519/2)

= 7,80206 kNm ≥ Mu/ϕ = 2,39017 kN/m →OK!

→dipakai tulangan Ly $P_{10\text{-}200}$

4.2.3 Pembebanan Pelat Lantai Miring

- Fungsi bangunan : gelagar rumah sakit (ql) = 3 KN/m^2
- Spesifikasi bahan : mutu beton (f'c) = 25 Mpa
  mutu baja (fy) = 240 Mpa
- Perhitungan beban

  Tebal pelat lantai (h) = 120 cm

  - Beban mati (qD)

    Berat pelat beton = 0,12.24 = 2,88 KN/m^2

    Berat spesi (3cm) = 3.0,24 = 0,72 KN/m^2

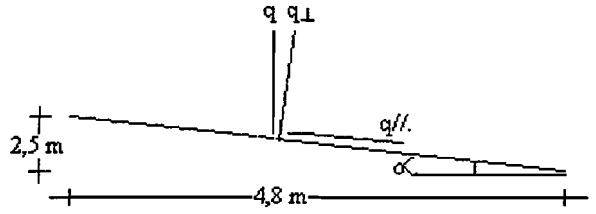
    qD = 3,6 KN/m^2

  - Beban hidup (qL) = 3 KN/m^2

    Qu = 1,2.qD + 1,6.qL

    = 1,2.3,6 + 1,6.3

    = 9,12 KN/m^2



Gambar 4.4 Distribusi beban pada pelat lantai miring

α = arc tg (0,86/4,8)

= 10,157°

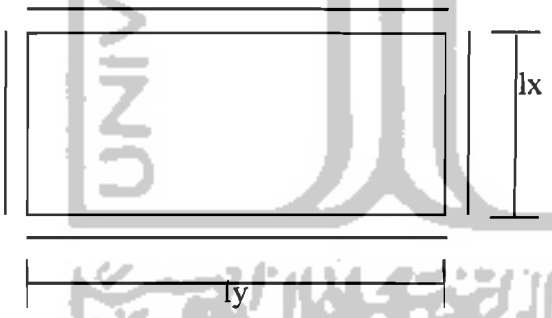
Qu⊥ = Qu.cos α = 9,12.cos 10,157°

= 8,977 KN/m^2

Qu// = Qu.sin α = 9,12.sin 10,157°

= 1,608 KN/m^2

- Menghitung distribusi momen



Ly = 4800 mm dan Lx = 2500 mm

Ly/Lx = 1,92

dari tabel 13.3.2 PBI 1997 (tumpuan tepi dianggap jepit elastis)

didapat clx = 61,2 ctx = 61,2

cly = 35 cty = 35

97

Mulx = 0,001.qu⊥.lx^2.clx = 0.001.8,977.2,5^2.61,2 = 3,43373 kNm

Mutx = -0,001.qu⊥.lx^2.ctx = -0.001.8,977.2,5^2.61,2 = -3,43373 kNm

Muly = 0,001.qu⊥.lx^2.cly = 0.001.8,977.2,5^2.35 = 1,96373 kNm

Mutx = -0,001.qu⊥.lx^2.cyx = -0.001.8,977.2,5^2.35 = -1,96373 kNm

4.2.4 Perhitungan Tulangan Pelat Lantai Miring

a. Perencanaan tulangan Lx = Tx

Perkiraan tebal pelat:

$\beta = \frac{7,2}{3} = 2,4$

$hmin = \frac{ln(0,8 + fy/1500)}{36 + 9.\beta}$

$= \frac{ln(0,8 + fy/1500)}{36 + 9.\beta}$

$= \frac{3000.(0,8 + 240/1500)}{36 + 9.2,4} = 50$ mm = 5 cm < 12 cm, maka

h pakai = 120 mm

Pb = 20 mm

Øtul = 10 mm

d = h-pb-1/2. Øtul

= 120-20-1/2.10 = 95 mm

Mu Lx = 3,43373 kN/m

Mu/ϕ = 3,43373/0,8 = 4,29216 kN/m

$Rn = \frac{Mu/\phi}{bd^2} = \frac{4,29216.10^6}{1000.96^2} = 0,47559$ Mpa

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.25} = 11,29412$$

$$\rho \text{ perlu} = \frac{1}{m}\left[1-\sqrt{1-\frac{2m.Rn}{fy}}\right] = \frac{1}{11,29412}\left[1-\sqrt{1-\frac{2.11,29412.0,47559}{240}}\right] =$$

$$= 0,002004$$

$$\rho b = \frac{0,85.f'c..\beta}{fy}\left[\frac{600}{600+fy}\right] = \frac{0,85.25.0,85}{240}\left[\frac{600}{600+240}\right] =$$

$$= 0,05376$$

$$\rho maks = 0,75.\ \rho b = 0,75.0,05376 = 0,040318$$

$$\rho min = \frac{1,4}{fy} = \frac{1,4}{240} = 0,00583$$

$$1,33.\ \rho perlu = 1,33.0,002004 = 0,00266 < \rho min = 0,00583$$

$$\rho \text{ terpakai} = 1,33.\ \rho perlu = 0,00266$$

$$\text{As perlu} = \rho \text{ terpakai.b.d} = 0,00266.1000.95 = 253,24248 \text{ mm}^2$$

Dipakai tulangan pokok Ø 10 mm dengan A1Ø = 78,53982 mm^2

$$\text{Jarak tulangan : } S \leq \frac{A1Ø}{\text{As perlu}} = \frac{78,53982.1000}{253,24248} = 310,13682 \text{ mm}$$

Dipakai jarak (S) =200 mm

$$\text{As ada} = \frac{A1Ø.1000}{\text{S terpakai}} = \frac{78,53982.1000}{200} = 392,69908 \text{ mm}^2 > \text{As perlu}$$

KONTROL KAPASITAS MOMEN (Mn)

$$a = \frac{\text{Asada.fy}}{0,85.f'c.b} = \frac{392,69908.240}{0,85.25.1000} = 4,43519 \text{ mm}$$

Mn = As.fy.(d-a/2) = 392,69908.240(95-4,43519/2)

= 8,74454 kNm ≥ Mu/ϕ = 4,293 kN/m →OK!

→dipakai tulangan Lx = Tx $P_{10\text{-}200}$

b. Perencanaan tulangan Ty

H = 120 mm

Pb = 20 mm

Øtul = 10 mm

d = h-pb-1/2. Øtul

= 120-20-1/2.10 = 95 mm

Mu Ty = 1,96373 kN/m

Mu/ϕ = 1,96373/0,8 = 2,45457 kN/m

$$Rn = \frac{Mu/\phi}{bd^2} = \frac{2,45457.10^6}{1000.95^2} = 0,27199 \text{ Mpa}$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.25} = 11,29412$$

$$\rho \text{ perlu} = \frac{1}{m}\left[1-\sqrt{1-\frac{2m.Rn}{fy}}\right] = \frac{1}{11,29412}\left[1-\sqrt{1-\frac{2.11,29412.0,27199}{240}}\right] =$$

= 0,00114

$$\rho b = \frac{0,85.f'c.\beta}{fy}\left[\frac{600}{600+fy}\right] = \frac{0,85.25.0,85}{240}\left[\frac{600}{600+240}\right] =$$

= 0,05376

ρmaks = 0,75. ρb =0,75.0,05376 = 0,040318

$$\rho min = \frac{1,4}{fy} = \frac{1,4}{240} = 0,00583$$

1,33. ρperlu = 1,33.0,00114 = 0,001517 < ρmin = 0,00583

ρ terpakai = 1,33. ρperlu = 0,001517

As perlu = ρ terpakai.b.d = 0,001517.1000.95 = 144,11728 mm

Dipakai tulangan pokok Ø 10 mm dengan A1Ø = 78,53982 mm^2

Jarak tulangan : $S \leq \frac{A1Ø}{As\,perlu} = \frac{78,53982.1000}{144,11728} = 544,97155$ mm

Dipakai jarak (S) =200 mm

As ada = $\frac{A1Ø.1000}{S\,terpakai} = \frac{78,53982.1000}{200} = 392,69908\ mm^2 >$ As perlu

KONTROL KAPASITAS MOMEN (Mn)

a = $\frac{As\,ada.fy}{0,85.f'c.b} = \frac{392,69908.240}{0,85.25.1000} = 4,43519$ mm

Mn = As.fy.(d-a/2) = 392,69908.240(95-4,43519/2)

= 8,74454 kNm ≥ Mu/ϕ = 2,45467 kN/m →OK!

→dipakai tulangan Ty $P_{10\text{-}200}$

c. Perencanaan tulangan Ly

H = 120 mm

Pb = 20 mm

Øtul = 10 mm

d = h-pb-1,5 Øtul

= 120-20-1,5.10 = 85 mm

Mu Ly = 1,96373 kN/m

Mu/ϕ = 1,96373/0,8 =2,45467 kN/m

$$Rn = \frac{Mu/\phi}{bd^2} = \frac{2,45467.10^6}{1000.85^2} = 0,33975 \text{ Mpa}$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.25} = 11,29412$$

$$\rho \text{ perlu} = \frac{1}{m}\left[1-\sqrt{1-\frac{2m.Rn}{fy}}\right] = \frac{I}{11,29412}\left[1-\sqrt{1-\frac{2.11,29412.0,33975}{240}}\right] =$$

$$= 0,001427$$

$$\rho b = \frac{0,85.f'c.\beta}{fy}\left[\frac{600}{600+fy}\right] = \frac{0,85.25.0,85}{240}\left[\frac{600}{600+240}\right] =$$

$$= 0,05376$$

$$\rho maks = 0,75.\ \rho b = 0,75.0,05376 = 0,04032$$

$$\rho min = \frac{1,4}{fy} = \frac{1,4}{240} = 0,00583$$

$1,33.\ \rho perlu = 1,33.0,001427 = 0,001898 < \rho min = 0,00583$

ρ terpakai $= 1,33.\ \rho perlu = 0,001898$

As perlu $= \rho$ terpakai.b.d $= 0,001898.1000.85 = 161,33496 \text{ mm}^2$

Dipakai tulangan pokok Ø 10 mm dengan A1Ø = 78,53982 mm^2

Jarak tulangan : $S \leq \frac{A1Ø}{As\,perlu} = \frac{50,265.1000}{155,76} = 486,81214 \text{ mm}^2$

Dipakai jarak (S) = 200 mm

$$As\ ada = \frac{A1Ø.1000}{S\,terpakai} = \frac{78,53982.1000}{200} = 392,69908 \text{ mm}^2 > As\ perlu$$

KONTROL KAPASITAS MOMEN (Mn)

$$a = \frac{As\,ada.fy}{0,85.f'c.b} = \frac{392,69908.240}{0,85.25.1000} = 4,43519 \text{ mm}^2$$

Mn = As.fy.(d-a/2) = 392,69908.240(85-4,43519/2)

= 7,80206 kNm ≥ Mu/ϕ = 2,45457 kN/m →OK!

→dipakai tulangan Ly $P_{10\text{-}200}$

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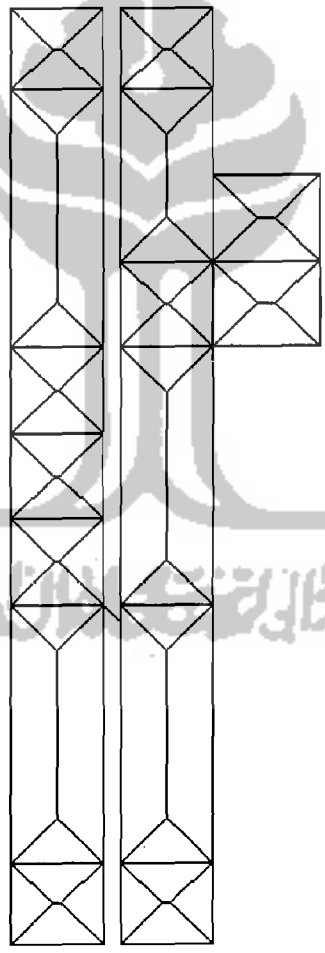
4.3 Perencanaan Struktur

Pada perencanaan ulang gedung Ramp RSUP Dr Sardjito ini, perencanaan portal dianalisis dengan SAP2000 dengan analisis struktur tiga (3) Dimensi. Adapun distribusi dan pola-pola pembebanan pada struktur bangunan fakultas hukum adalah sebagai berikut :

4.3.1 Distribusi Beban Pelat Pada Balok

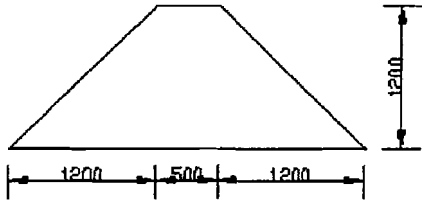
- Beban mati (qD) pelat = 3,6 kN/m^2
- Beban hidup (qL) pelat = 3 kN/m^2



Gambar 4.5 Distribusi beban pelat

4.3.2 Pola Distribusi Pembebanan Balok

1 Tipe 1



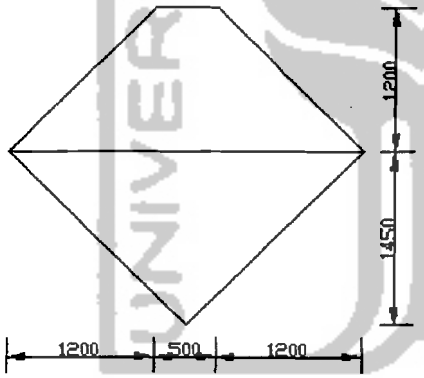
Gambar 4.6 Pola Pembebanan 1

t = 1,2 m

qD = 1,2.3,6 = 4,32 kN/m'

qL = 1,2.3 = 3,6 kN/m'

2 Tipe 2



Gambar 4.7 Pola Pembebanan 1

t_1 = 1,2 m

qD = 1,2.3,6 = 4,32 kN/m'

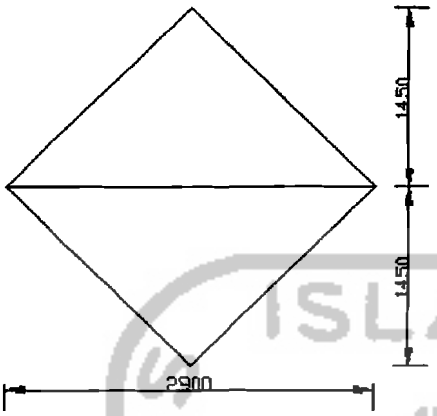
qL = 1,2.3 = 3,6 kN/m'

t_2 = 1,45 m

qD = 1,45.3,6 = 5,23 kN/m'

qL = 1,45.3 = 4,35 kN/m'

3 Tipe 3



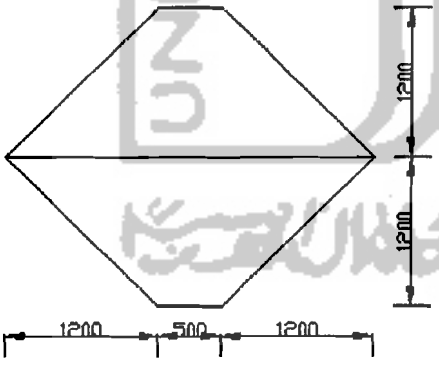
Gambar 4.8 Pola Pembebanan 1

$t_1 = t_2 = 1,45$ m

$qD_1 = qD_2 = 1,45.3,6$ = 5,23 kN/m'

$qL_1 = qL_2 = 1,45.3$ = 4,35 kN/m'

4 Tipe 4



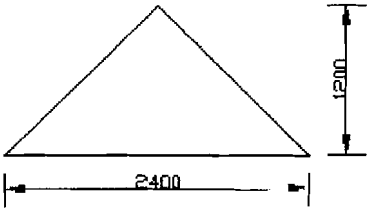
Gambar 4.9 Pola Pembebanan 1

$t_1 = t_2 = 1,2$ m

$qD_1 = qD_2 = 1,2.3,6 = 4,32$ kN/m'

$qL_1 = qL_2 = 1,2.3$ = 3,6 kN/m'

5 Tipe 5



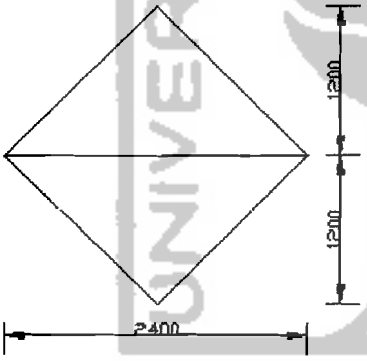
Gambar 4.10 Pola Pembebanan 1

t = 1,2 m

qD = 1,2.3,6 = 4,32 kN/m'

qL = 1,2.3 = 3,6 kN/m'

6 Tipe 6



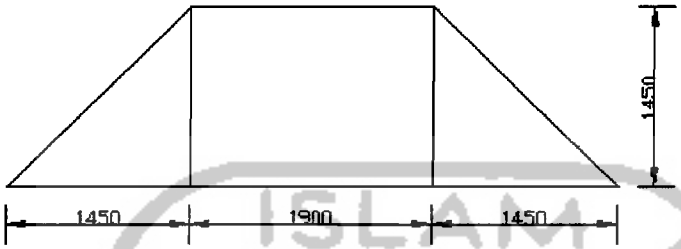
Gambar 4.11 Pola Pembebanan 1

$t_1 - t_2 - 1,2$ m

$qD_1 = qD_2 = 1,2.3,6 = 4,32$ kN/m'

$qL_1 = qL_2 = 1,2.3 = 3,6$ kN/m'

7 Tipe 7



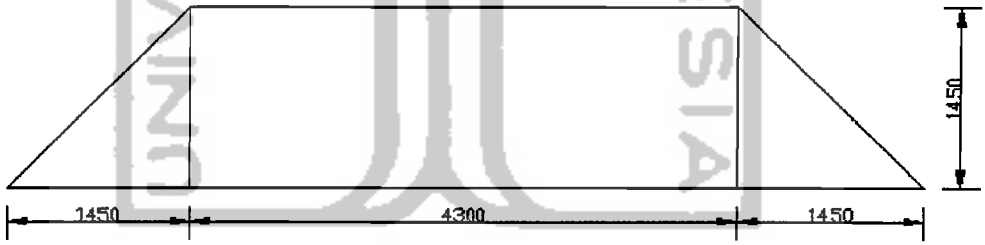
Gambar 4.12 Pola Pembebanan 1

t = 1,45 m

qD = 1,45.3,6 = 5,23 kN/m'

qL = 1,45.3 = 4,35 kN/m'

8 Tipe 8



Gambar 4.13 Pola Pembebanan 1

t = 1,45 m

qD = 1,45.3,6 = 5,23 kN/m'

qL = 1,45.3 = 4,35 kN/m'

4.4 Perencanaan Balok

4.4.1 Perencanaan Tulangan Lentur Balok

Desain tulangan lentur balok

A. Momen Rencana Balok

Momen rencana balok diambil yang terbesar dari hasil perhitungan analisa struktur dengan menggunakan SAP 2000.

Berikut contoh perhitungan balok portal as A bentang 2-4 lantai 4. Untuk perhitungan balok lainnya dapat dilihat pada lampiran .....

B. Perencanaan Tulangan Tumpuan

Mu maks = 137,52 kNm

Mn $= \frac{Mu}{\Phi} = 171,9$ kNm

d = h-pb-Øsengkang-0,5. Øtul memanjang

= 500-40-10-0,5.19 = 440,5 mm

ρ balance $= \frac{0,85.f'c.\beta}{fy}\left[\frac{600}{600+fy}\right]$

$= \frac{0,85.25.0,85}{400}\left[\frac{600}{600+400}\right] = 0,02709$

ρ maks = 0,75. ρ balance

= 0,75.0,02709 = 0,02032

ρ rencana = 0,5. ρ maks

= 0,5.0,02032 =0,01016

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ρ min $= \frac{1,4}{fy}$

$= \frac{1,4}{400} = 0,0035$

m $= \frac{fy}{0,85.f'c}$

$= \frac{400}{0,85.25} = 18,82353$

Rn $= \rho.fy\left(1-\frac{1}{2}.\rho.m\right)$

$= 0,010156.400\left(1-\frac{1}{2}.0,01016.18,8235\right)$

$= 3,67544$ Mpa

$b.d_{perlu}^2 = \frac{Mn}{Rn}$

$d_{perlu} = \sqrt{\frac{Mn.10^6}{Rn.b}} = \sqrt{\frac{171,9.10^6}{3,67544.300}}$

$= 394,8415$ mm < d ada = 440,5 mm

→ maka digunakan tulangan sebelah

Rn baru $= \frac{Mu/\Phi}{b.d^2}$

$= \frac{(137,52/0,8).10^6}{300.440,5^2} = 2,953$ Mpa

ρ baru $= \frac{2,953}{3,67544} \cdot 0,01016 = 0,00816$

As perlu = ρ baru.b.d

= 0,00816.300.440,5

= 1134,752mm^2

digunakan D19, maka A1Ø = 283,6429 mm^2

jumlah tulangan terpakai = $\frac{1134,752}{283,6429}$ = 4,00064 batang

→ maka digunakan 4D19.

C. Perencanaan Tulangan Lapangan

Mu maks = 78,28/ kNm

Mn = $\frac{Mu}{\Phi}$ = 97,85 kNm

d = h-pb-Øsengkang-0,5. Øtul memanjang

= 500-40-10-0,5.19 = 440,5 mm

ρ balance = $\frac{0,85.f'c.\beta}{fy}\left[\frac{600}{600+fy}\right]$

= $\frac{0,85.25.0,85}{400}\left[\frac{600}{600+400}\right]$ = 0,02709

ρ maks = 0,75. ρ balance

= 0,75.0,02709 = 0,02032

ρ rencana = 0,5. ρ maks

= 0,5.0,02032 = 0,01016

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ρ min $= \frac{1,4}{fy}$

$= \frac{1,4}{400} = 0,0035$

m $= \frac{fy}{0,85.f'c}$

$= \frac{400}{0,85.25} = 18,82353$

Rn $= \rho.fy\left(1-\frac{1}{2}.\rho.m\right)$

$= 0,01016.400\left(1-\frac{1}{2}.0,01016.18,82353\right)$

$= 3,67544$ Mpa

$b.d_{perlu}^2 = \frac{Mn}{Rn}$

$d_{perlu} = \sqrt{\frac{Mn.10^6}{Rn.b}} = \sqrt{\frac{97,85.10^6}{3,67544.300}}$

$= 297,8964$ mm < d ada = 440,5 mm

→ maka digunakan tulangan sebelah

Rn baru $= \frac{Mu/\Phi}{b.d^2}$

$= \frac{(78,28/0,8).10^6}{300.440,5^2} = 1,68092$ Mpa

ρ baru $= \frac{1,68092}{3,67544} \cdot 0,01016 = 0,00465$

As perlu = ρ baru.b.d

= 0,00465.300.440,5

= 614,0537 mm^2

digunakan D19, maka A1Ø = 283,6429 mm^2

jumlah tulangan terpakai = $\frac{614,0537}{283,6429}$ = 2,16488 batang

→ maka digunakan 3D19.

MOMEN NOMINAL AKTUAL BALOK NEGATIF

ρ aktual = $\frac{1134,5716}{300.500}$

= 0,00756

Rn = ρ.fy.(1-0.5. ρm)

= 0,00756.400.(1-0,5.0,00756. 18,8235)

= 2,8088 Mpa

Mnak⁻ = $2,8088.300.500^2.10^{-6}$

= 210,66 KNm.

MOMEN NOMINAL AKTUAL BALOK POSITIF

ρ aktual = $\frac{850,6429}{300.500}$

= 0,00567

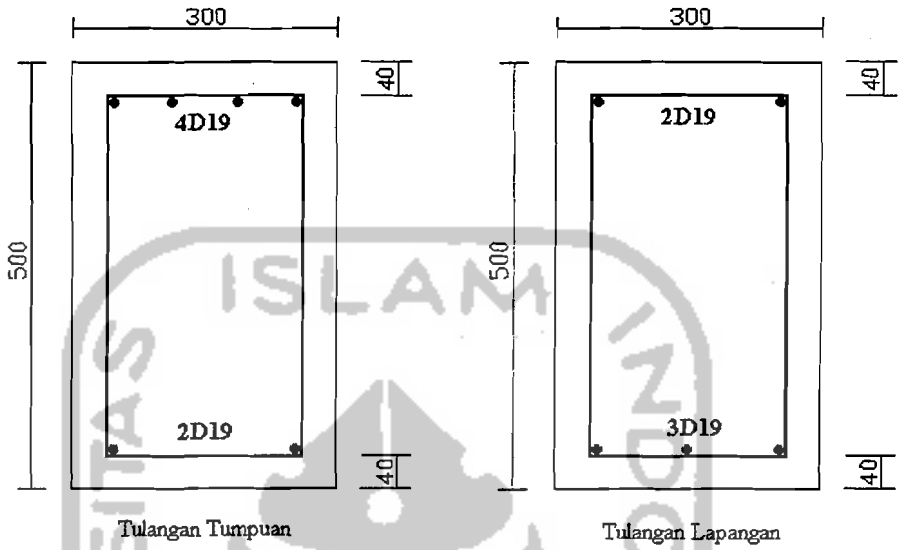
Rn = ρ.fy.(1-0.5. ρm)

= 0,00567.400.(1-0,5.0,00567. 18,8235)

= 2,14696 Mpa

Mnak$^-$ = 2,14696.300.500^2.10^{-6}

= 161,022 KNm.



Gambar 4.14 Penulangan balok As A/2-4 lantai 4

4.4.2 Perencanaan Tulangan Geser

Vu = 107,5 kN

$$Vc = \frac{1}{6}\cdot\sqrt{fc'}\cdot bw\cdot d = \frac{1}{6}\cdot\sqrt{25}\cdot 300\cdot 440,5$$

= 110625 N = 110,625 kN

θVc – 0,6.110,625 = 66,075 kN

0,5.θ.Vc= 0,5.66,075 = 33,0375 kN

$$Vs_1 = \frac{1}{6}\cdot\sqrt{fc'}\cdot bw\cdot d = \frac{1}{6}\cdot\sqrt{25}\cdot 300\cdot 440,5$$

= 22025 N = 220,25 kN

$Vs_2 = \frac{2}{3} \cdot \sqrt{fc'} \cdot bw \cdot d = \frac{2}{3} \cdot \sqrt{25} \cdot 300 \cdot 440,5$

$= 440500$ KN $= 440,5$ kN

$\theta(Vc+Vs_1) = 0,6(110,125+220,25) = 198,225$ kN

$\theta(Vc+Vs_2) = 0,6(110,125+440,5) = 330,375$ kN

gaya geser rencana pada jarak d dari perletakan

Vu kritis $= \frac{3,6-(0,4405+0,15)}{3,6}.107,5$

$= 89,867$ kN

→ $\theta Vc = 66,075$ kN < Vu kritis $= 89,867$ kN < $\theta(Vc+Vs_1) = 198,225$ kN

Berarti ukuran penampang dapat digunakan tetapi dibutuhkan tulangan geser.

Titik dimana Vu = $\theta Vc = 66,075$ kN

x1 $= (66,075/107,5)3600$

$= 2212,7442$ mm

Titik dimana Vu = $0,5.\theta Vc = 33,0375$ kN

x1 $= (33,0375/107,5)3600$

$= 1106,3721$ mm

- Daerah 1

  $\theta.Vs = Vu-\theta.Vc$

  $= 198,225 - 66,075 = 132,15$ KN

  $Vs = 132,15/0,6 = 220,25$ kN

$$s = \frac{Av.fy.d}{Vs} = \frac{(2.0{,}25.\pi.8^2).240.442{,}5}{220{,}25.10^3}$$

$$= 75{,}4286 \text{ mm}$$

$$s = \frac{d}{4} = \frac{442{,}5}{4}$$

$$= 110{,}625 \text{ mm}$$

$$s = 600 \text{ mm}$$

s pakai = 70 mm

→ P8-70

- Daerah 2

dipakai sengkang geser minimum

$$s = \frac{3.Av.fy}{b} = \frac{3.(2.0{,}25.\pi.8^2).240}{300.}$$

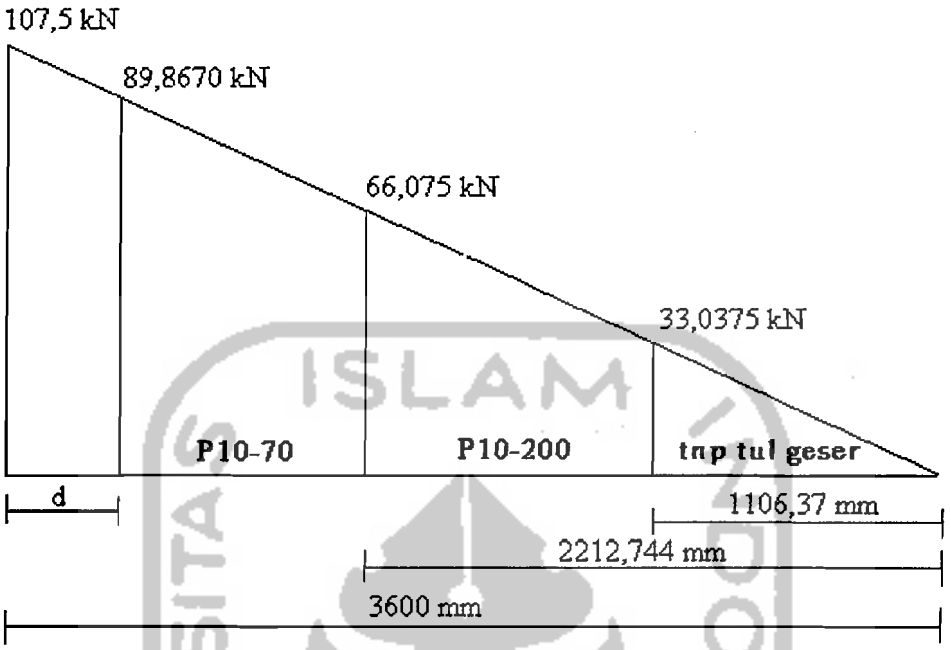
$$= 377{,}1429 \text{ mm}$$

$$s = \frac{d}{2} = \frac{442{,}5}{2}$$

$$= 221{,}25 \text{ mm}$$

s pakai = 220 mm

→P8-200



Gamber 4.15 Diagram tulangan geser balok as A / 2-4 lantai 4

4.5 Perencanaan Kolom

4.5.1 Perhitungan Grafik Mn-Pn

Berikut ini adalah contoh perhitungan Mn-Pn untuk Ast = 1% Ag.

1. Batang Desak Aksial

Ag = b.h

= 400.400 = 160000 mm^2

Ast = 1%.Ag

= 1%.160000 = 1600 mm^2

Po = (0,85.fc'.(Ag-Ast) + Ast.fy).10^{-3}

= (0,85.25.(160000-1600) + 1600.400). 10^{-3}

= 4006 KN

2. Kondisi seimbang

$$xb = \frac{600}{600 + fy}.d$$

$$= \frac{600}{600 + 400}.350 = 210 \text{ mm}$$

$$fs' = \frac{xb - d'}{xb}.600$$

$$= \frac{210 - 50}{210}.600 = 457 \text{ Mpa} > fy = 400 \text{ Mpa}$$

→fs' pakai = 400 Mpa

ab = β1.xb

= 0,85.210 = 178,5 mm

Cc = 0,85.fc'.b.ab.10^{-3}

= 0,85.25.400.178,5.10^{-3} = 1517,250 KN

As = As' = ½.Ast = ½.1600 = 800 mm^2

Cs = As'.(fs'-0,85.fc').10^{-3}

= 800.(400-0,85.25).10^{-3} = 303 KN

Ts = As.fy.10^{-3}

= 800.400.10^{-3} = 320KN

Mnb = Cc.(h/2-a/2) + Cs.(h/2-d') + Ts.(d-h/2)

= 1517,250.(400/2-178,5/2) + 303.(400/2-50) + 320.(350-400/2)

= 261,485 KNm

Pnb = Cc + Cs – Ts

= 1517,250 + 303 – 320 = 1500,250 KN

$eb = \frac{Mnb}{Pnb} = \frac{261,485}{1500,250} = 0,1743$ m

3. Kondisi Patah Desak

- x = 180%.xb

  = 1,8.210 = 378 mm

  $fs' = \frac{x - d'}{x}.600$

  $= \frac{378 - 50}{378}.600 = 520,635$ Mpa > fy = 400 Mpa

  →fs' pakai = 400 Mpa

  $fs = \frac{d - x}{x} \cdot 600$

  $= \frac{350 - 378}{378} \cdot 600 = -44,444$ Mpa < fy = 400 Mpa

  →fs pakai = -44,444 MPa

  a = β1.x

  = 0,85.378 = 321,300 mm

  $Cc = 0,85.fc'.b.a.10^{-3}$

  $= 0,85.400.400.321,300.10^{-3} = 2731,050$ KN

  $Cs = As'.(fs'-0,85.fc').10^{-3}$

  $= 800.(400-0,85.25).10^{-3} = 303$ KN

  $Ts = As.fs.10^{-3}$

  $= 800.-44,444.10^{-3} = -35,556$ KN

  Mn = Cc.(h/2-a/2) + Cs.(h/2-d') + Ts.(d-h/2)

  = 2731,05.(400/2-321,3/2) + 303.(400/2-50) – 35,556.(350-400/2)

  = 147,583 KNm

Pn = Cc + Cs – Ts

= 2731,050 + 303 + 35,556 = 3069,606 kn

$e = \frac{Mn}{Pn} = \frac{147,583}{3069,606} = 0,048$ m

- x = 160%.xb

  = 1,6.210 = 336 mm

  $fs' = \frac{b - d'}{b}.600$

  $= \frac{336 - 50}{336}.600 = 510,714$ Mpa > fy = 400 Mpa

  →fs' pakai = 400 Mpa

  $fs = \frac{d - x}{x} \cdot 600$

  $= \frac{350 - 336}{350} \cdot 600 = 25$ Mpa < fy = 400 Mpa

  →fs pakai = 25 MPa

  $a = \beta 1.x$

  = 0,85.336 = 285,6 mm

  $Cc = 0,85.fc'.b.a.10^{-3}$

  $= 0,85.25.400.285,6.10^{-3} = 2427,6$ KN

  $Cs = As'.(fs'-0,85.fc').10^{-3}$

  $= 800.(400-0,85.25).10^{-3} = 303$ KN

  $Ts = As.fs.10^{-3}$

  $= 800.25.10^{-3} = 20$ KN

  Mn = Cc.(h/2-a/2) + Cs.(h/2-d') + Ts.(d-h/2)

  = 2427,6.(400/2-285,6/2) + 303.(400/2-50) + 20.(350-400/2)

  = 187,309 KNm

  Pn = Cc + Cs – Ts

  = 2427,6 + 303 – 20 = 2710,6 KN

e $= \frac{Mn}{Pn} = \frac{187,309}{2710,6} = 0,069$ m

- x = 140%.xb

  = 1,4.210 = 294 mm

  $fs' = \frac{x - d'}{x}.600$

  $= \frac{294 - 50}{294}.600 = 497,959$ Mpa > fy = 400 Mpa

  → fs' pakai = 400 Mpa

  $fs = \frac{d - x}{x}.600$

  $= \frac{350 - 294}{350}.600 = 114,286$ Mpa < fy = 400 Mpa

  → fs pakai = 114,286 MPa

  a $= \beta 1.x$

  = 0,85.294 = 249,9 mm

  $Cc = 0,85.fc'.b.a.10^{-3}$

  $= 0,85.25.400.249,9.10^{-3} = 2124,15$ KN

  $Cs = As'.(fs' - 0,85.fc').10^{-3}$

  $= 800.(400 - 0,85.25).10^{-3} = 303$ KN

  $Ts = As.fs.10^{-3}$

  $= 800.114,286.10^{-3} = 91,429$ KN

  Mn = Cc.(h/2-a/2) + Cs.(h/2-d') + Ts.(d-h/2)

  = 2124,15.(400/2-249,9/2) + 303.(400/2-50) + 91,429.(350-400/2)

  = 218,582 KNm

  Pn = Cc + Cs – Ts

  = 2124,15 + 303 – 91,429 = 2335,721 KN

  e $= \frac{Mn}{Pn} = \frac{218,582}{2335,721} = 0,094$ m

- x = 120%.xb

  = 1,2.210 = 252 mm

  $fs' = \frac{x - d'}{x}.600$

  $= \frac{252 - 50}{252}.600 = 480{,}952$ Mpa > fy = 400 Mpa

  →fs' pakai = 400 Mpa

  $fs = \frac{d - x}{x} \cdot 600$

  $= \frac{350 - 252}{252} \cdot 600 = 233{,}333$ Mpa < fy = 400 Mpa

  →fs pakai = 233,333 MPa

  $a = \beta 1.x$

  $= 0{,}85.252 = 214{,}2$mm

  $Cc = 0{,}85.fc'.b.a.10^{-3}$

  $= 0{,}85.25.400.214{,}2.10^{-3} = 1820{,}7$ KN

  $Cs = As'.(fs' - 0{,}85.fc').10^{-3}$

  $= 800.(400 - 0{,}85.25).10^{-3} = 303$ KN

  $Ts = As.fs.10^{-3}$

  $= 800.233{,}333.10^{-3} = 186{,}667$ KN

  Mn = Cc.(h/2-a/2) + Cs.(h/2-d') + Ts.(d-h/2)

  = 1820,7.(400/2-214,2/2) + 303.(400/2-50) + 186,667.(350-400/2)

  = 242,593 KNm

  Pn = Cc + Cs – Ts

  = 1820,7 + 303 – 186,667 = 1937,033 KN

  $e = \frac{Mn}{Pn} = \frac{242{,}593}{1937{,}033} = 0{,}125$ m

4. Patah Tarik

- x = 90%.xb

  = 0,9.210 = 189 mm

$$fs' = \frac{x - d'}{x}.600$$

$$= \frac{189 - 50}{189}.600 = 441{,}27 \text{ Mpa} > fy = 400 \text{ Mpa}$$

→fs' pakai = 400 Mpa

$$fs = \frac{d - x}{x} \cdot 600$$

$$= \frac{350 - 189}{350} \cdot 600 = 511 \text{ Mpa} > fy = 400 \text{ Mpa}$$

→fs pakai = 400 MPa

$a = \beta 1.x$

$= 0{,}85.189 = 160{,}65$ mm

$Cc = 0{,}85.fc'.b.a.10^{-3}$

$= 0{,}85.25.400.160{,}64.10^{-3} = 1365{,}525$ KN

$Cs = As'.(fs'-0{,}85.fc').10^{-3}$

$= 800.(400-0{,}85.25).10^{-3} = 303$ KN

$Ts = As.fs.10^{-3}$

$= 800.400.10^{-3} = 320$ KN

$Mn = Cc.(h/2-a/2) + Cs.(h/2-d') + Ts.(d-h/2)$

$= 136{,}525.(400/2-160{,}65/2) + 303.(400/2-50) + 320.(350-400/2)$

$= 256{,}869$ KNm

$Pn = Cc + Cs - Ts$

$= 1365{,}525 + 303 - 320 = 1348{,}525$ KN

$$e = \frac{Mn}{Pn} = \frac{256{,}869}{1348{,}525} = 0{,}190 \text{ m}$$

- $x = 80\%.xb$

  $= 0{,}8.210 = 168$ mm

$fs' = \frac{x - d'}{x}.600$

$= \frac{168 - 50}{168}.600 = 421{,}429$ Mpa > fy = 400 Mpa

→fs' pakai = 400 Mpa

$fs = \frac{d - x}{x} \cdot 600$

$= \frac{350 - 168}{168} \cdot 600 = 650$ Mpa > fy = 400 Mpa

→fs pakai = 400 MPa

$a = \beta 1.x$

$= 0{,}85.168 = 142{,}8$ mm

$Cc = 0{,}85.fc'.b.a.10^{-3}$

$= 0{,}85.25.400.142{,}8.10^{-3} = 1213{,}8$ KN

$Cs = As'.(fs'-0{,}85.fc').10^{-3}$

$= 800.(400-0{,}85.25).10^{-3} = 303$ KN

$Ts = As.fs.10^{-3}$

$= 800.400.\ 10^{-3} = 320$ KN

Mn = Cc.(h/2-a/2) + Cs.(h/2-d') + Ts.(d-h/2)

= 1213,8.(400/2-142,8/2) + 303.(400/2-50) + 320.(350-400/2)

= 249,545 KNm

Pn = Cc + Cs – Ts

= 1213,8 + 303 – 320 = 1196,8 KN

$e = \frac{Mn}{Pn} = \frac{249{,}545}{1196{,}8} = 0{,}208$ m

- $x = 70\%.xb$

$= 0{,}7.210 = 147$ mm

$fs' = \frac{x - d'}{x}.600$

$= \frac{147 - 50}{147}.600 = 395{,}918$ Mpa < fy = 400 Mpa

→fs' pakai = 395,918 Mpa

$fs = \frac{d-x}{x} \cdot 600$

$= \frac{350-147}{147} \cdot 600 = 828{,}571$ Mpa > fy = 400 Mpa

→fs pkai = 400 MPa

$a = \beta 1.x$

$= 0{,}85.147 = 124{,}950$ mm

$Cc = 0{,}85.fc'.b.a.10^{-3}$

$= 0{,}85.25.400.124{,}95.10^{-3} = 1062{,}075$ KN

$Cs = As'.(fs'-0{,}85.fc').10^{-3}$

$= 800.(395{,}918 - 0{,}85.25).10^{-3} = 299{,}735$ KN

$Ts = As.fs.10^{-3}$

$= 800.400.10^{-3} = 320$ KN

Mn = Cc.(h/2-a/2) + Cs.(h/2-d') + Ts.(d-h/2)

= 1062,075.(400/2-124,95/2) + 299,735.(400/2-50) + 320.(350-400/2)

= 239,022 KNm

Pn = Cc + Cs – Ts

= 1062,075 + 299,735 – 320 = 1041,810 KN

$e = \frac{Mn}{Pn} = \frac{239{,}022}{1041{,}810} = 0{,}229$ m

- x = 60%.xb

= 0,6.210 = 126 mm

$fs' = \frac{x-d'}{x}.600$

$= \frac{126-50}{126}.600 = 361{,}905$ Mpa < fy = 400 Mpa

→fs' pakai = 361,905 Mpa

$$fs = \frac{d-x}{x} \cdot 600$$

$$= \frac{350-126}{126} \cdot 600 = 1066{,}667 \text{ Mpa} > fy = 400 \text{ Mpa}$$

→fs pakai = 400 MPa

$a = \beta 1.x$

$= 0{,}85.126 = 107{,}1$ mm

$Cc = 0{,}85.fc'.b.a.10^{-3}$

$= 0{,}85.25.400.107{,}1.10^{-3} = 910{,}35$ KN

$Cs = As'.(fs'-0{,}85.fc').10^{-3}$

$= 800.(361{,}905-0{,}85.25).10^{-3} = 272{,}527$ KN

$Ts = As.fs.10^{-3}$

$= 800.400.10^{-3} = 320$ KN

$Mn = Cc.(h/2-a/2) + Cs.(h/2-d') + Ts.(d-h/2)$

$= 910{,}35.(400/2-107{,}1/2) + 272{,}527.(400/2-50) + 320.(350-400/2)$

$= 222{,}199$ KNm

$Pn = Cc + Cs - Ts$

$= 910{,}35 + 272{,}527 - 222{,}199 = 862{,}874$ KN

$$e = \frac{Mn}{Pn} = \frac{222{,}199}{862{,}874} = 0{,}257 \text{ m}$$

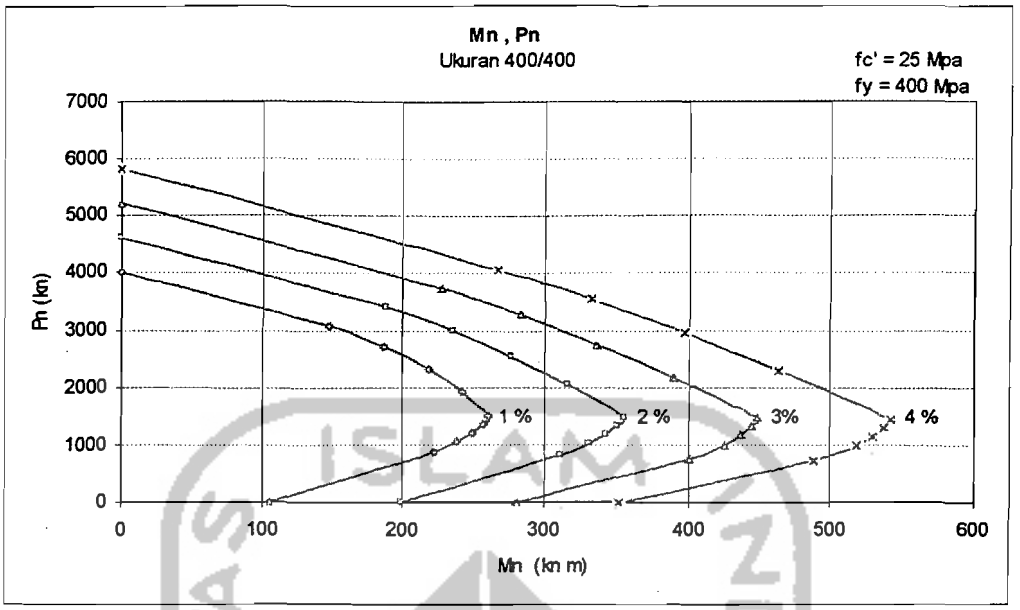
5. Batang Lentur Murni

$$a = \frac{As.fy}{0{,}85.fc.b}$$

$$= \frac{800.400}{0{,}85.25.400} = 37{,}647 \text{ mm}$$

$Mn = As.fy.(d-a/2).10^{6}$

$= 800.400.(350-37{,}467/2).10^{6} = 105{,}976$ KNm



Gambar 4.16 Grafik Mn-Pn

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4.5.2 Perencanaan Tulangan Lentur Kolom

Dari hasil output SAP didapat data sebagai berikut:

Untuk kolom lantai 3 B-2

	P	M2	M3		P	M2	M3
Da	136,68	8,53	14,65	Db	152,21	8,03	12,7
La	30,76	3,353	3,09	Lb	30,76	3,32	2,38
Mua	213,232	15,596	22,524	Mub	231,868	14,948	19,048

Perencanaan kolom sisi atas

Pu = 213,232 KN

Mux = 15,596 KNm

Muy = 22,524 KNm

L = 4,2 m

fy = 400 MPa

fc' = 25 MPa

d' = 50 mm

b = 400 mm

h = 400 mm

d = 350 mm

Φ = 0,65

Pu/Φ = 328,04923 KN

Mux/Φ = 23,99384 KNm

Muy/Φ = 34,65230 KNm

β1 = 0,85

Perencancaan arah x:

Pn desain = 328,04923 KN

M desain = 23.993845 KNm

e = 0,073141 m

Dari grafik Pu & Mn didapat:

Ast = 1 %

= 0,01 x 400 x 400 = 1600 mm^2

Øtul = 19 mm

A1Ø = 283,64 mm^2

$$\text{Jml tul} = \frac{1600}{283,64} = 5,6409$$

Jml pakai = 6 batang (6D19) → Ast = 6 x 283,64 = 1701,9 mm^2

As = As' = 0,5.Ast = 0,5. 1701,9 = 850,93 mm^2

$$xb = \frac{600.d}{600 + fy} = \frac{600..350}{600 + 400}$$

= 210 mm

ab = xb.β1

= 210.0,85 = 178,5 mm

$$fs' = \frac{xb - d'}{xb} x600$$

$$= \frac{210 - 50}{210} x600 = 457,14 \text{ mm}$$

fs' < fy = 400 MPa

Pnb = $\beta 1.fc'.b.d + As.fy - As'.fs'$

= $0,85.25.400.400 + 850,93.400 - 850.93.400$

= 1517,3 KN

Mnb = $0,85.fc'b.a.(h/a-a/2) + As'(fs'-0,85.fc')(h/2-d') + As.fy(d-h/2)$

= $0,85.20.400.178.5(400/178.5-178.5/2) + 850,93.(400-0,85.25).(400/2-$

$50) + 850,93.400(350-400/2)$

= 267,43 KNm

eb = $\frac{267,43}{1517,3} = 0,1763mm \rightarrow e = 0,073141\ m < eb = 0,1763\ m$

Patah Desak

Perencancaan arah y:

Pn desain = 213,232 KN

M desain = 22,524 KNm

e = 0,10563 m

Dari grafik Pu & Mn didapat:

Ast = 1 %

= $0,01 \times 400 \times 400 = 1600\ mm^2$

Øtul = 19 mm

AlØ = $283,64\ mm^2$

Jml tul = $\frac{1600}{283,64} = 5,6409$

Jml pakai = 6 batang (6D19) → Ast = $6 \times 283,64 = 1701,9\ mm^2$

As = As' = $0,5.Ast = 0,5.\ 1701,9 = 850,93\ mm^2$

$$xb = \frac{600.d}{600+fy} = \frac{600..350}{600+400}$$

$= 210$ mm

$ab = xb.\beta 1$

$= 210.0{,}85 = 178{,}5$ mm

$$fs' = \frac{xb-d'}{xb} x600$$

$$= \frac{210-50}{210} x600 = 457{,}14 \text{ mm}$$

$fs' < fy = 400$ MPa

$Pnb = \beta 1.fc'.b.d + As.fy - As'.fs'$

$= 0{,}85.25.400.400 + 850{,}93.400 - 850.93.400$

$= 1517{,}3$ KN

$Mnb = 0{,}85.fc'b.a.(h/a-a/2) + As'(fs'-0{,}85.fc')(h/2-d') + As.fy(d-h/2)$

$= 0{,}85.20.400.178.5(400/178.5-178.5/2) + 850{,}93.(400-0{,}85.25).(400/2-$

$50) + 850{,}93.400(350-400/2)$

$= 267{,}43$ KNm

$$eb \quad -\frac{267{,}43}{1517{,}3} = 0{,}1763 \text{mm} \rightarrow e = 0{,}10563 \text{ m} < eb = 0{,}1763 \text{ m}$$

Patah desak

4.6 Perencanaan Fondasi Tiang

Dalam perencanaan proyek ini digunakan pondasi dalam karena lapisan tanah keras yang terletak pada kedalaman 9 m, adapun pondasi dalam yang digunakan adalah pondasi *bored pile* karena lokasi proyek yang terletak dalam area rumah sakit.

4.6.1 Perencananaan Tulangan Pondasi *Bored Pile*

Pondasi bored pile yang digunakan direncanakan sebagai tiang panjang pada tanah berbutir.

d = 60 cm

Qa = 390,1284 ton > Paksial =156,0133 ton

γ = 1,686667 gr/cm^3

= 1686,667 kg/m^3

θ = 38,66°

Kp = tg^2(45°+1/2.θ)

= tg^2(45°+1/2.38,66°)

= tg^2 64,33 = 4,329

$$\frac{Ho}{Kp.\gamma.D^2} = \frac{10167,867}{4,329.1686,667.0,6^2}$$

= 3,8682

→ Grafik $\frac{Ho}{Kp.\gamma.D^2}$ Vs $\frac{Mo}{Kp.\gamma.D^3}$ untuk ujung atas ditahan, didapat

$$\frac{Mo}{Kp.\gamma.D^3} = 1,1$$

$Mo = 1,1.Kp.\gamma.D^3$

$= 1,1.4,329.1686,667.0,6\text{^}3$

$= 1734,855$ kgm

$P = 146941$ kg

$eu = \frac{1734,855}{146941} = 0,011806$ m

$= 1,11806$ cm

$ht = 60$ cm

$\frac{eu}{ht} = \frac{1,11806}{60}$

$= 0,01967$

$\sigma^*ou = \frac{Nu}{\frac{1}{4}.\pi.ht^2}$

$= \frac{146941}{0,25.3,14.60^2}$

$= 51,9488$ kg/cm^2

$\sigma'bk = 175$ kg/cm^2

$\frac{\sigma^*ou}{2.ko.\sigma'bk} = \frac{51,9488}{2.0,6.175}$

$= 0,247375$

berdasarkan $\frac{eu}{ht}$ dan, dari grafik Reinforced Concrete Column didapat qtot minimum, sehingga digunakan syarat tulangan minimum pada Buku Pedoman Perencanaan Untuk Struktur Beton Bertulang Biasa dan Struktur Tembok Bertulang untuk Gedung 1983.

Untuk D<80 cm, maka:

$$\text{Amin} = \sqrt{\frac{Ag}{2}}$$

$$= \sqrt{\frac{0{,}25.3{,}14.60^2}{2}} = 37{,}6070 \text{ cm}^2$$

$$\text{Amin} = 0{,}01.Ag$$

$$= 0{,}01.0{,}25.3{,}14.60^2 = 28{,}2857 \text{ cm}^2$$

$$\text{Amaks} = 0{,}06.Ag$$

$$= 0{,}06.0{,}25.3{,}14.60^2 = 169{,}7143 \text{ cm}^2$$

→Amin terpakai = 37,6070 cm^2

Digunakan tulangan pokok D19

$$A_{1D19} = 0{,}25.3{,}14.19^2$$

$$= 283{,}6429 \text{ mm}^2$$

$$= 2{,}8364 \text{ cm}^2$$

$$n = \frac{37{,}6070}{2{,}8364}$$

$$= 13{,}2586 \rightarrow 16D19$$

Tulangan spiral

Digunakan tulangan spiral P10

Daerah 1, s maks $= \frac{D}{5} = \frac{60}{5} = 12{,}5$ cm, atau

= 12 cm, atau

= 6.D19 = 6.19 = 114 mm = 11,4 cm

s dipakai yang terkecil dari ketiga s maks yang ada

→P10-11

Daerah 2, s maks $= \frac{D}{2} = \frac{60}{2} = 30$ cm, atau

= 30 cm, atau

= 12.D19 = 6.19 = 228 mm = 22,8 cm

s dipakai yang terkecil dari ketiga s maks yang ada

→P10-20

4.6.2 Perencanaan *Pile Cap* (kepala tiang)

ht = diameter tiang bor

= 60 cm

b = 120 cm

d = 5 cm

h = 100-5 = 95 cm

e = 42,5 cm

M = 0,35.1.1.2,4.0,425+0,35.1.1.1,8.0,425

= 0,4845 tm

Digunakan

Beton mutu K175 = 175 kg/cm^2

Beton mutu U39 = 3390 kg/cm^2

$$Cu = \frac{h}{\sqrt{\frac{M}{2.ko.\sigma'bk.b}}}$$

$$= \frac{95}{\sqrt{\frac{0,4845.1000.100}{2.0,6.175.120}}} = 62,544$$

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cu diluar chart Wiratman sehinga digunakan

qmin = 0,0433

$$A = q.\frac{2.ko.\sigma' bk.b.h}{\sigma * au}$$

$$= 0{,}0433.\frac{2.0{,}6.175.120.95}{3390} = 30{,}5782\ cm^2$$

Digunakan tulangan D19

$A_{1D19} = 283{,}6426\ mm^2$

$= 2{,}836426\ cm^2$

n $\frac{30{,}5782}{2{,}836426} = 10{,}7805 \rightarrow$ 12D19 dengan jarak s = 10 cm.