BAB IV

PERENCANAAN STRUKTUR

4.1 Rangka Atap Kuda-kuda Baja

4.1.1 Data Konstruksi Rangka Atap

- Jarak antar kuda-kuda maksimum (b) = 3,6 m
- Panjang bentang (L) = 11,975 m
- Mutu baja profil :

  Tegangan leleh (fy) = 36 Ksi = 2531 kg/cm^2

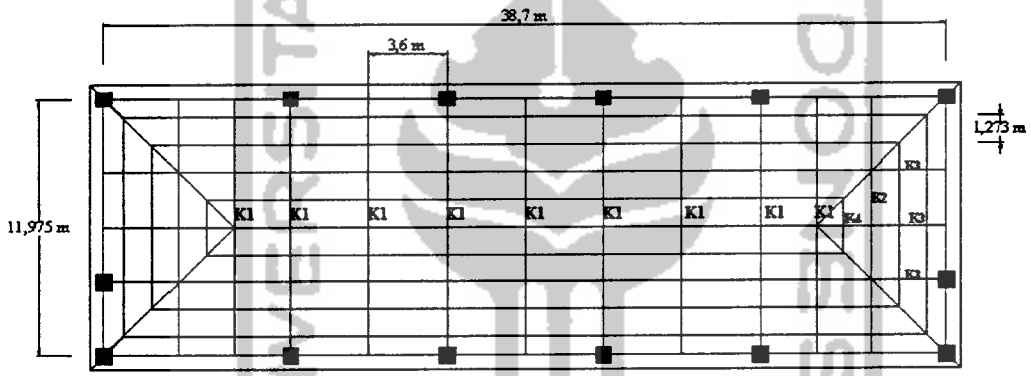
  Kuat tarik (Fu) = 58 Ksi = 4077 kg/cm^2
- Mutu baut A325N (Full Draat) :

  Tegangan tarik (Ft) = 33 Ksi

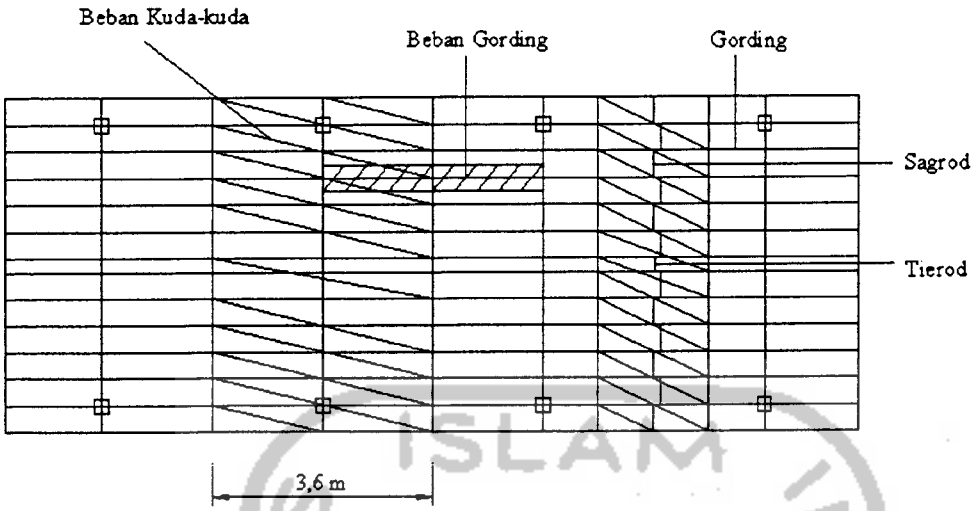
  Tegangan geser (Fv) = 21 Ksi
- Untuk atap genteng $\alpha \geq 22{,}5°$, sedangkan untuk atap asbes, seng $\alpha \geq 10°$. Pada perencanaan ini dipakai atap genteng.
- Usuk dan reng dipakai kayu sedangkan gording dipakai baja jenis Light Lip Channel
- Jurai menggunakan profil Double Light Lip Channel dan rangka kuda-kuda menggunakan profil Double Angel.

4.1.2 Jumlah dan Jarak Antar Gording

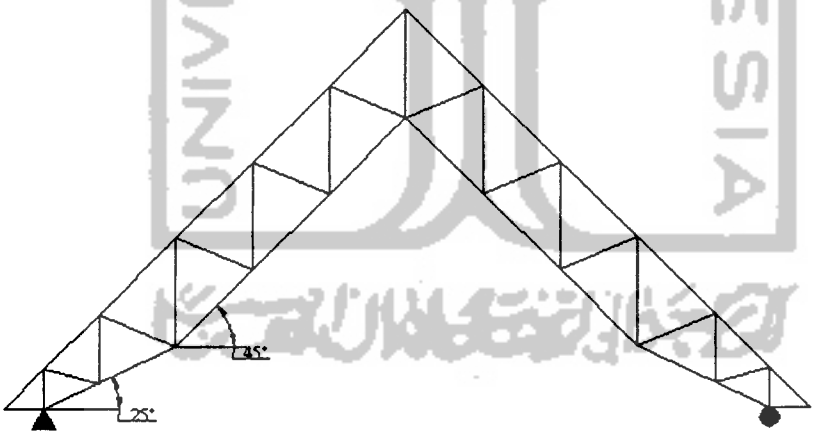
- Jarak gording maksimum (Atap Genteng) = 2,5 m
- Panjang sisi miring kuda-kuda (M) $= \frac{0{,}5L}{\cos\alpha} = \frac{0{,}5.12}{\cos 45} = 8{,}468$ m
- Jumlah gording setengah bentang (n) = 7 buah
- Jarak antar gording (Lg) $= \frac{1{,}273}{\cos 45} = 1{,}800$ m < 2,5 m



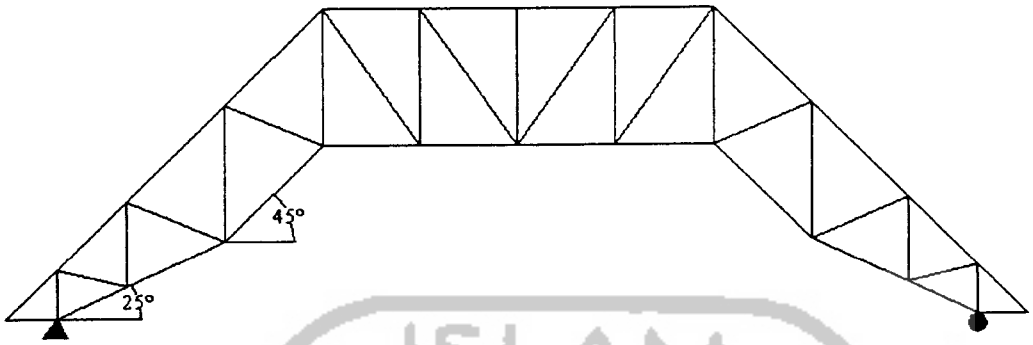
Gambar 4.1 Rencana Denah Kuda – kuda Blok-C



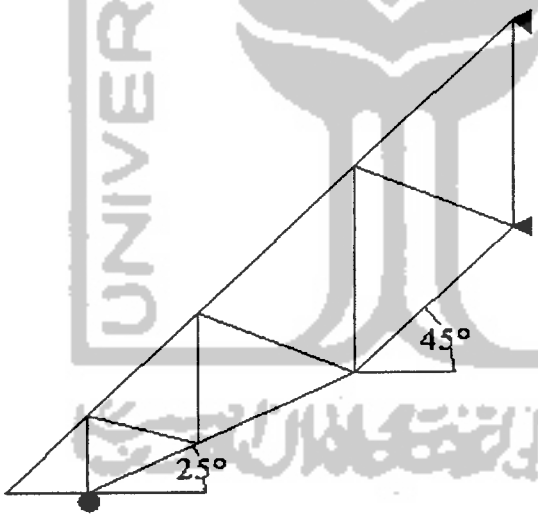
Gambar 4.2 Pembebanan



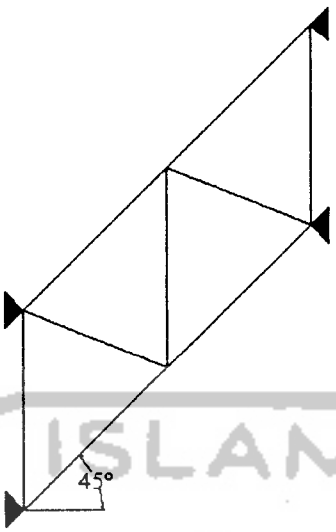
Gambar 4.3 Rencana Kuda – Kuda K1



Gambar 4.4 Rencana Kuda – Kuda K2

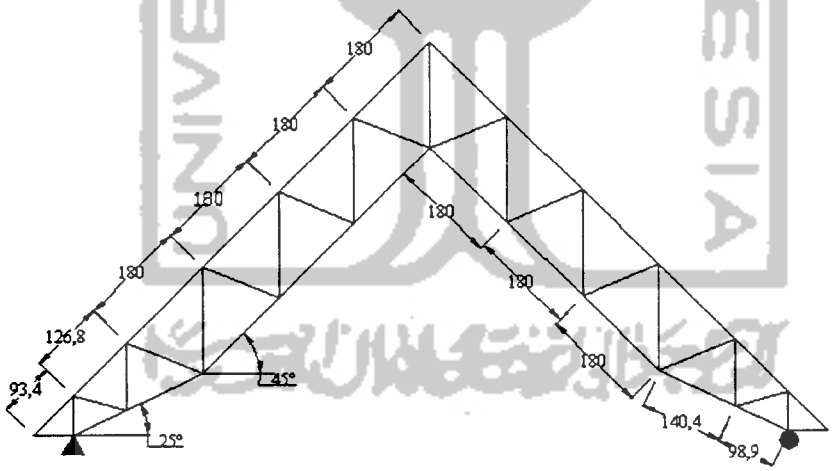


Ganbar 4.5 Rencana Kuda – Kuda K3



Ganbar 4.6 Rencana Kuda – Kuda K4

4.1.3 Perencanaan Gording



Gambar 4.7 Dimensi Batang Pada K1

a. Pembebanan Gording

1. Beban Tetap

- Berat genteng (table 2.1.PPIUG'83) = 50 x 1,800 = 90,00 kg/m'

- Beban hidup (Pasal 3.2.2.b.PPIUG'83) = 20 x 1,800 = 36,00 kg/m'
- Berat gording taksiran (7 s/d 10 kg/m') = 10 kg/m'

q_{total} = 136 kg/m'

Mekanika gording

$q\perp = q_{total} . \cos \alpha = 136{,}00 . \cos 45° = 96{,}167$ kg/m'

$q // = q_{total} . \sin \alpha = 136{,}00 . \sin 45° = 96{,}167$ kg/m'

2. Beban Angin

$Wa = 25$ kg/m^2 (pasal 4.2.1.PPIUG'83)

- Angin Tekan (Wt)

$C_1 = 0{,}02\ \alpha - 0{,}4 = 0{,}02 . 45 - 0{,}4 = + 0{,}5$ kg/m'(tekan)

$Wt = C_1 . Wa . Lg = 0{,}5 . 25 . 1{,}800 = + 22{,}500$ kg/m' (tekan)

- Angin Hisap (Wh)

$C_2 = - 0{,}4$

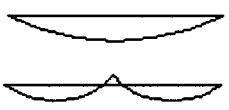
$Wh = C_2 . Wa . Lg = - 0{,}4 . 25 . 1{,}800 = - 18{,}000$ kg/m'(hisap)

$W\perp = + 22{,}500$ kg/m' (tekan)

W // = 0 (karena beban angin bekerja atap, PPIUG '83)

b. Momen yang terjadi

- akibat beban tetap



$M\perp \text{ maks} = \frac{1}{8} q\perp L^2 = \frac{1}{8} 96{,}167.3{,}6^2 = 155{,}791$ kgm'

$M // \text{ maks} = \frac{1}{32} q // L^2 = \frac{1}{32} 96{,}167.3{,}6^2 = 38{,}948$ kgm'

- akibat beban angin

$$M \perp maks = \frac{1}{8} W \perp L^2 = \frac{1}{8} 22{,}5.3{,}6^2 = 36{,}45 \text{ kgm'}$$

c. Penentuan Profil Baja

Dicoba profil *Light Lip Channel* (Ir.Morisco, hal 46) **150x50x20x3,2**

Sx = 21,3 cm^3 fy = 2531 kg/cm^2

Sy = 7,80 cm^3 E = 2,1 x 10^6 kg/cm^2

Ix = 107 cm^4 Fu = 4077 kg/cm^2

Iy = 24,5 cm^4

W = 5,5 kg/m

d. Kontrol Tegangan

$$fbx = \frac{M \perp maks}{Sx} = \frac{(155{,}791 + 36{,}45)x100}{21{,}3} = 902{,}540 \text{ kg/cm}^2$$

$$fby = \frac{M // maks}{Sy} = \frac{38{,}948x100}{7{,}8} = 499{,}333 \text{ kg/cm}^2$$

$$\frac{fbx}{0{,}66fy} + \frac{fby}{0{,}75fy} = \frac{902{,}540}{0{,}66.2531} + \frac{499{,}333}{0{,}75.2531} = 0{,}803 \leq 1{,}0 \quad \textbf{(Ok!)}$$

e. Kontrol Lendutan

$$\delta \perp = \frac{5}{384} . \frac{q \perp .L^4}{E.Ix} = \frac{5}{384} \frac{0{,}8(96{,}167 + 22{,}5).3{,}6^4.10^6}{2{,}1.10^6.107}$$

$$= 0{,}924 \text{ cm} \leq \frac{L}{360} = \frac{3{,}6.100}{360} = 1{,}00 \text{ cm } \textbf{(Ok!)}$$

$$\delta \perp = \frac{5}{384} . \frac{q \perp .L^4}{E.Ix} = \frac{5}{384} \frac{1{,}0(96{,}167).3{,}6^4.10^6}{2{,}1.10^6.107}$$

$$= 0{,}936 \text{cm} \leq \frac{L}{360} = \frac{3{,}6.100}{360} = 1{,}00 \text{ cm } \textbf{(Ok!)}$$

$$\delta // = \frac{5}{384}\frac{q//(L/(a+1))^4}{E.Iy} = \frac{5}{384}\frac{96{,}167(3{,}6/(1+1))^4.10^6}{2{,}1.10^6.24{,}5}$$

$$= 0{,}256\text{cm} \leq \frac{L}{360} = \frac{3{,}6.100}{360} = 1{,}00 \text{ cm } \textbf{(Ok!)}$$

jadi profil *Light Lip Channel* **150x50x20x3,2**dapat dipakai.

4.1.4 Perencanaan Sagrod dan Tierod

1. Sagrod

   Beban Sagrod dan Tierod :

   - Berat penutup atap x sisi miring (M) = 50 x 8,468 = 423,40 kg/m'
   - Beban hidup x sisi miring (M) = 20 x 8,468 = 169,36 kg/m'
   - Beban gording = berat gording x jml gording = 5,5 x 7 = 38,500 kg/m'

   P = 631,26 kg/m'

   Ss = L/2 = 3,6/2 = 1,8 m

   P// = P.sin α . Ss = 631,260 . sin 45 . 1,80 = 803,462 kg

   $$A_{sagrod} = \frac{P//}{0{,}33.Fu} = \tfrac{1}{4}.\pi.D^2_{sagrod}$$

   $$D = \sqrt{\frac{P//.4}{0{,}33.Fu.\pi}} = \sqrt{\frac{803{,}462.4}{0{,}33.4077.\pi}} = 0{,}4574 \text{ cm} = 4{,}574 \text{ mm}$$

   dipakai sagrod = D + 3 = 4,574 + 3 = 7,574 mm

2. Tierod

   Beban Tierod = T = P// . cos α = 803,462 . cos 45 = 568,133 kg

   $$A_{tierod} = \frac{T}{0{,}33.Fu} = \tfrac{1}{4}.\pi.D^2_{tierod}$$

$$D = \sqrt{\frac{T.4}{0{,}33.Fu.\pi}} = \sqrt{\frac{568{,}133.4}{0{,}33.4077.\pi}} = 0{,}540 \text{ cm} = 5{,}400 \text{ mm}$$

dipakai tierod = D + 3 = 5,400 + 3 = 8,400 mm

Sagrod dan Tierod dipakai diameter = 10 mm (P10)

4.1.5 Perencanaan Kuda – Kuda

4.1.5.1 Pembebanan Kuda – Kuda

Beban Tetap :

- Berat gording (Light Lip Channel) = 5,50 kg/m'
- Berat eternity (Tabel 2.1,PPIUG '83) = 11,0 kg/m^2
- Penggantung langit – langit (dari kayu)= 7,00 kg/m^2
- Berat penutup atap (genteng) = 50,0 kg/m^2
- Beban hidup = 20,0 kg/m^2
- Berat kuda-kuda taksiran :

Berat kuda-kuda $= \left\{\left\{10 + \left(\frac{12-12}{3}\right) \times 5\right\} \times 3{,}6 = 35{,}85 \text{ kg/m'}\right.$

Berat baut dan plat sambung = 20% x berat kuda-kuda

= 0,2 x 35,85 = 7,17 kg/m'

Beban kuda – kuda = 35,85 + 7,17 = 43,02 kg/m'

Beban – beban pada joint :

a. $P_1 = P_{13}$

Beban gording	= 5,5 x 3,6	= 19,80	kg
Berat penutup atap	= 50 x 3,60 x ½ (0,934)	= 84,060	kg
		qD = 103,86	kg

Beban hidup (qL) = 20 x 3,60 x ½(0,934) = 33,624 kg

b. $P_2 = P_{12}$

Beban gording = 5,5 x 3,6 = 19,8 kg

Berat penutup atap = 50x3,60x½(0,934+1,268) = 198,180 kg

qD = 217,98 kg

Beban hidup (qL) = 20x3,6x½(0,934+1,268) = 79,272 kg

c. $P_3 = P_{11}$

Beban gording = 5,5 x 3,6 = 19,80 kg

Berat penutup atap = 50 x 3,60 x (1,268+1,800) = 276,12 kg

qD = 295,92 kg

Beban hidup (qL) = 20 x 3,6 x (1,268+1,800) = 110,448 kg

d. $P_4 = P_5 = P_6 = P_7 = P_8 = P_9 = P_{10}$

Beban gording = 5,5 x 3,6 = 19,80 kg

Berat penutup atap = 50 x 3,60 x 1,8 = 324 kg

qD = 343,8 kg

Beban hidup (qL) = 20 x 3,6 x 1,800 = 129,6 kg

e. $P_1' = P_{13}'$

Berat eternit dan penggantung = 18 x 3,6 x ½.0,66 = 21,384 kg

Berat kuda-kuda = 43,200 x ½.0,66 = 14,197 kg

= 35,581 kg

f. $P_2' = P_{12}'$

Berat eternit =18x3,6x½(0,66+0,989) = 53,428 kg

Berat kuda-kuda = 43,200 x ½(0,66+0,989) = 35,470 kg

= 88,898 kg

g. $P_3' = P_{11}$

Berat eternit = 18 x 3,6 x½(0,989+1,404) = 77,533 kg

Berat kuda-kuda = 43,200 x ½(0,989+1,404) = 51,473 kg

= 129,006 kg

h. $P_4' = P_{10}'$

Berat eternit = 18 x 3,60 x½(1,800+1,404) = 103,809 kg

Berat kuda-kuda = 43,200 x ½(1,800+1,404) = 68,918 kg

= 172,727 kg

i. $P_5' = P_6' = P_7' = P_8' = P_9$

Berat eternit = 18 x 3,60 x 1,800 = 116,640 kg

Berat kuda-kuda = 43,200 x 1,800 = 77,436 kg

= 194,076 kg

Beban angin :

Wa = 25 kg/m^2 (pasal 4.2.1.PPIUG '83)

Koefisien angin :

- Angin Tekan (Wt)

$C_1 = 0,02\ \alpha - 0,4 = +0,5$ (tekan)

- Angin Hisap (Wh)

$C_2 = -0,4$

Beban-beban Angin :

$Wt = C_1 . Wa = + 0,5 . 25 = + 12,5$ kg/m^2 (tekan)

$Wh = C_2 . Wa = - 0,4 . 25 = - 10$ kg/m^2 (hisap)

a. Angin kiri

akibat angin tekan :

$Wt_1 = 12,5 \times 3,60 \times ½(0,934)$ $= + 21,015$ kg

$Wt_2 = 12,5 \times 3,60 \times ½(0,934+1,268)$ $= + 49,545$ kg

$Wt_3 = 12,5 \times 3,60 \times ½(1,268+1,800)$ $= + 69,03$ kg

$Wt_4 = Wt_5 = Wt_6 = 12,5 \times 3,60 \times 1,800$ $= + 81,00$ kg

$Wt_7 = 12,5 \times 3,6 \times ½(1,800)$ $= + 40,5$ kg

Akibat angin hisap :

$Wh_1 = -10 \times 3,6 \times ½(0,934)$ $= - 16,812$ kg

$Wh_2 = -10 \times 36 \times ½(0,934+1,268)$ $= - 39,636$ kg

$Wh_3 = -10 \times 3,60 \times (1,268+1,8)$ $= - 55,224$ kg

$Wh_4 = Wh_5 = Wh_6 = -10 \times 3,60 \times 1,8$ $= - 35,071$ kg

$Wh_7 = -10 \times 3,6 \times ½(1,8)$ $= - 32,4$ kg

b. Angin kanan

Besar angin kanan sama dengan angin kiri

4.1.6 Perhitungan Rangka

Analisis rangka menggunakan SAP2000 dapat dilihat dalam lampiran 1 dan beban rencana kuda-kuda KK1 dapat dilihat pada table 4.5.

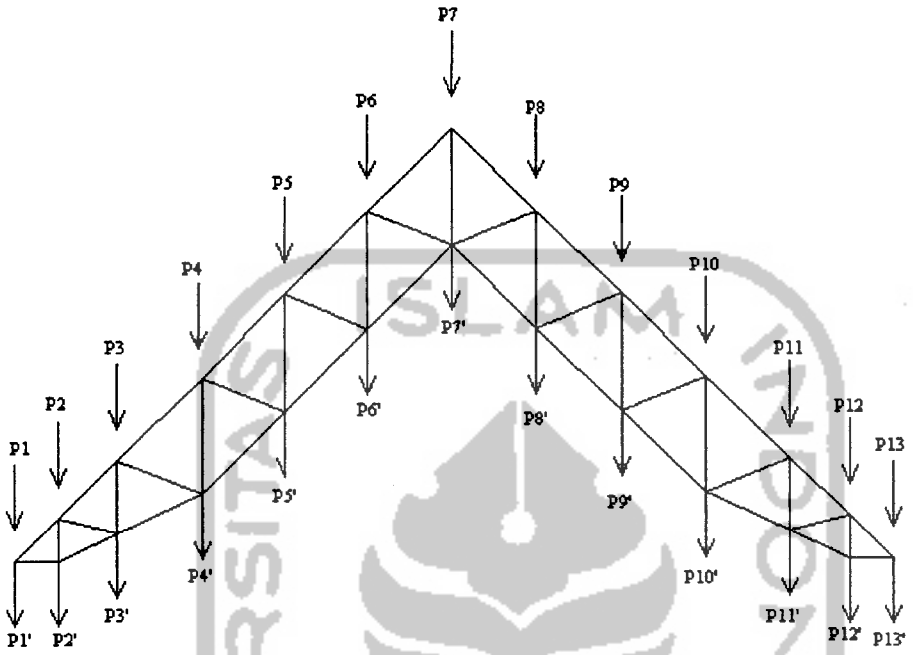
1. Data profil baja yang digunakan

Modulus of Elasticity (Es) = 2,039.10^{10} kg/m^2 = 2,039.10^6 kg/cm^2

fy = 25310507 kg/m^2 = 2531 kg/cm^2

2. Data – data pembebanan yang dimasukkan pada SAP2000

a. Akibat beban tetap



Gambar 4.8 Gaya Akibat Beban Tetap

Tabel 4.1 Gaya P_1 sampai dengan P_{13}

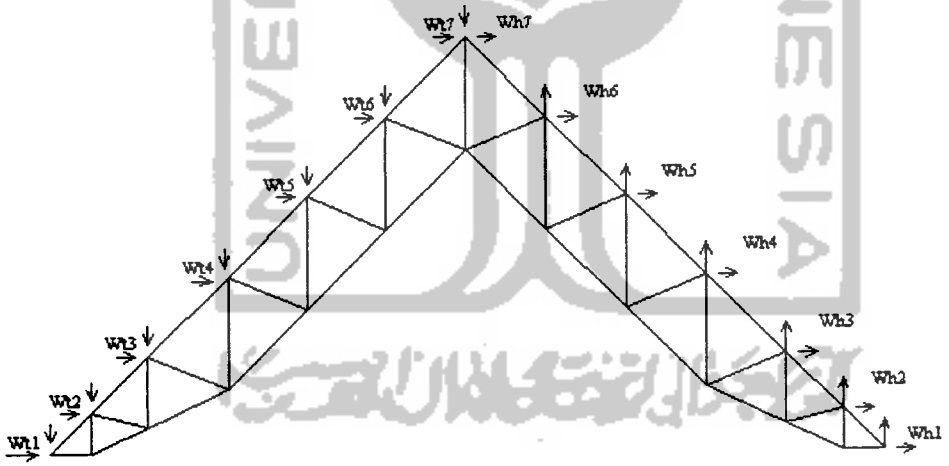
Nama Gaya	Beban Mati (qD) kg	Beban Hidup (qL) kg
$P_1 = P_{13}$	- 103,860	- 33,624
$P_2 = P_{12}$	- 217,980	- 79,272
$P_3 = P_{11}$	- 295,920	- 110,448
$P_4 = P_5 = P_6 = P_7 = P_8 = P_9 = P_{10}$	-343,8	- 129,600

Untuk pembebanan P_1' s/d P_{13}' pada perhitungan SAP2000, berat kuda-kuda sudah termasuk berat sendiri maka tidak dimasukkan dalam perhitungan.

Tabel 4.2 Gaya P_1' s/d P_{13}'

Nama Gaya	Beban Mati (qD) kg
$P_1' = P_{13}'$	- 35,581
$P_2' = P_{12}'$	- 88,898
$P_3' = P_{11}'$	- 129,006
$P_4' = P_{10}'$	- 172,727
$P_5' = P_6' = P_7' = P_8' = P_9'$	- 194,076

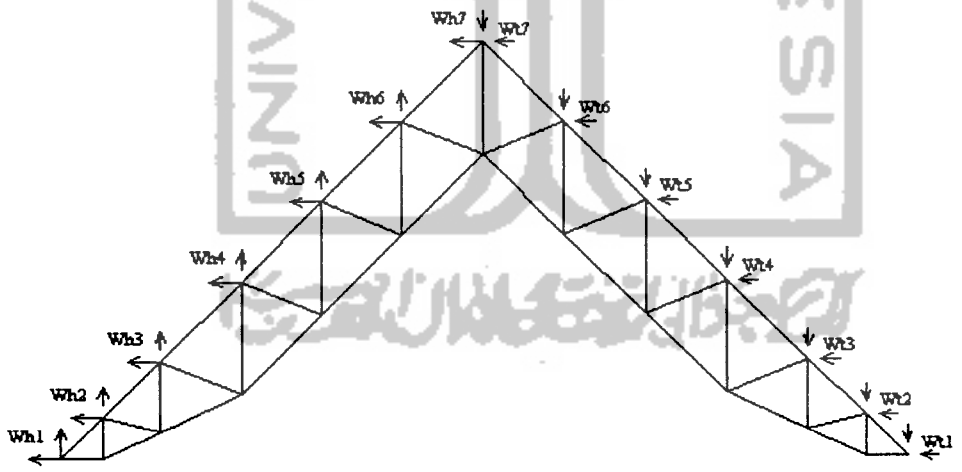
b. Akibat beban angin



Gambar 4.9 Gaya Akibat Angin Kiri

Tabel 4.3 Gaya Tekan dan Hisap Angin Kiri

Nama Gaya	Gaya akibat Beban Angin Kiri (Wki) kg	Gaya Horizontal = Wki x cos 45° (kg)	Gaya Vertikal = Wki x sin 45° (kg)
Wt_1	21,015	+ 14,859	- 14,859
Wt_2	49,545	+ 35,034	- 35,034
Wt_3	69,030	+ 48,812	- 48,812
$Wt_4=Wt_5=Wt_6$	81	+ 52,276	- 52,276
Wt_7	40,500	+ 28,638	+ 28,638
Wh_1	16,812	+ 11,888	+ 11,888
Wh_2	39,636	+ 28,027	+ 28,027
Wh_3	55,224	+ 39,049	+ 39,049
$Wh_4=Wh_5=Wh_6$	64,800	+ 45,821	+ 45,821
Wh_7	32,400	+ 22,910	+ 22,910



Gambar 4.10 Gaya Akibat Angin Kanan

Tabel 4.4 Gaya Hisap dan Tekan Angin Kanan

Nama Gaya	Gaya akibat Beban Angin Kanan (Wka) kg	Gaya Horizontal = Wka x cos 45° (kg)	Gaya Vertikal = Wka x sin 45° (kg)
Wt_1	21,015	-14,859	-14,859
Wt_2	49,545	-35,034	-35,034
Wt_3	63,030	-48,812	-48,812
$Wt_4=Wt_5=Wt_6$	81,000	-52,276	-52,276
Wt_7	40,500	-28,638	-28,638
Wh_1	16,812	-11,888	+11,888
Wh_2	39,636	-28,027	+28,027
Wh_3	55,224	-39,049	+39,049
$Wh_4=Wh_5=Wh_6$	64,800	-45,821	+45,821
Wh_7	32,400	-22,910	+22,910

Tabel 4.5 Gaya batang Yang Terjadi Pada Kuda – Kuda 1

Batang	Panjang (m)			RVI				
Atas								
A1	0,934	241,160	20,910	-16,880	313,508	262,070	224,280	241,160
A2	1,268	-3233,530	-232,920	196,000	-4203,589	-3466,450	-3037,530	-3233,530
A3	1,800	-4468,850	-312,340	264,860	-5809,505	-4781,190	-4203,990	-4468,850
A4	1,800	-6225,310	-629,600	580,460	-8092,903	-6854,910	-5644,850	-6225,310
A5	1,800	-7273,310	-842,490	804,510	-9455,303	-8115,800	-6468,800	-7273,310
A6	1,800	-7597,620	-950,770	936,640	-9876,906	-8548,390	-6660,980	-7597,620
A7	1,800	-7597,620	-1023,680	1009,550	-9876,906	-8621,300	-6588,070	-7597,620
A8	1,800	-7273,310	-642,420	604,440	-9455,303	-7915,730	-6668,870	-7273,310
A9	1,800	-6225,310	-349,680	300,540	-8092,903	-6574,990	-5924,770	-6225,310
A10	1,800	-4468,850	-147,260	99,770	-5809,505	-4616,110	-4369,080	-4468,850
A11	1,268	-3233,530	-74,730	37,800	-4203,589	-3308,260	-3195,730	-3233,530
A12	0,934	241,160	-16,860	20,890	313,508	224,300	262,050	241,160
Bawah								
B1	0,660	-166,440	-30,400	24,590	-216,372	-196,840	-141,850	-166,440
B2	0,989	-183,680	542,900	-549,310	-238,784	359,220	-732,990	-732,990
B3	1,404	2518,720	704,450	-686,700	3274,336	3223,170	1832,020	2518,720
B4	1,800	4462,860	912,540	-892,960	5801,718	5375,400	3569,900	4462,860

B5	1,800	6215,730	1155,390	-1143,320	8080,449	7371,120	5072,410	6215,730
B6	1,800	7254,310	1293,070	-1301,210	9430,603	8547,380	5953,100	7254,310
B7	1,800	7254,310	880,950	-889,090	9430,603	8135,260	6365,220	7254,310
B8	1,800	6215,730	525,760	-513,690	8080,449	6741,490	5702,040	6215,730
B9	1,800	4462,860	258,640	-239,060	5801,718	4721,500	4223,800	4462,860
B10	1,404	2518,720	102,440	-84,700	3274,336	2621,160	2434,020	2518,720
B11	0,989	-183,680	26,370	-32,780	-238,784	-157,310	-216,460	-183,680
B12	0,660	-166,440	23,800	-29,610	-216,372	-142,640	-196,050	-166,440
V1	0,660	-3431,400	-252,600	210,650	-4460,820	-3684,000	-3220,750	-3431,400
V2	1,139	-1669,970	-107,940	91,760	-2170,961	-1777,910	-1578,210	-1669,970
V3	1,818	-2277,240	-349,830	344,460	-2960,412	-2627,070	-1932,780	-2277,240
V4	1,818	-1561,600	-244,500	252,080	-2030,080	-1806,100	-1309,520	-1561,600
V5	1,818	-840,520	-138,430	158,820	-1092,676	-978,950	-681,700	-840,520
V6	1,818	10215,430	1385,490	-1377,000	13280,059	11600,920	8838,430	10215,430
V7	1,818	-840,520	-358,020	378,410	-1092,676	-1198,540	-462,110	-1198,540
V8	1,818	-1561,600	-269,400	276,980	-2030,080	-1831,000	-1284,620	-1561,600
V9	1,818	-2277,240	-177,970	172,590	-2960,412	-2455,210	-2104,650	-2277,240
V10	1,139	-1669,970	-50,810	34,630	-2170,961	-1720,780	-1635,340	-1669,970
V11	0,660	-3431,400	-31,470	-10,480	-4460,820	-3462,870	-3441,880	-3431,400
D1	0,929	2527,990	151,350	-128,750	3286,387	2679,340	2399,240	2527,990
D2	1,385	949,890	7,540	-9,960	1234,857	957,430	939,930	949,890
D3	1,385	1350,960	187,120	-192,890	1756,248	1538,080	1158,070	1350,960
D4	1,385	801,500	106,010	-121,560	1041,950	907,510	679,940	801,500
D5	1,385	292,490	30,880	-56,260	380,237	323,370	236,230	292,490
D6	1,385	292,490	347,750	-373,130	380,237	640,240	-80,640	292,490
D7	1,385	801,500	273,540	-289,090	1041,950	1075,040	512,410	1075,040
D8	1,385	1350,960	205,490	-211,260	1756,248	1556,450	1139,700	1350,960
D9	1,385	949,890	97,900	-100,330	1234,857	1047,790	849,560	949,890
D10	0,929	2527,990	71,220	-48,620	3286,387	2599,210	2479,370	2527,990

Tabel 4.6 Reaksi Yang Terjadi Pada Kuda – Kuda K1

2	Tetap	0	3616,3245
	LWL	-523,508	24,3629
	RWL	523,508	20,4711
12	Tetap	0	3616,3245
	LWL	0	20,4711
	RWL	0	24,3629

4.1.7 Perencanaan Profil Kuda – Kuda I

1. Batang Tekan (A_2 s/d A_{11})

Gaya batang (tekan) maksimum = 7597,620 kg

Panjang = 1,80 m = 180 cm

ambil $\frac{k.L}{r} = 50$

$$Cc = \frac{6440}{\sqrt{fy}} = \frac{6440}{\sqrt{2531}} = 128{,}009 > \frac{k.L}{r} = 50 \text{ , maka :}$$

$$Fs = \frac{5}{3} + \frac{3}{8}\frac{kL/r}{Cc} - \frac{1\left(kL/r\right)^3}{8.Cc^3} = \frac{5}{3} + \frac{3}{8}\frac{.50}{128{,}009} - \frac{1}{8}\frac{50^3}{128{,}009^3} = 1{,}806$$

$$Fa_{perlu} = \frac{fy}{Fs}\left(1 - 0{,}5\left(\frac{kL/r}{Cc}\right)^2\right) = \frac{2531}{1{,}806}\left(1 - 0{,}5\left(\frac{50}{128{,}009}\right)^2\right) = 1294{,}533 \text{ kg/cm}^2$$

$$A_{perlu} = \frac{P}{Fa_{perlu}} = \frac{7597{,}620}{1294{,}533} = 5{,}869 \text{ cm}^2$$

Profil **2L 60x60x6** (profil yang biasa digunakan di lapangan), dengan :

$A = 2 \times 6{,}91 = 13{,}82 \text{ cm}^2$

r = 1,82 cm

Kontrol Local Buckling :

$$\frac{bf}{tf} \le \frac{76}{\sqrt{fy}}$$

$$\frac{60}{6} \le \frac{76}{\sqrt{36}}$$

$10 \le 12{,}667$ **(Ok!)**

Kontrol Beban :

$$\frac{kL}{r} = \frac{1.180}{1,82} \leq Cc = \frac{6440}{\sqrt{fy}} = \frac{6440}{\sqrt{2531}}$$

$= 98,901 \leq 128,009$ (terjadi tekuk elastis)

$$Fs = \frac{5}{3} + \frac{3}{8}\frac{kL/r}{Cc} - \frac{1(kL/r)^3}{8.Cc^3} = \frac{5}{3} + \frac{3}{8}\frac{98,901}{128,009} - \frac{1}{8}\frac{98,901^3}{128,009^3} = 1,898$$

$$Fa_{ada} = \frac{2531}{1,898}\left(1 - 0,5\left(\frac{98,901}{128,009}\right)^2\right) = 936,123 \text{ kg/cm}^2$$

$P_{ada} = Fa_{ada} . A_{ada} \geq P_{tjd}$

$= 936,123 . 13,82$

$= 12937,220 \text{ kg} \geq 7597,620$..... (Ok!)

2. Batang Tarik (B6)

$P_{tarik\ maks} = 7254,310$ kg $fy = 2531 \text{ kg/cm}^2$

Panjang = 1,80 m $Fu = 4077 \text{ kg/cm}^2$

$$r_{min} = \frac{L}{240} = \frac{180}{240} = 0,75 \text{ cm}$$

• Untuk batang ada lubang

$\mu = 0,85$ (semua profil dengan jumlah baut $\geq$ 3 buah/baris)

$\mu = 0,75$ (jumlah baut 2 buah/baris)

$$A_{netto\ perlu} = \frac{P}{0,5.Fu.\mu} = \frac{7254,310}{0,5.4077.0,75} = 4,745 \text{ cm}^2$$

Dicoba profil **2L 40x40x4**, dimana :

$A = 2 \times 3,08 = 6,16 \text{ cm}^2$

$r = 1,21$ cm

tw = 0,4 cm

d baut = ½ " = 1,27 cm

Kontrol kelangsingan (λ) :

$$\lambda_{ada} = \frac{kL}{r_{ada}} = \frac{1.180}{1,82} = 98,901 \leq 240 \text{ (Ok!)}$$

Kontrol Tegangan :

• Untuk batang tidak ada lubang

$$fa = \frac{T}{A_{profil}} \leq 0,6fy$$

$$= \frac{7254,310}{6,16} = 934,443 \text{ kg/cm}^2 \leq 0,6.2531 = 1518,6 \text{ kg/cm}^2 \text{ (Ok!)}$$

• Untuk batang ada lubang

$$A_{netto\ profil} = A_{profil} - (d_{baut} + 1/8'')t_{pelat}$$

$$= 6,16 - (1,27 + 0,3175) \,.\, 0,4 \,.\, 2$$

$$= 4,890 \text{ cm}^2$$

$$A_{efektif} = A_{netto} \,.\, \mu = 4,890 \,.\, 0,75 = 3,668 \text{ cm}^2$$

$$fa = \frac{T}{A_{efektif}} \leq 0,5Fu$$

$$= \frac{7254,310}{3,668} \leq 0,5.4077 = 1973,729 \text{ kg/cm}^2 \leq 2038,5 \text{ kg/cm}^2 \text{ (Ok!)}$$

Tabel 4.7 Profil Terpakai dan Berat Profil Terpakai

Batang	Profil	Berat Profil (kg/m)	Panjang (m)	Berat (kg)
		A	B	A x B
Atas	2L 60x60x6	2 x 5,42 = 10,84	18,804	203,835
Bawah	2L 40x40x4	2 x 2,42 = 4,84	16,907	81,830
Vertikal	2L 40x40x5	2 x 2,97 = 5,94	16,324	96,840
Diagonal	2L 40x40x4	2 x 2,42 = 4,84	12,936	62,610
			Wtotal (kg)	439,063

Kontrol berat kuda – kuda :

- Berat total kuda-kuda (Wtotal) = 439,063 kg
- Berat baut dan plat sambung = 20% x berat total kuda-kuda

  = 0,2 x 439,063 = 87,813 kg

Jumlah (Σ) = W total + 20%berat total kuda-kuda

= 439,063 + 87,813 = 526,876 kg

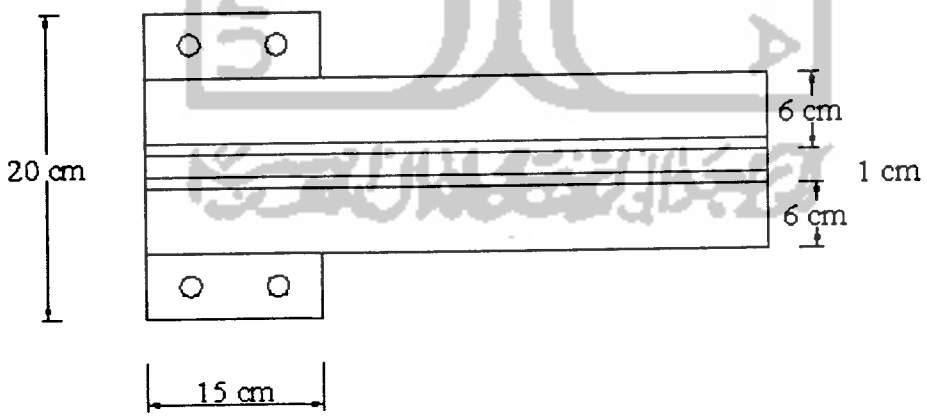
- Panjang bentang kuda-kuda (L) = 12 m

$\frac{\Sigma}{L} \leq$ Berat taksiran kuda – kuda

$\frac{526,876}{11,975} \approx 43,20 \, {}^{kg}/_{m'}$

43,998 kg/m' ≈ 43,20 kg/m'

4.1.8 Perencanaan Pelat Kuda – Kuda



Gambar 4.11 Pelat Kuda – Kuda

Beban P dimbil dari reaksi dukungan dari perhitungan SAP2000 :

P_{maks} = 3616,3245 kg f'c = 25 Mpa = 250 kg/cm^2

$$A_{perlu} = \frac{P}{0{,}33f'c} = \frac{3616{,}3245}{0{,}33.250} = 43{,}834 \text{ cm}^2$$

Dipakai ukuran pelat = 15 cm x 20 cm = 300 cm^2 > A_{perlu} = 43,834 cm^2

$$q = \frac{P}{BxL} = \frac{3616{,}3245}{15x20} = 12{,}054 \text{ kg/cm}$$

$$x = \frac{20-(6+1+6)}{2} = 3{,}5 \text{ cm}$$

$M = \frac{1}{2}.q.x^2 = \frac{1}{2} . 12{,}054 . 3{,}5^2 = 73{,}831$ kgcm

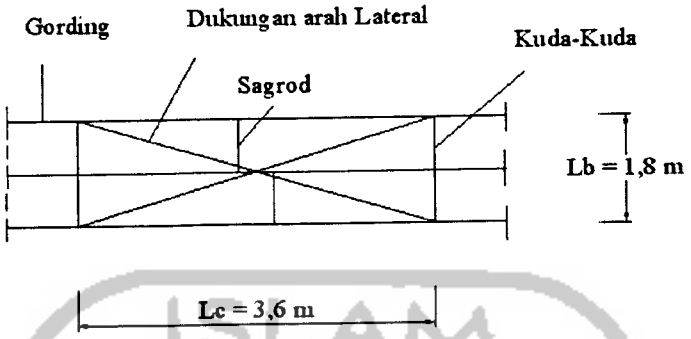
Syarat :

$$0{,}6.Fy = \frac{M}{\frac{1}{6}.1.tp}$$

$$tp = \sqrt{\frac{10.M}{Fy}} = \sqrt{\frac{10.73{,}831}{2531}} = 0{,}540 \text{ cm} \approx 0{,}6 \text{ cm}$$

Pelat kuda – kuda berukuran : 15 x 20 x 0,6 cm^3

4.1.9 Perencanaan Dukungan Lateral



Gambar 4.12 Dukungan Arah Lateral

Diketahui :

Lb = jarak antar gording = 1,8 m

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

$L = \sqrt{Lb^2 + Lc^2} = \sqrt{1{,}8^2 + 3{,}6^2} = 4{,}025 \text{ m} = 402{,}5 \text{ cm}$

Syarat :

L/refleks ≤ 300 , sehingga :

$r_{min} \geq \frac{L}{300} = \frac{402{,}5}{300} = 1{,}342 \text{ cm} = 13{,}42 \text{ mm}$

Keterangan :

- L ≤ 3 m, dipakai baja tulangan diameter 12 mm
- L ≥ 3 m, dipakai baja tulangan diameter 19 mm
- 3 m < L ≤ 5 m, dipakai baja tulangan diameter 16 mm

Karena L = 4,025 m, maka dukungan arah lateral dipakai baja tulangan diameter 16 mm > r_{min} = 13,42 mm

4.1.10 Perencanaan Sambungan

- Tebal pelat sambung = 1 cm, d_{baut} = ½" = 1,27 cm
- Mutu Pelat Baja :

  Tegangan leleh (fy) = 2531 kg/cm^2

  Kuat tarik (Fu) = 3700 kg/cm^2
- Mutu Baut A325N (Full Draat) :

  Kekuatan ultimit (Fu) = 8250 kg/cm2

Tinjauan Tegangan Geser 1 Baut :

P_{geser} = ¼ . π . D^2_{baut} x 0,17 . Fu x jumlah bidang geser (n)

= ¼ . π . $1,27^2$. 0,17 . 8250 . 2

= 3551,484 kg

Tinjauan Tegangan Tumpu 1 Baut :

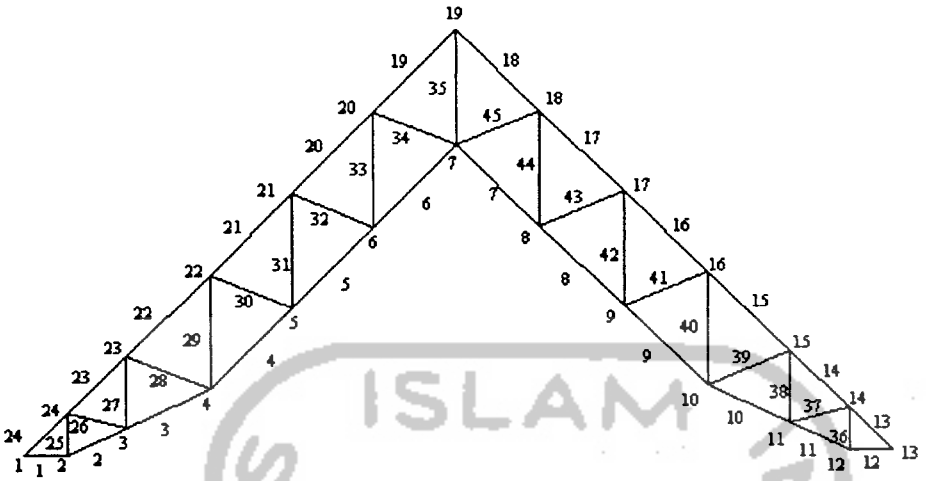
P_{tumpu} = 1,2 . Fu . D_{baut} . t . jumlah tumpuan (n)

= 1,2 . 3700 . 1,27 . 1 . 1

= 5638,800 kg

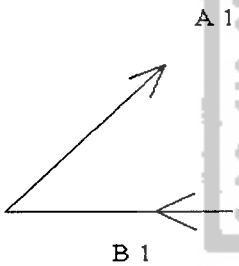
Jadi $P_{1\ baut}$ dipakai P_{geser} = 3551,484 kg

Jumlah Baut (N) = $\frac{P_{terjadi}}{P_{1baut}}$



Gambar 4.13 Rangka Kuda – Kuda

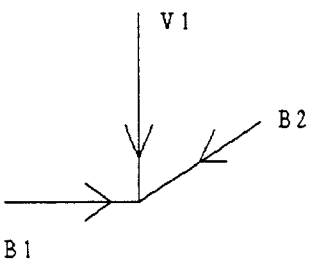
- Joint 1



$$n\,A1 = \frac{241{,}160}{3551{,}484} = 0{,}068 \approx 2 \text{ buah}$$

$$n\,B1 = \frac{166{,}410}{3551{,}484} = 0{,}047 \approx 2 \text{ buah}$$

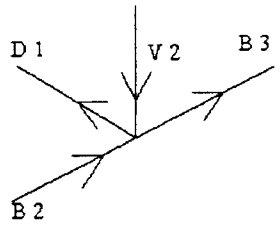
- Joint 2



$$n\,B2 = \frac{732{,}990}{3551{,}484} = 0{,}206 \approx 2 \text{ buah}$$

$$n\,V1 = \frac{3431{,}400}{3551{,}484} = 0{,}966 \approx 2 \text{ buah}$$

- Joint 3

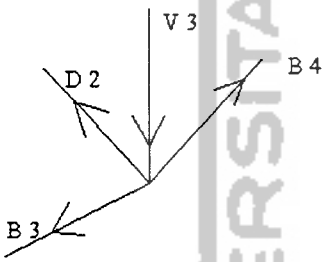


$$n\ B3 = \frac{2518{,}720}{3551{,}484} = 0{,}709 \approx 2 \text{ buah}$$

$$n\ V2 = \frac{1669{,}970}{3551{,}484} = 0{,}470 \approx 2 \text{ buah}$$

$$n\ D1 = \frac{2527{,}990}{3551{,}484} = 0{,}711 \approx 2 \text{ buah}$$

- Joint 4

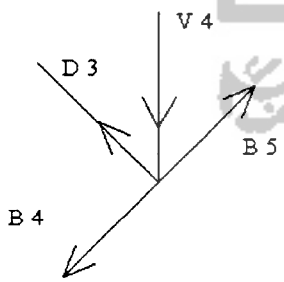


$$n\ B4 = \frac{4462{,}860}{3551{,}484} = 1{,}257 \approx 2 \text{ buah}$$

$$n\ V3 = \frac{2277{,}240}{3551{,}484} = 0{,}641 \approx 2 \text{ buah}$$

$$n\ D2 = \frac{949{,}89}{3551{,}484} = 0{,}267 \approx 2 \text{ buah}$$

- Joint 5

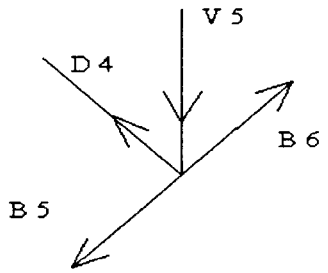


$$n\ B5 = \frac{6215{,}730}{3551{,}484} = 1{,}750 \approx 2 \text{ buah}$$

$$n\ V4 = \frac{1561.600}{3551{,}484} = 0{,}440 \approx 2 \text{ buah}$$

$$n\ D3 = \frac{1350.960}{3551{,}484} = 0{,}381 \approx 2 \text{ buah}$$

- Joint 6

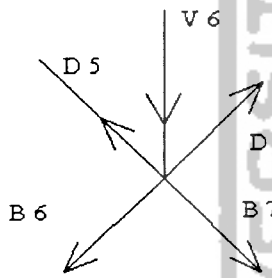


$$n\ B6 = \frac{7254{,}310}{3551{,}484} = 2{,}043 \approx 3 \text{ buah}$$

$$n\ V5 = \frac{840.520}{3551{,}484} = 0{,}237 \approx 2 \text{ buah}$$

$$n\ D4 = \frac{801.500}{3551{,}484} = 0{,}226 \approx 2 \text{ buah}$$

- Joint 7

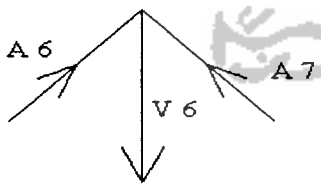


$$n\ B7 = n\ B6 = 3 \text{ buah}$$

$$n\ D5 = n\ D6 = \frac{292.490}{3551{,}484} = 0{,}082 \approx 2 \text{ buah}$$

$$n\ V6 = \frac{10215.430}{3551{,}484} = 2{,}8 \approx 3 \text{ buah}$$

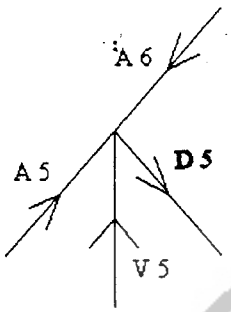
- Joint 19



$$n\ A6 = n\ A7 = \frac{7597.620}{3551{,}484} = 2{,}140 \approx 3 \text{ buah}$$

$$n\ V6 = \frac{10215.430}{3551{,}484} = 2{,}8 \approx 3 \text{ buah}$$

- Joint 20

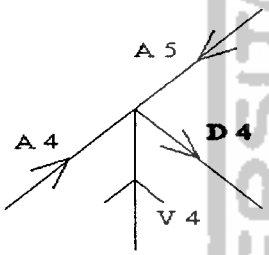


$$n\ A5 = \frac{7273.310}{3551.484} = 2.048 \approx 3 \text{ buah}$$

$$n\ V5 = \frac{840.520}{3551.484} = 0,237 \approx 2 \text{ buah}$$

$$n\ D5 = \frac{292.490}{3551.484} = 0,082 \approx 2 \text{ buah}$$

- Joint 21

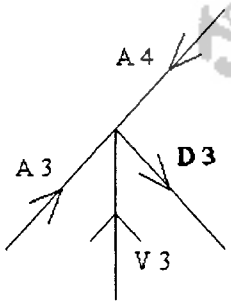


$$n\ A4 = \frac{6225.310}{3551.484} = 1,750 \approx 2 \text{ buah}$$

$$n\ V4 = \frac{1561.600}{3551.484} = 0,440 \approx 2 \text{ buah}$$

$$n\ D4 = \frac{801.500}{3551.484} = 0,225 \approx 2 \text{ buah}$$

- Joint 22

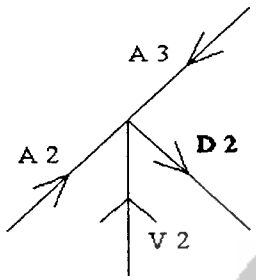


$$n\ A3 = \frac{4468.850}{3551.484} = 1,258 \approx 2 \text{ buah}$$

$$n\ V3 = \frac{2277.240}{3551.484} = 0,641 \approx 2 \text{ buah}$$

$$n\ D3 = \frac{1350.960}{3551.484} = 0,380 \approx 2 \text{ buah}$$

- Joint 23

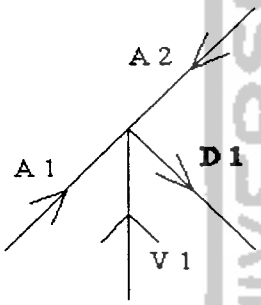


$$n\ A2 = \frac{3233.530}{3551.484} = 0{,}910 \approx 2 \text{ buah}$$

$$n\ V2 = \frac{1669.970}{3551.484} = 0{,}470 \approx 2 \text{ buah}$$

$$n\ D2 = \frac{949.890}{3551.484} = 0{,}267 \approx 2 \text{ buah}$$

- Joint 24



$$n\ A1 = \frac{241.160}{3551.484} = 0{,}067 \approx 2 \text{ buah}$$

$$n\ V1 = \frac{3431.400}{3551.484} = 0.966 \approx 2 \text{ buah}$$

$$n\ D1 = \frac{2527.990}{3551.484} = 0{,}712 \approx 2 \text{ buah}$$

Tabel 4.8 Jumlah Baut pada setengah bentang Kuda-kuda I

Joint	Batang	Jumlah Baut (buah)
1	A1 ; B1	2
2	(B1,B2) ; V1	2
3	(B2,B3) ; V2 ; D1	2
4	(B3,B4) ; V3 ; D2	2
5	(B4,B5) ; V4 ; D3	2
6	(B5,B6) ; V5 ; D4	2
7	(B6,B7) ; V6 ; (D5,D6)	3
19	(A6,A7) ; V6	3
20	(A5,A6) ; V5 ; D5	3
21	(A4,A5) ; V4 ; D4	3
22	(A3,A4) ; V3 ; D3	2
23	(A2,A3) ; V2 ; D2	2
24	(A1,A2) ; V1 ; D1	2

4.2 Perencanaan Pelat

4.2.1 Perencanaan Pelat Lantai

a. Pembebanan Pelat Lantai

- Beban mati pelat lantai :

1. berat sendiri pelat (perkiraan) : 0,12 x 24 = 2,88 KN/m^2
2. pasir (tebal 5 cm) : 0,05 x 16 = 0,80 KN/m^2
3. Spesi (tebal 3 cm) : 0,03 x 21 = 0,63 KN/m^2
4. Keramik : 0,01 x 20 = 0,20 KN/m^2 +

Beban mati total (qD) = 4,51 KN/m^2

- Beban hidup pelat lantai :

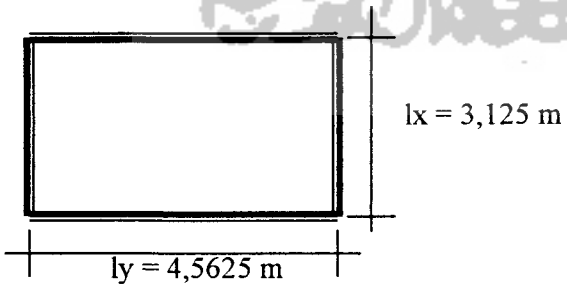
Gedung ini berfungsi sebagai kantor dan ruang kuliah, sehingga beban hidup (qL) sebesar 250 kg/cm^2 atau 2,5 KN/m^2 (PPIUG, 1983 tabel 3.1, halaman 17)

- Kombinasi pembebanan (SK SNI T-15-1991-03, Pasal 3.2.2)

qU = 1,2.qD + 1,6.qL = 1,2.4,51 + 1,6.2,5 = 9,412 KN/m^2

b. Perencanaan Pelat Lantai Tipe PC 1

Pelat dianggap terjepit elastis pada keempat sisinya.



$\frac{ly}{lx} = \frac{4,5625}{3,125} = 1,466$, dihitung sebagai pelat dua arah.

Koefisien momen (C) pada tabel 13.3.2 halaman 203 PBBI 1971 NI-2.

Koef. Momen Pelat (C)	1,4	1,466	1,5
Mlx = -Mtx	53	54,80	56
Mly = -Mty	38	37,40	37

Untuk nilai koefisien momen pelat (C) diantara yang tercantum pada tabel, maka nilainya diperoleh dengan cara interpolasi linier, yaitu sebagai berikut :

$$C = 53 + \left(\frac{1,466 - 1,4}{1,5 - 1,4} x(56 - 53) \right) = 54,80$$

Diperkirakan balok tepi pelat mempunyai lebar , b = 200 mm

maka : lnx = 3125 – 200 = 2925 mm

lny = 4563 – 200 = 4363 mm

perbandingan bentang bersih sisi panjang dan pendek :

$$\beta = \frac{\ln y}{\ln x} = \frac{4363}{2925} = 1,492$$

sehingga tebal pelat tidak boleh kurang dari :

$$h = \frac{\ln(0,8 + {fy}/{1500})}{36 + 9.\beta} = \frac{4363(0,8 + {240}/{1500})}{36 + 9.1,492} = 84,739 \text{ mm}$$

tetapi tidak perlu lebih besar sama dengan dari :

$$h = \frac{\ln(0,8 + {fy}/{1500})}{36} = \frac{4363(0,8 + {240}/{1500})}{36} = 116,347 \text{ mm}$$

Berdasarkan PBBI, 1971, NI – 2, pasal 9.1 ayat 1, bahwa dalam segala hal tebal pelat tidak boleh kurang dari 7 cm untuk pelat atap dan 12 cm untuk

pelat lantai. Karena tebal pelat maksimum < 12 cm, maka tebal pelat lantai dipakai 12 cm.

- Digunakan tulangan pokok Ø 8 mm
- Penutup beton (Pb) digunakan 20 mm

Tinggi manfaat tulangan pelat lantai :

- Arah x : $dx = h - Pb - \frac{1}{2}Ø_{tul.x}$

  $= 120 - 20 - \frac{1}{2}.8$

  $= 96$ mm

- Arah y : $dy = h - Pb - Ø_{tul.x} - \frac{1}{2}Ø_{tul.y}$

  $= 120 - 20 - 8 - \frac{1}{2}.8$

  $= 88$ mm

Momen – momen yang bekerja pada pelat :

$Mulx = - Mutx = 0,001 . qU . lx^2 . C$

$= 0,001 . 9,412 . 3,125^2 . 54,80 = 5,037$ KNm

$Muly = - Muty = 0,001 . qU . lx^2 . C$

$= 0,001 . 9,412 . 3,125^2 . 37,40 = 3,438$ KNm

1) Perencanaan Tulangan lx dan tx

Mulx = - Mutx = 5,037 KNm

$$Mu/\phi = 5,037/0,8 = 6,296 \text{ KNm}$$

Rasio Tulangan (ρ)

$$\rho b = \frac{0,85.f'c.\beta_1}{fy}\left(\frac{600}{600+fy}\right) = \frac{0,85.25.0,85}{240}\left(\frac{600}{600+240}\right) = 0,053757$$

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

$$\rho_{max} = 0,75 \cdot \rho b = 0,75 \cdot 0,053746 = 0,04032$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.25} = 11,294$$

Koefisien ketahanan (Rn) :

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{6,296.10^6}{1000.96^2} = 0,683 \text{ Mpa}$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2m.Rn}{fy}}\right) = \frac{1}{11,294}\left(1 - \sqrt{1 - \frac{2.11,294.0,683}{240}}\right)$$

$$= 0,0029 < \rho_{max} = 0,04032$$

$$< \rho_{min} = 0,00583$$

$1,33\ \rho = 1,333 \cdot 0,0029 = 0,00386 < \rho_{min} = 0,00583$, maka :

$$\rho_{pakai} = 1,33\ \rho = 0,00386$$

$$As_p = \rho_{pakai} \cdot b \cdot d \geq 0,002 \cdot b \cdot h$$

$$= 0,00386 \cdot 1000 \cdot 96 \geq 0,002 \cdot 1000 \cdot 120$$

$$= 370,560\ mm^2 \geq 240\ mm^2$$

digunakan tulangan pokok Ø 8 mm, sehingga :

$$A_1Ø = ¼ \cdot \pi \cdot D^2 = ¼ \cdot \pi \cdot 8^2 = 50,265\ mm^2$$

$$\text{jarak tulangan (s)} = \frac{A_1\phi.b}{As_p}$$

$$= \frac{50,265.1000}{370,560}$$

$$= 135,646 \text{ mm}$$

dipakai s = 130 mm, maka Tulangan Pokok : **P8 – 130**

$$As_{ada} = \frac{A_1 \phi . b}{s} = \frac{50{,}265.1000}{130} = 386{,}654 \text{ mm}^2$$

Kontrol Kapasitas Lentur Pelat (arah x) :

$$a = \frac{As_{ada} . fy}{0{,}85 . f'c . b} = \frac{386{,}654 . 240}{0{,}85 . 25 . 1000} = 4{,}3669 \text{ mm}$$

$$Mn = As_{ada} . fy \left(d - \tfrac{a}{2}\right) \geq 1{,}33 . \tfrac{Mu}{\phi} \text{ (karena } \rho_{pakai} = 1{,}33\rho \text{)}$$

$$= 386{,}654 . 240 \left(96 - \tfrac{4{,}3669}{2}\right) / 10^6$$

$$= 8{,}706 \text{ KNm} \geq 1{,}33 . 6{,}296 = 8{,}374 \text{ KNm} \text{ .....OK !}$$

2) Perencanaan Tulangan ly dan ty

$$Muly = - Muty = 3{,}438 \text{ KNm}$$

$$\tfrac{Mu}{\phi} = \tfrac{3{,}438}{0{,}8} = 4{,}2975 \text{ KNm}$$

Rasio Tulangan (ρ)

$$\rho b = \frac{0{,}85 . f'c . \beta_1}{fy} \left(\frac{600}{600 + fy}\right) = \frac{0{,}85 . 25 . 0{,}85}{240} \left(\frac{600}{600 + 240}\right) = 0{,}053757$$

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

$$\rho_{max} = 0{,}75 . \rho b = 0{,}75 . 0{,}053746 = 0{,}04032$$

$$m = \frac{fy}{0{,}85 . f'c} = \frac{240}{0{,}85 . 25} = 11{,}294$$

Koefisien ketahanan (Rn) :

$$Rn = \frac{\tfrac{Mu}{\phi}}{b . d^2} = \frac{4{,}2975 . 10^6}{1000 . 88^2} = 0{,}555 \text{ Mpa}$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2m.Rn}{fy}}\right) = \frac{1}{11{,}294}\left(1-\sqrt{1-\frac{2.11{,}294.0{,}555}{240}}\right)$$

$= 0{,}00234 < \rho_{max} = 0{,}04032$

$< \rho_{min} = 0{,}00583$

$1{,}33\ \rho = 1{,}333 \,.\, 0{,}00234 = 0{,}00311 < \rho_{min} = 0{,}00583$, maka :

$\rho_{pakai} = 1{,}33\ \rho = 0{,}00311$

$As_p = \rho_{pakai} \,.\, b \,.\, d \geq 0{,}002 \,.\, b \,.\, h$

$= 0{,}00311 \,.\, 1000 \,.\, 88$

$= 273{,}680\ mm^2 \geq 240\ mm^2$

digunakan tulangan pokok Ø 8 mm, sehingga :

$A_1Ø = ¼ \,.\, \pi \,.\, D^2 = ¼ \,.\, \pi \,.\, 8^2 = 50{,}265\ mm^2$

jarak tulangan (s) $= \dfrac{A_1\phi.b}{As_p}$

$= \dfrac{50{,}265.1000}{273{,}680}$

$= 183{,}663$ mm

Berdasarkan SK SNI T-15-1991-03, pasal 3.6.4, ayat 2 bahwa jarak antar tulangan pada penampang kritis tidak boleh melebihi dua kali tebal pelat, maka :

dipakai s = 180 mm, maka Tulangan Pokok : **P8 – 180**

$$As_{ada} = \frac{A_1\phi.b}{s} = \frac{50{,}265.1000}{180} = 279{,}250\ mm^2$$

Kontrol Kapasitas Lentur Pelat (arah y) :

$$a = \frac{As_{ada}.fy}{0,85.f'c.b} = \frac{279,250.240}{0,85.25.1000} = 3,154 \text{ mm}$$

$Mn = As_{ada} . fy (d - a/2) \geq 1,33. Mu/\phi$ (karena $\rho_{pakai} = 1,33\rho$)

$= 279,250 . 240 (88 - 3,154/2) / 10^6$

$= 5,792$ KNm $\geq 1,33.4,2975 = 5,716$ KNm .....OK !

3) Perencanaan Tulangan Bagi Pelat Lantai

$As_{bagi} = 0,002 . b . h$

$= 0,002 . 1000 . 120 = 240 \text{ mm}^2$

dugunakan tulangan bagi Ø 8 mm, sehingga :

$A_1Ø = ¼ . \pi . D^2 = ¼ . \pi . 8^2 = 50,265 \text{ mm}^2$

jarak tulangan bagi (s) $= \frac{A_1\phi.b}{As_{bagi}}$

$= \frac{50,265.1000}{240} = 209,4375$ mm

dipakai s bagi = 200 mm, maka Tulangan Bagi : **P8 – 200**

4.2.2 Perencanaan Pelat Atap

a. Pembebanan Pelat Atap

- Beban mati pelat atap :

1. Berat sendiri pelat (perkiraan)	$= 0,10 \times 24 = 2,40$ KN/m^2	
2. Lapis kedap air (tebal 3 cm)	$= 0,03 \times 22 = 0,66$ KN/m^2	+
Beban mati total (qD)	$= 3,06$ KN/m^2	

- Beban hidup pelat atap :

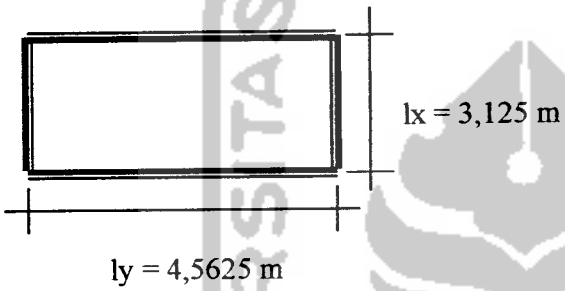
Pada pelat atap terdapat beban hidup (qL) berupa beban pekerja atau air hujan sebesar 100 kg/cm^2 atau 1,0 KN/m^2 (PPIUG, 1983 tabel 3.1 , halaman 17)

- Kombinasi pembebanan (SK SNI T-15-1991-03, pasal 3.2.2)

  qU = 1,2qD + 1,6qL = 1,2 . 3,06 + 1,6 . 1,0 = 5,272 KN/m^2

b. Perencanaan Pelat Atap Tipe PA1

Pelat dianggap terjepit elastis pada keempat sisinya.



$\frac{ly}{lx} = \frac{4,5625}{3,125} = 1,46$, dihitung sebagai pelat dua arah.

Koefisien momen (C) pada table 13.3.2 halaman 203 PBBI 1971 NI-2.

Nilai koefisien momen untuk $^{ly}/_{lx} = 1,46$ adalah sebagai berikut :

	Koefisien Momen Pelat (C)
Mlx = - Mtx	54,8
Mly	37,94
- Mty	37,94

Diperkirakan balok tepi pelat mempunyai lebar , b = 200 mm

maka : $lnx = 3125 - 200 = 2925$ mm

$lny = 4562,5 - 200 = 4362,5$ mm

perbandingan bentang bersih sisi panjang dan pendek :

$$\beta = \frac{\ln y}{\ln x} = \frac{4362,5}{2925} = 1,491$$

sehingga tebal pelat tidak boleh kurang dari :

$$h = \frac{\ln(0,8 + fy/1500)}{36 + 9.\beta} = \frac{4362,5(0,8 + 240/1500)}{36 + 9.1,491} = 84,745 \text{ mm}$$

tetapi tidak perlu lebih besar sama dengan dari :

$$h = \frac{\ln(0,8 + fy/1500)}{36} = \frac{4362,5(0,8 + 240/1500)}{36} = 116,333 \text{ mm}$$

digunakan tebal pelat atap 10 cm.

- Digunakan tulangan pokok Ø 8 mm
- Penutup beton (Pb) digunakan 20 mm

Tinggi manfaat tulangan pelat lantai :

- Arah x : $dx = h - Pb - \frac{1}{2}\emptyset_{tul.x}$

  $= 100 - 20 - \frac{1}{2}.6$

  $= 77$ mm

- Arah y : $dy = h - Pb - \emptyset_{tul.x} - \frac{1}{2}\emptyset_{tul.y}$

  $= 100 - 20 - 6 - \frac{1}{2}.6$

  $= 71$ mm

Momen – momen yang bekerja pada pelat :

$Mulx = - Mutx = 0,001 . qU . lx^2 . C$

$= 0,001 . 5,272 . 3,125^2 . 54,8 = 2,821$ KNm

Muly = - Muty = 0,001 . qU . lx^2 . C

= 0,001 . 5,272 . $3,125^2$. 37,94 = 1,953 KNm

1) Perencanaan Tulangan lx dan tx

Mulx = - Mutx = 2,821 KNm

$Mu/\phi = 2,281/0,8 = 3,526$ KNm

Rasio Tulangan (ρ)

$$\rho b = \frac{0,85.f'c.\beta_1}{fy}\left(\frac{600}{600+fy}\right) = \frac{0,85.25.0,85}{240}\left(\frac{600}{600+240}\right) = 0,053746$$

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

$$\rho_{max} = 0,75 \cdot \rho b = 0,75 \cdot 0,053746 = 0,04032$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.25} = 11,294$$

Koefisien ketahanan (Rn) :

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{3,526.10^6}{1000.77^2} = 0,595 \text{ Mpa}$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2m.Rn}{fy}}\right) = \frac{1}{11,294}\left(1-\sqrt{1-\frac{2.11,294.0,595}{240}}\right)$$

= 0,00250 < ρ_{max} = 0,04032

< ρ_{min} = 0,00583

1,33 ρ = 1,333 . 0,00250 = 0,00325 < ρ_{min} = 0,00583, maka :

ρ_{pakai} = 1,33 ρ = 0,00325

As_p = ρ_{pakai} . b . d ≥ 0,002 . b . h

= 0,00325 . 1000 . 77

$= 249{,}865\ mm^2 > 200\ mm^2$

dipakai $As_p = 249{,}865\ mm^2$

digunakan tulangan pokok Ø 8 mm, sehingga :

$A_1Ø = ¼ . \pi . D^2 = ¼ . \pi . 8^2 = 50{,}265\ mm^2$

jarak tulangan (s) $= \dfrac{A_1\phi.b}{As_p}$

$= \dfrac{50{,}265.1000}{249{,}865}$

$= 201{,}169mm$

Berdasarkan SK SNI T-15-1991-03, pasal 3.6.4, ayat 2 bahwa jarak antar tulangan pada penampang kritis tidak boleh melebihi dua kali tebal pelat, maka :

dipakai s = 200 mm, maka Tulangan Pokok : **P8 –200**

$As_{ada} = \dfrac{A_1\phi.b}{s} = \dfrac{50{,}265.1000}{190} = 264{,}552\ mm^2$

Kontrol Kapasitas Lentur Pelat (arah x) :

$a = \dfrac{As_{ada}.fy}{0{,}85.f'c.b} = \dfrac{264{,}552.240}{0{,}85.25.1000} = 2{,}988\ mm$

$Mn = As_{ada} . fy\ (d - a/2) \geq 1{,}33.\ Mu/\phi$ (karena $\rho_{pakai} = 1{,}33\rho$)

$= 264{,}552 . 240\ (77 - 2{,}988/2) / 10^6$

$= 4{,}794\ KNm \geq 1{,}33.3{,}526 = 4{,}600KNm$ .....OK !

2) Perencanaan Tulangan ly dan ty

Muly = - Muty = 1,953 KNm

$$Mu/\phi = 1{,}1862/0{,}8 = 2{,}441 \text{ KNm}$$

Rasio Tulangan (ρ)

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25.0{,}85}{240}\left(\frac{600}{600+240}\right) = 0{,}053746$$

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

$$\rho_{max} = 0{,}75 \,.\, \rho b = 0{,}75 \,.\, 0{,}053746 = 0{,}04032$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{240}{0{,}85.25} = 11{,}294$$

Koefisien ketahanan (Rn) :

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{2{,}441.10^6}{1000.71^2} = 0{,}484 \text{ Mpa}$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2m.Rn}{fy}}\right) = \frac{1}{11{,}294}\left(1-\sqrt{1-\frac{2.11{,}294.0{,}484}{240}}\right)$$

$$= 0{,}00204 < \rho_{max} = 0{,}04032$$

$$< \rho_{min} = 0{,}00583$$

$1{,}33\ \rho = 1{,}333 \,.\, 0{,}00204 = 0{,}00271 < \rho_{min} = 0{,}00583$, maka :

$\rho_{pakai} = 1{,}33\ \rho = 0{,}00271$

$As_p = \rho_{pakai} \,.\, b \,.\, d \geq 0{,}002 \,.\, b \,.\, h$

$= 0{,}00271 \,.\, 1000 \,.\, 71$

$= 192{,}410 \text{ mm}^2 < 200 \text{ mm}^2$

dipakai $As_p = 200 \text{ mm}^2$

digunakan tulangan pokok Ø 8 mm, sehingga :

$A_1Ø = ¼ \,.\, \pi \,.\, D^2 = ¼ \,.\, \pi \,.\, 8^2 = 50{,}265 \text{ mm}^2$

jarak tulangan (s) $= \frac{A_1\phi.b}{As_p}$

$= \frac{50,265.1000}{200}$

$= 251,325$ mm

Berdasarkan SK SNI T-15-1991-03, pasal 3.6.4, ayat 2 bahwa jarak antar tulangan pada penampang kritis tidak boleh melebihi dua kali tebal pelat, maka :

dipakai s = 200 mm, maka Tulangan Pokok : **P8 – 200**

$As_{ada} = \frac{A_1\phi.b}{s} = \frac{50,265.1000}{200} = 251,325\ mm^2$

Kontrol Kapasitas Lentur Pelat (arah y) :

$a = \frac{As_{ada}.fy}{0,85.f'c.b} = \frac{251,325.240}{0,85.25.1000} = 2,838$ mm

$Mn = As_{ada} . fy\ (d - \frac{a}{2}) \geq 1,33.\frac{Mu}{\phi}$ (karena $\rho_{pakai} = 1,33\rho$)

$= 251,325 . 240\ (71 - \frac{2,838}{2}) / 10^6$

$= 4,197$ KNm $\geq 1,33.2,441 = 3,247$ KNm .....OK !

3) Perencanaan Tulangan Susut Pelat Atap

$As_{susut} = 0,002 . b . h$

$= 0,002 . 1000 . 100 = 200\ mm^2$

dugunakan tulangan bagi Ø 8 mm, sehingga :

$A_1Ø = ¼ . \pi . D^2 = ¼ . \pi . 8^2 = 50,265\ mm^2$

jarak tulangan susut (s) = $\frac{A_1 \phi . b}{As_{susut}}$

$$= \frac{50{,}265.1000}{200} = 251{,}325 \text{ mm}$$

dipakai s bagi = 200 mm, maka Tulangan Bagi : **P8 – 200**

4.3 Perencanaan Struktur Portal Dengan Daktilitas Penuh

Pada perencanaan ulang gedung Fakultas Teknik Industri ini, untuk perencanaan portal dianalisis dengan SAP2000 dengan analisis struktur tiga (3) Dimensi. dan beban yang bekerja pada struktur adalah sebagai berikut :

1. Beban mati

- Pembebanan pelat lantai :

  Beban pelat lantai (QD) = 4,51 KN/m^2

  Beban Plafond = 0,18 KN/m^2 +

  = 4,69 KN/m^2

- Pembebanan pelat atap :

  Beban pelat atap (QD) = 3,06 KN/m^2

  Beban plafond = 0,18 KN/m^2 +

  = 3,24 KN/m^2

- Beban Dinding ½ bata = 2,5 KN/m^2

2. Beban hidup

Beban hidup pelat lantai untuk ruang kuliah = 2,5 KN/m^2

Beban hidup pelat atap = 1,0 KN/m^2

4.3.1 Perhitungan Gaya Geser Dasar Horizontal Total Akibat Gempa

Gaya geser dasar horizontal akibat gempa dipengaruhi oleh berat total dari keseluruhan struktur yang direncanakan ditambah dengan beban hidup yang bakerja. Sesuai fungsi penggunaan gedung yaitu sebagai gedung pendidikan, maka menurut Peraturan Pembebanan Indonesia 1983 (Tabel 3.3) untuk perencanaan beban gempa, beban hidup direduksi sebesar 0,5.

a. Berat total bangunan

1) Lantai 2

- Beban mati :

Pelat lantai	= 642,175 . 4,69	= 3011,801 KN
Balok	= 0,35 . (0,7 – 0,12) . 200,625 . 24	= 977,445 KN
	= 0,25 . (0,45 . 0,12) . 272,350 . 24	= 539,253 KN
Kolom	= 0,4 . 0,7 . 3,9 . 26 . 24	= 681,408 KN
Dinding	= 180,24 . 3,9 . 2,5	= 1757,34 KN +
		WD = 6967,247 KN

- Beban hidup :

Beban Hidup pelat lantai (WL) = 0,5 . 2,5 .642,175 = 802,719 KN

Wt_1 = 6967,247 + 802,719 = 7769,966 KN

2) Lantai 3

- Beban mati :

Pelat lantai	= 633,523 . 4,69	= 2971,223 KN
Balok	= 0,35 . (0,7 – 0,12) . 198,675 . 24	= 967,945 KN
	= 0,25 . (0,45 . 0,12) . 271,050 . 24	= 536,679 KN
Kolom	= 0,4 . 0,7 . 3,9 . 26 . 24	= 681,408 KN
Dinding	= 180,24 . 3,9 . 2,5	= 1757,34 KN +
	WD	= 6914,595 KN

- Beban hidup :

Beban Hidup pelat lantai (WL) = 0,5 . 2,5 .633,523 = 791,904 KN

Wt_2 = 6914,595 + 791,904 = 7706,498 KN

3) Lantai 4

- Beban mati :

Pelat lantai	= 633,523 . 4,69	= 2971,223 KN
Balok	= 0,35 . (0,7 – 0,12) . 198,675 . 24	= 967,945 KN
	= 0,25 . (0,45 . 0,12) . 271,050 . 24	= 536,679 KN
Kolom	= 0,4 . 0,7 . 3,9 . 26 . 24	= 681,408 KN
Dinding	= 180,24 . 3,9 . 2,5	= 1757,34 KN
	= 37 . 1,54 . 2,5	= 142,450 KN +
	WD	= 7057,045 KN

- Beban hidup :

Beban Hidup pelat lantai (WL) = 0,5 . 2,5 .633,523 = 791,904 KN

Wt_3 = 7057,045 + 791,904 = 7848,949 KN

4) Pelat Atap

- Beban mati :

Pelat lantai	= 237,272 . 3,24	= 768,761 KN
Balok	= 0,35 . 0,7 . 5,5 . 24	= 70,875 KN
	= 0,25 . 0,45 . 26,24 . 24	= 32,340 KN
	= 0,35 . (0,7 – 0,1). 146,425 . 24	= 737,982 KN
	= 0,25 . (0,45 – 0,1). 89,3 . 24	= 187,530 KN
Kolom	= 0,35 . 0,35 . 1,54 . 2,5	= 90,552 KN
Dinding	= 162 . 1,54 . 2,5	= 623,700 KN
Lap Kedap	= 202,897 . 0,03 . 24	= 146,086 KN +
		WD = 2657,826 KN

- Beban hidup :

Beban Hidup pelat lantai (WL) = 0,5 . 202,897 . 1,0 = 101,449 KN

Wt_4 = 2657,826 + 101,449 = 2757,275 KN

5) Ring Balk

- Beban mati :

Berat atap	= (532,488/cos 45) x 0,5	= 376,526 KN
Berat kuda – kuda		= 58,625 KN
Balok	= 0,25 . 0,45 . 167,75 . 24	= 452,925 KN

Berat baut dan plat sambung = 11,725 KN +

WD = 899,801 KN

- Beban hidup :

Beban Hidup atap (WL) = 0,5 x (532,440/cos 45) x 0,2 = 75,298 KN +

WL = 75,298 KN

$Wt_5 = 899,801 + 75,298 = 975,099$ KN

$W_{total} = Wt_1 + Wt_2 + Wt_3 + Wt_4 + Wt_5$

$= 7769,966 + 7706,498 + 7848,949 + 2757,275 + 975,099$

$= 27057,787$ KN

b. Waktu Getar Bangunan (T)

Waktu getar struktur untuk struktur portal terbuka beton bertulang dapat dihitung dengan :

$T = 0,06 \cdot H^{3/4} = 0,06 \cdot 17,1^{3/4} = 0,505$ dt

c. Koefisien Gempa Dasar (C)

Pada Redisain ini bangunan berada dalam wilayah gempa 3 pada kondisi tanah keras. Waktu getar struktur (T) = 0,505 dt, maka berdasarkan Respon spectrum wilayah 3 didapatkan koefisien gempa dasar (C) = 0,05

d. Faktor keutamaan (I) dan faktor jenis struktur (K)

Berdasarkan funsi bangunan, maka faktor keutamaan bangunan (I) diambil = 1,0. (PPKGURG 1987, tabel 2.1)Sedangkan untuk faktor jenis struktur (K) diambil = 1,0 yaitu untuk portal daktail.

e. Gaya Geser Horizontal Akibat Gempa (V)

Gaya geser horizontal akibat gempa yang bekerja dapat dihitung dengan :

$V = C \cdot I \cdot K \cdot Wt = 0{,}05 \cdot 1{,}0 \cdot 1{,}0 \cdot 27057{,}787 = 1352{,}889$ KN

f. Distribusi gaya horizontal total akibat gempa ke sepanjang tinggi gedung

1) Arah x

$$H/B = \frac{17{,}1}{47{,}875} = 0{,}357 < 3, \text{ maka :}$$

$$F_i x = \frac{Wi.hi}{\sum Wi.hi}.V$$

2) Arah y

$$H/B = \frac{17{,}1}{15{,}125} = 1{,}131 < 3, \text{ maka :}$$

$$F_i y = \frac{Wi.hi}{\sum Wi.hi}.V$$

Tabel 4.9 Distribusi Gaya Geser Horizontal total akibat gempa arah x dan arah y

Tingkat					
Ring Balk	17,1	975,099	1352,889	16674,193	93,646
Pelat atap	15,56	2757,275	1352,889	42903,199	240,953
Lantai 4	11,66	7848,949	1352,889	91518,745	513,987
Lantai 3	7,76	7706,498	1352,889	59802,424	335,862
Lantai 2	3,86	7769,966	1352,889	29992,069	168,441
				240890,630	1352,889

Tabel 4.10 Distribusi Gaya Geser Horizontal portal arah x kanan

Tingkat	Arah X kn			
	Portal 1	Portal 2	Portal 3	Portal 4
	0,327 Fi x (KN)	0,044 Fi x (KN)	0,042 Fi x (KN)	0,115 Fi x (KN)
Ring Balk	30,622	4,121	3,933	10,769
	Portal 1	Portal 2	Portal 3	Portal 4
	0,557 Fi x (KN)	0,089 Fi x (KN)	0,06 Fi x (KN)	0,072 Fi x (KN)
Pelat atap	134,211	21,445	14,457	17,349
	Portal 1	Portal 2	Portal 3	Portal 4
	0,377 Fi x (KN)	0,065 Fi x (KN)	0,364 Fi x (KN)	0,057 Fi x (KN)
Lantai 4	193,773	33,409	187,091	29,297
Lantai 3	126,617	21,831	122,254	19,144
	Portal 1	Portal 2	Portal 3	Portal 4
	0,386 Fi x (KN)	0,064 Fi x (KN)	0,359 Fi x (KN)	0,057 Fi x (KN)
Lantai 2	65,018	10,781	60,471	9,601

Tabel 4.11 Distribusi Gaya Geser Horizontal portal arah x kiri

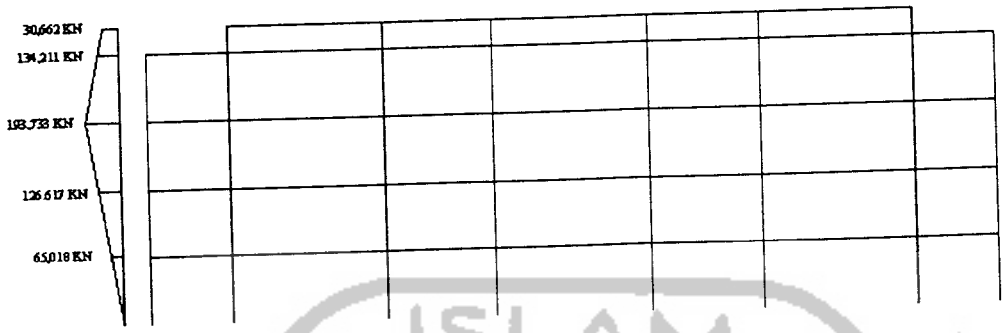
Tingkat	Arah X kl			
	Portal 1	Portal 2	Portal 3	Portal 4
	0,327 Fi x (KN)	0,044 Fi x (KN)	0,042 Fi x (KN)	0,115 Fi x (KN)
Ring Balk	30,622	4,121	3,933	10,769
	Portal 1	Portal 2	Portal 3	Portal 4
	0,557 Fi x (KN)	0,089 Fi x (KN)	0,06 Fi x (KN)	0,072 Fi x (KN)
Pelat atap	134,211	21,445	14,457	17,349
	Portal 1	Portal 2	Portal 3	Portal 4
	0,377 Fi x (KN)	0,065 Fi x (KN)	0,364 Fi x (KN)	0,057 Fi x (KN)
Lantai 4	193,773	33,409	187,091	29,297
Lantai 3	126,617	21,831	122,254	19,144
	Portal 1	Portal 2	Portal 3	Portal 4
	0,386 Fi x (KN)	0,064 Fi x (KN)	0,359 Fi x (KN)	0,057 Fi x (KN)
Lantai 2	65,018	10,781	60,471	9,601

Tabel 4.12 Distribusi Gaya Geser Horizontal Portal Arah Y kanan

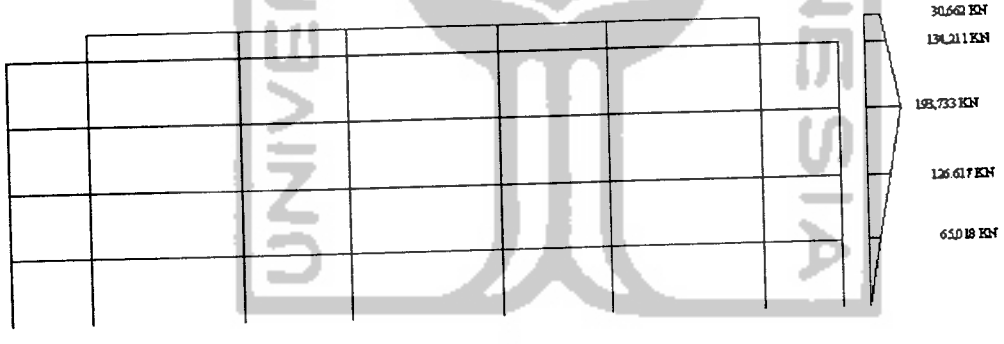
Tingkat	Arah Y ka							
	Portal A	Portal B	Portal C	Portal D	Portal E	Portal F	Portal G	Portal H
	0	0,113 Fi y (KN)	0,194 Fi y (KN)	0,194 Fi y (KN)	0,194 Fi y (KN)	0,194 Fi y (KN)	0,113 Fi y (KN)	0
Ring Balk		10.582	18.167	18.167	18.167	18.167	10.582	
	Portal A	Portal B	Portal C	Portal D	Portal E	Portal F	Portal G	Portal H
	0,113 Fi y (KN)	0,158 Fi y (KN)	0,036 Fi y (KN)	0,036 Fi y (KN)	0,036 Fi y (KN)	0,036 Fi y (KN)	0,158 Fi y (KN)	0,113 Fi y (KN)
Pelat atap	27.228	38.071	8.674	8.674	8.674	8.674	38.071	27.228
	Portal A	Portal B	Portal C	Portal D	Portal E	Portal F	Portal G	Portal H
	0,042 Fi y (KN)	0,142 Fi y (KN)	0,258 Fi y (KN)	0,258 Fi y (KN)	0,258 Fi y (KN)	0,258 Fi y (KN)	0,142 Fi y (KN)	0,042 Fi y (KN)
Lantai 4	21.587	72.986	132.609	132.609	132.609	132.609	72.986	21.587
Lantai 3	14.106	47.692	86.652	86.652	86.652	86.652	47.692	14.106
	Portal A	Portal B	Portal C	Portal D	Portal E	Portal F	Portal G	Portal H
	0,042 Fi y (KN)	0,14 Fi y (KN)	0,156 Fi y (KN)	0,156 Fi y (KN)	0,156 Fi y (KN)	0,160 Fi y (KN)	0,147 Fi y (KN)	0,044 Fi y (KN)
Lantai 2	7.075	23.582	26.277	26.277	26.277	26.951	24.761	7.411

Tabel 4.13 Distribusi Gaya Geser Horizontal Portal Arah Y kiri

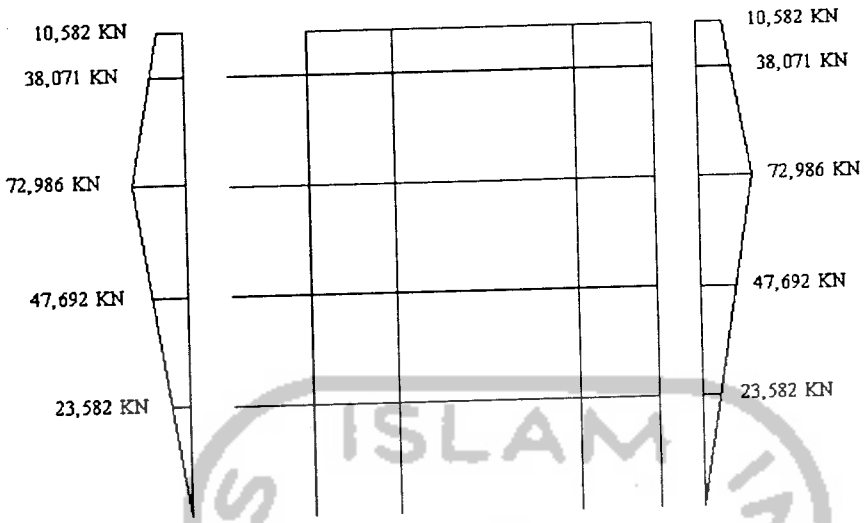
Tingkat	Arah Y ki							
	Portal A	Portal B	Portal C	Portal D	Portal E	Portal F	Portal G	Portal H
	0	0,113 Fi y (KN)	0,194 Fi y (KN)	0,194 Fi y (KN)	0,194 Fi y (KN)	0,194 Fi y (KN)	0,113 Fi y (KN)	0
Ring Balk		10.582	18.167	18.167	18.167	18.167	10.582	
	Portal A	Portal B	Portal C	Portal D	Portal E	Portal F	Portal G	Portal H
	0,113 Fi y (KN)	0,158 Fi y (KN)	0,078 Fi y (KN)	0,078 Fi y (KN)	0,078 Fi y (KN)	0,078 Fi y (KN)	0,158 Fi y (KN)	0,113 Fi y (KN)
Pelat atap	27.228	38.071	18.794	18.794	18.794	18.794	38.071	27.228
	Portal A	Portal B	Portal C	Portal D	Portal E	Portal F	Portal G	Portal H
	0,042 Fi y (KN)	0,142 Fi y (KN)	0,258 Fi y (KN)	0,258 Fi y (KN)	0,258 Fi y (KN)	0,258 Fi y (KN)	0,142 Fi y (KN)	0,042 Fi y (KN)
Lantai 4	21.587	72.986	132.609	132.609	132.609	132.609	72.986	21.587
Lantai 3	14.106	47.692	86.652	86.652	86.652	86.652	47.692	14.106
	Portal A	Portal B	Portal C	Portal D	Portal E	Portal F	Portal G	Portal H
	0,042 Fi y (KN)	0,14 Fi y (KN)	0,156 Fi y (KN)	0,156 Fi y (KN)	0,156 Fi y (KN)	0,160 Fi y (KN)	0,147 Fi y (KN)	0,044 Fi y (KN)
Lantai 2	7.075	23.582	26.277	26.277	26.277	26.951	24.761	7.411



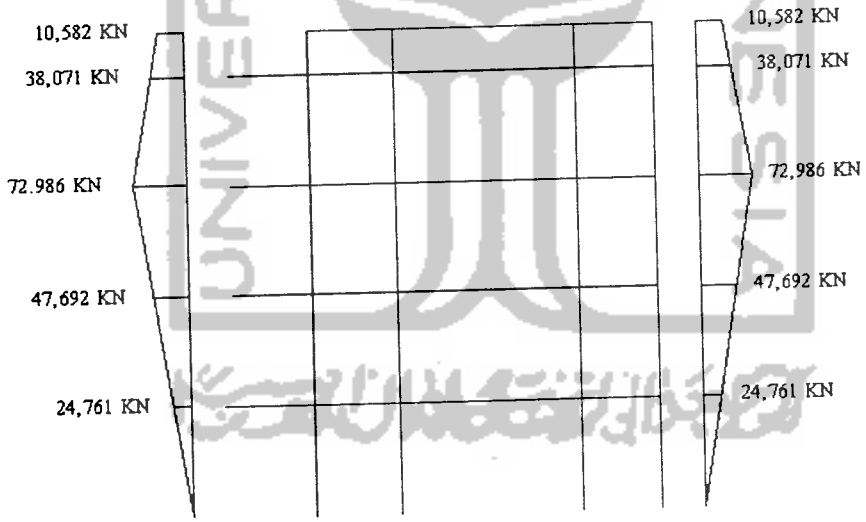
Gambar 4.14 Distribusi gempa Portal as 1 (x kiri)



Gambar 4.15 Distribusi gempa Portal as 1 (x kanan)



Gambar.4.16 Distribusi gempa Portal as B (y kiri dan y kanan)



Gambar.4.17 Distribusi gempa Portal as G (y kiri dan y kanan)

4.4 Disain Balok

4.4.1 Disain Tulangan Balok Induk

4.4.1.1 Disain Tulangan Lentur Balok Induk

A. Momen Rencana Balok

Momen rencana balok diambil yang terbesar dari hasil kombinasi beban sebagai berikut:

1. $1{,}2\ M_D + 1{,}6\ M_L$
2. $1{,}05\ (M_D + 0{,}9\ M_L \pm M_E)$

B. Disain balok induk ukuran 350/700

Berikut diberikan contoh perhitungan balok BI lantai 2 bentang 178-179 dengan spesifikasi :

- f'c = 25 Mpa
- fy deform = 400 Mpa
- ϕ tul pokok = 22 mm
- ϕ tul sengkang = 10 mm
- β_1 = 0,85 (f'c < 30 Mpa)

Perhitungan :

$$\rho_b = \frac{0{,}85.fc'}{fy}\beta_1\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25.0{,}85}{400}\left(\frac{600}{600+400}\right) = 0{,}0271$$

$\rho_{maks} = 0{,}75\ \rho b = 0{,}75.0{,}0271 = 0{,}0203$

rasio tulangan rencana = 0,5. $\rho_{maks} = 0{,}5.0{,}0203 = 0{,}01015$

$$\rho_{min} = {}^{1,4}/_{fy} = {}^{1,4}/_{400} = 0,0035$$

$$m = \frac{fy}{0,85.fc'} = \frac{400}{0,85.25} = 18,824$$

$$Rn = \rho.fy.\left(1 - \frac{1}{2}.\rho.m\right) = 0,01015.400.\left(1 - \frac{1}{2}.0,01015.18,824\right) = 3,675 \text{ Mpa}$$

1. Tulangan Tumpuan Kiri Mu = 362,62

Mu = 362,62

$$\frac{Mu}{\phi} = \frac{362,62}{0,8} = 453,275 \text{ KNm}$$

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{453,275.10^6}{3,675.350}} = 593,633 \text{ mm}$$

d_{ada} = h – d' (d' = 100 mm, diasumsikan menggunakan tulangan 2 lapis)

= 700 – 100 = 600 mm > d_{perlu}, maka dipakai tulangan sebelah.

$$Rn_{ada} = \frac{Mu/\phi}{b.d^2} = \frac{453,275.10^6}{350.600^2} = 3,597 \text{ Mpa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn} = \frac{3,597}{3,675}.0,01015 = 0,0099 > \rho_{min} = 0,0035$$

$$< \rho_{maks} = 0,0203$$

$As_{perlu} = \rho_{ada}.b.d = 0,0099.350.600 = 2086,504 \text{ mm}^2$

Dipakai tulangan Ø22 dengan A1Ø = 380,133 mm^2

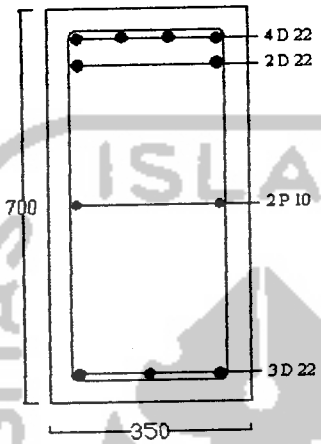
$$\text{jumlah tulangan (n)} = \frac{As_{perlu}}{A1\phi} = \frac{2086,504}{380,133} = 5,489 \text{ batang}$$

dipakai 6D22, maka $As_{ada} = 6 \,.\, 380,133 = 2280,796 \text{ mm}^2 > As_{perlu}$

$$s = \frac{b - 2.Pb - 2.\phi \text{ sengkang} - n.\phi \text{ tul.}}{(n-1)}$$

$$= \frac{350 - 2.40 - 2.10 - 6.22}{(6-1)}$$

= 23,6 mm < 25 mm, maka dipakai tulangan 2 lapis



Gambar 4.18 tulangan pokok balok tumpuan Kiri

Kontrol kapasitas momen nominal :

$$a = \frac{As_{ada}.fy}{0,85.fc'.b} = \frac{2280,796.400}{0,85.25.350} = 122,665 \text{ mm}$$

$$Mn = As_{ada}.fy.(d - {}^{a}/_{2})$$

$$= 2280,796.400.(600 - {}^{122,665}/_{2})$$

$$= 491,436 \text{ KNm} > {}^{Mu}/_{\phi} = 459,4 \text{ KNm} \rightarrow \text{OK!}$$

2. Tulangan Lapangan Mu = 222,66

Mu = 222,62 KNm

$$\frac{Mu}{\phi} = \frac{222,62}{0,8} = 278,275 \text{ KNm}$$

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{278,275.10^6}{3,675.350}} = 465,131 \text{ mm}$$

d_{ada} = h – d' (d' = 100 mm, diasumsikan menggunakan tulangan 2 lapis)

= 700 – 100 = 600 mm > d_{perlu}, maka dipakai tulangan sebelah.

$$Rn_{ada} = \frac{Mu/\phi}{b.d^2} = \frac{278,275.10^6}{350.600^2} = 2,208 \text{ Mpa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn} = \frac{2,208}{3,675}.0,01015 = 0,0061 > \rho_{min} = 0,0035$$

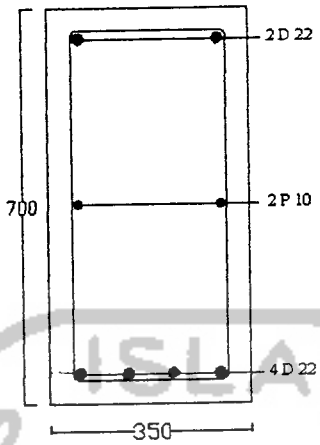
$$< \rho_{maks} = 0,0203$$

$As_{perlu} = \rho_{ada}.b.d = 0,0061.350.600 = 1281 \text{ mm}^2$

Dipakai tulangan Ø22 dengan A1Ø = 380,133 mm^2

$$\text{jumlah tulangan (n)} = \frac{As_{perlu}}{A1\phi} = \frac{1281}{380,133} = 3,369 \text{ batang}$$

dipakai 4D22, maka $As_{ada} = 4 . 380,133 = 1520,532 \text{ mm}^2 > As_{perlu}$



Gambar 4.19 tulangan pokok balok lapangan

Kontrol kapasitas momen nominal :

$$a = \frac{As_{ada}.fy}{0,85.fc'.b} = \frac{1520,532.400}{0,85.25.350} = 81,776 \text{ mm}$$

$$Mn = As_{ada}.fy.(d - {}^{a}/_{2})$$

$$= 1520,532.400.(600 - {}^{81,776}/_{2})$$

$$= 340,059 \text{ KNm} > {}^{Mu}/_{\phi} = 278,275 \text{ KNm} \rightarrow \textbf{OK!}$$

3. Tulangan Tumpuan Kanan Mu = 293,02

Mu = 293,02 KNm

$$\frac{Mu}{\phi} = \frac{293,02}{0,8} = 366,275 \text{ KNm}$$

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{366{,}275.10^6}{3{,}675.350}} = 533{,}631 \text{ mm}$$

$d_{ada} = h - d'$ (d' = 100 mm, diasumsikan menggunakan tulangan 2 lapis)

$= 700 - 100 = 600$ mm $> d_{perlu}$, maka dipakai tulangan sebelah.

$$Rn_{ada} = \frac{Mu/\phi}{b.d^2} = \frac{366{,}275.10^6}{350.600^2} = 2{,}907 \text{ Mpa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn} = \frac{2{,}907}{3{,}675}.0{,}01015 = 0{,}008 > \rho_{min} = 0{,}0035$$

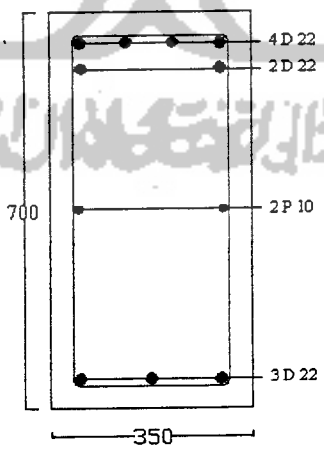
$$< \rho_{maks} = 0{,}0203$$

$As_{perlu} = \rho_{ada}.b.d = 0{,}008.350.600 = 1686{,}028 \text{ mm}^2$

Dipakai tulangan Ø22 dengan $A1Ø = 380{,}133 \text{ mm}^2$

$$\text{jumlah tulangan (n)} = \frac{As_{perlu}}{A1\phi} = \frac{1686{,}028}{380{,}133} = 4{,}435 \text{ batang}$$

dipakai 6D22, maka $As_{ada} = 6 \,.\, 380{,}133 = 2280{,}798 \text{ mm}^2 > As_{perlu}$



Gambar 4.20 tulangan pokok balok tumpuan kanan

Kontrol kapasitas momen nominal :

$$a = \frac{As_{ada}.fy}{0,85.fc'.b} = \frac{2280,798.400}{0,85.25.350} = 122,665 \text{ mm}$$

$$Mn = As_{ada}.fy.(d - {}^{a}/_{2})$$

$$= 2280,798.400.(600 - {}^{122,665}/_{2})$$

$$= 491,437 \text{ KNm} > {}^{Mu}/_{\phi} = 366,275 \text{ KNm} \rightarrow \textbf{OK!}$$

4. Momen Nominal Aktual Balok

1) Momen Aktual Balok Negatif

tulangan atas = 6D22 dengan $As_{ada} = 2280 \text{ mm}^2$

tulangan bawah = 3D22 dengan $As_{ada} = 1140 \text{ mm}^2$

$$\rho = \frac{As_{ada}}{b.d_{pakai}} = \frac{2280}{350.600} = 0,011$$

$$\rho' = \frac{As'_{ada}}{b.d_{pakai}} = \frac{1140}{350.600} = 0,0036$$

$$\rho_1 = \rho - \rho' = 0,011 - 0,0036 = 0,0074$$

$$fs' = 600\left\{1 - \frac{0,85.f'c.\beta_1}{(\rho - \rho').fy}\frac{d'}{d}\right\} = 600\left\{1 - \frac{0,85.25.0,85}{0,0074.400}\frac{100}{600}\right\}$$

$$= 10,22 \text{ Mpa}$$

fs' < fy dipakai fs' = 10,22 Mpa

$$a = \frac{(As_{ada}.fy) - (As'_{ada}.fs')}{0,85.fc'.b} = \frac{(2280.400) - (1140.10,22)}{0,85.25.350}$$

$$= 121,578 \text{ mm}^2$$

$$Mn_1 = (As_{ada}.fy - As'_{ada}.fs').(d - {}^{a}/_{2})$$

$= (2280.400 - 1140.10,22).(600 - {}^{121,578}/_{2}) = 487,572$ KNm

$Mn_2 = (As'_{ada}.fs').(d - d') = (1140.10,22).(600 - 100) = 3,884$ KNm

$Mnak^- = Mn1 + Mn_2 = 487,572 + 3,884 = 491,456$ KNm

2) Momen Aktual Balok Positif

$$\rho_{aktual} = \frac{As_{ada}}{b.d_{pakai}} = \frac{1520,532}{350.600} = 0,00724$$

$Rn = \rho.fy(1 - 1/2.\rho..m) = 0,00724.400.(1-1/2.\ 0,00724.18,824) = 2,697$ Mpa

$Mnak^+ = Rn.b.d^2 = 2,697.350.600^2.10^{-6} = 339,822$ KNm

4.4.1.2 Disain Tulangan Geser Balok Induk

Adapun syarat penentuan gaya geser rencana balok adalah sebagai berikut:

$$Vu,b = 0,7\ \phi_0 \left[\frac{M_{nak,b} + M_{nak,b'}}{Ln}\right] + 1,05.Vg$$

Tetapi tidak lebih besar dari $Vu,b = 1,07\ (\ V_{D,b} + V_{L,b} + 4/k\ .V_{E,b}\ )$

$V_D = 132,59$ KN ; $V_L = 37,26$ KN ; $V_E = 44,72$ KN

$$Vu,b = 0,7\ \phi_0 \left[\frac{M_{nak,b} + M_{nak,b'}}{Ln}\right] + 1,05.Vg$$

$$Vu,b = 0,7.1,25\left[\frac{491,456 + 339,822}{5,65}\right] + 1,05(132,59 + 37,26) = 307,081 \text{ KN}$$

Dengan syarat tidak lebih besar dari :

$Vu,b = 1,07\ (\ 132,59 + 37,26 + 4/1\ .\ 44,72\ ) = 373,141$ KN

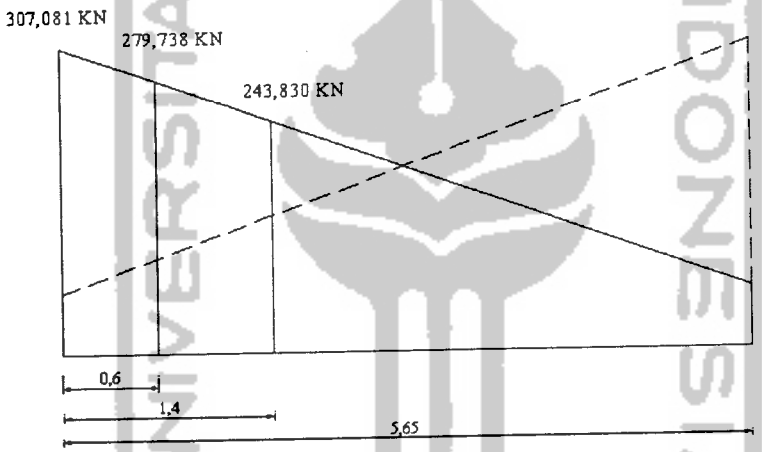
$$Vu,b_{\text{pakai}} = \left[1{,}05Vg - 0{,}7\phi_0\left(\frac{Mnak,b + Mnak,b'}{Ln}\right)\right] +$$

$$\frac{Ln-d}{Ln}\left[Vu,b - \left[1{,}05Vg - 0{,}7\phi_0\left[\frac{Mnak,b + Mnak,b'}{Ln}\right]\right]\right]$$

$$= \left[1{,}05.169{,}85 - 0{,}7.1{,}25\left(\frac{491{,}456 + 339{,}822}{5{,}65}\right)\right] +$$

$$\frac{5{,}65-0{,}6}{5{,}65}\left[307{,}081 - \left[1{,}05.169{,}85 - 0{,}7.1{,}25\left[\frac{491{,}456 + 339{,}822}{5{,}65}\right]\right]\right]$$

$$= 279{,}738 \text{ KN}$$



Gambar 4.21 Diagram tegangan geser balok

1) dalam daerah sendi plastis

Vu,b untuk perencanaan di dalam daerah sendi plastis diambil sejauh d dari tumpuan, yaitu :

Vu,b = 279,738 KN

Vc = 0

$$\frac{Vu,b}{\phi} = \frac{279,738}{0,6} = 466,230 \text{ KN}$$

Digunakan sengkang $\square\ \phi$ 10 mm, maka : $Av = 2 \cdot \frac{1}{4} \cdot \pi \cdot 10^2 = 157 \text{ mm}^2$

Jarak sengkang :

$$s \leq \frac{Av.fy.d}{Vu,b/\phi - Vc} = \frac{157.240.600}{466,230 - 0} \cdot 10^{-3} = 48,491 \text{ mm}$$

$$\leq d/4 = 600/4 = 150 \text{ mm}$$

Jadi dipakai tulangan geser **2P10 – 90 mm**

2) Diluar sendi plastis

Diambil jarak sejauh 2h = 2.700 = 1400 mm dengan Vu,b = 243,830 KN

$$Vc = 1/6.\sqrt{fc'}.b.d = 1/6.\sqrt{25}.350.600 = 175 \text{ KN}$$

$$Vs = \frac{Vu,b}{\phi} - Vc = \frac{243,830}{0,6} - 175 = 231,383 \text{ KN}$$

Digunakan sengkang $\square\ \phi$ 10 mm, maka : $Av = 2 \cdot \frac{1}{4} \cdot \pi \cdot 10^2 = 157 \text{ mm}^2$

Jarak sengkang :

$$s \leq \frac{Av.fy.d}{Vs} = \frac{157.240.600}{231,383} \cdot 10^{-3} = 97,708 \text{ mm}$$

$$\leq d/2 = 600/2 = 300 \text{ mm}$$

$$\leq 600 \text{ mm}$$

Jadi dipakai tulangan geser **P10 – 90 mm**

4.4.1.3 Perencanaan Tulangan Torsi

Tu = 28,54 KNm

$\sum x^2.y = 350^2 . 700 = 85,75.10^6 \text{ mm}^3$

Pada redisain ini komponen struktur portal merupakan komponen statis tak tentu. Untuk komponen statis tak tentu setelah terjadi retak akibat torsi , dalam rangka untuk mencapai keseimbangan terjadi redistribusi tegangan torsional yang mempengaruhi komponen lain yang bertemu pada satu titik buhul. Maka untuk menganalisis torsi dipakai torsi keserasian.

Kemampuan penampang beton menahan torsi untuk torsi keserasian :

$$Tu,b = \phi(\frac{1}{9}\sqrt{f'c}.\sum x^2.y = 0,6.(\frac{1}{9}\sqrt{25}.85.75.10^6$$

= 28,583 KNm > Tu = 28,54 KNm , tulangan torsi diabaikan.

4.4.1.4 Perencanaan Tulangan Torsi Balok Induk

Momen torsi terfaktor (balok 265 – 267) Tu = 29,00

$$\text{Tu maks} = \phi.\left[\left(\frac{1}{20}\sqrt{f'c}\right)Ex^2.y\right] = 0,6.\left[\left(\frac{1}{20}\sqrt{25}\right)350^2.700\right] = 12,8625 \text{ KNm}$$

Tu = 29,00 > Tu maks = 12,8625 KNm, maka perlu tulangan torsi.

Karena merupakan torsi keserasian, menurut SK – SNI boleh direncanakan terhadap momen torsi sebagai berikut :

$$Tu = \phi.\left[\left(\frac{1}{3}\sqrt{f'c}\right)\frac{1}{3}Ex^2.y\right] = 0,6.\left[\left(\frac{1}{3}\sqrt{25}\right)\frac{1}{3}.350^2.700\right] = 28,583 \text{ KNm}$$

Maka dipakai = 28,8625 KNm

- **Untuk Daerah sendi plastis**

Vu = 272 KNm

Sebagai tulangan geser dan tulangan torsi digunakan sengkang D 10 mm

1. Perencanaan sengkang torsi

d = 600 mm

$$Ct = \frac{b.d}{\Sigma x^2 y} = \frac{350.600}{\Sigma 350^2 700} = 0{,}0024 \text{ /mm}$$

Sumbangan beton dalam menahan torsi :

$$Tc = \frac{\left(\frac{1}{15}.\sqrt{f'c}\right)\Sigma x^2 y}{\sqrt{1+\left(\frac{0{,}4.Vu}{Ct.Tu}\right)^2}} = \frac{\left(\frac{1}{15}.\sqrt{25}\right)\Sigma 350^2 700}{\sqrt{1+\left(\frac{0{,}4.272.10^3}{0{,}0024.28{,}863.10^6}\right)^2}}$$

= 15,351 KNm

Torsi yang ditahan tulangan torsi :

$$Ts = \frac{Tu}{\phi} - Tc = \frac{28{,}863}{0.6} - 15{,}351 = 32{,}754 \text{ KNm}$$

X1 = b – 2.(pb + 0,5 . D sengkang) = 350 – 2.(40 + 0,5.10) = 260 mm

X2 = h – 2.(pb + 0,5 . D sengkang) = 700 – 2.(40 + 0,5.10) = 610 mm

$$\alpha t = \frac{1}{3}\left(2 + \frac{y1}{x1}\right) = \frac{1}{3}\left(2 + \frac{610}{210}\right) = 1{,}45 < 1{,}5$$

$$\frac{At}{s} = \frac{Ts}{\alpha t.x1.y1.fy} = \frac{32{,}754}{1{,}45.260.610.400} = 0{,}35 \text{ mm}^2\text{/mm jarak/kaki}$$

2. Perencanaan Sengkang geser

$$\frac{Vu}{\phi} = \frac{272}{0,6} = 453,333 \text{ KN}$$

Sumbangan beton dalam menahan geser :

$$Vc = \frac{\frac{1}{6}.\sqrt{f'c}.b.d}{\sqrt{1+\left(2,5.Ct.\frac{Tu}{Vu}\right)^2}} = \frac{\frac{1}{6}.\sqrt{25}.350.600}{\sqrt{1+\left(2,5.0,0024.\frac{28,863.10^6}{272.10^3}\right)^2}}$$

$$= 147,617 \text{ KN}$$

Geser yang ditahan tulangan geser :

$$Vs = \frac{Vu}{\phi} - Vc = 453,333 - 147,617 = 305,716 \text{ KN}$$

$$\frac{Av}{s} = \frac{Vs}{fy.d} = \frac{305,716.10^3}{400.600} = 1,27 \text{ mm}^2/\text{mm jarak/ dua kaki}$$

3. Perencanaan sengkang geser dan torsi (gabungan)

$$\frac{Avt}{s} = 2.\frac{At}{s} + \frac{Av}{s} = 2.0,35 + 1,27 = 1,97 \text{ mm}^2$$

dipakai sengkang D 10 mm, dengan luas dus kaki Av = 157 mm^2 sehingga jarak sengkang

s = As/Avt = 157 / 1,97 = 79,695 mm

Jarak sengkang maksimum:

S = ¼ .(x1 + y1) = ¼. (260 + 610) = 217,5 mm

Jadi pakai sengkang **P10 – 70**

- **Untuk daerah luar sendi plastis**

Vu = 252,512 KNm

Sebagai tulangan geser dan tulangan torsi digunakan sengkang D 10 mm

1. Perencanaan sengkang torsi

d = 600 mm

$$Ct = \frac{b.d}{\sum x^2 y} = \frac{350.600}{\sum 350^2 700} = 0{,}0024 \text{ /mm}$$

Sumbangan beton dalam menahan torsi :

$$Tc = \frac{\left(\frac{1}{15}.\sqrt{f'c}\right)\sum x^2 y}{\sqrt{1+\left(\frac{0{,}4.Vu}{Ct.Tu}\right)^2}} = \frac{\left(\frac{1}{15}.\sqrt{25}\right)\sum 350^2 700}{\sqrt{1+\left(\frac{0{,}4.252{,}512.10^3}{0{,}0024.28{,}863.10^6}\right)^2}}$$

= 16,166 KNm

Torsi yang ditahan tulangan torsi :

$$Ts = \frac{Tu}{\phi} - Tc = \frac{28.863}{0.6} - 16{,}166 = 31{,}939 \text{ KNm}$$

X1 = b – 2.(pb + 0,5 . D sengkang) = 350 – 2.(40 + 0,5.10) = 260 mm

X2 = h – 2.(pb + 0,5 . D sengkang) = 700 – 2.(40 + 0,5.10) = 610 mm

$$\alpha t = \frac{1}{3}.\left(2+\frac{yl}{xl}\right) = \frac{1}{3}.\left(2+\frac{610}{210}\right) = 1{,}45 < 1{,}5$$

$$\frac{At}{s} = \frac{Ts}{\alpha t.x1.y1.fy} = \frac{31{,}939}{1{,}45.260.610.400} = 0{,}34 \text{ mm}^2\text{/mm jarak/kaki}$$

2. Perencanaan Sengkang geser

$$\frac{Vu}{\phi} = \frac{252{,}512}{0{,}6} = 420{,}853 \text{ KN}$$

Sumbangan beton dalam menahan geser :

$$Vc = \frac{\frac{1}{6}.\sqrt{f'c}.b.d}{\sqrt{1+\left(2{,}5.Ct.\frac{Tu}{Vu}\right)^2}} = \frac{\frac{1}{6}.\sqrt{25}.350.600}{\sqrt{1+\left(2{,}5.0{,}0024.\frac{28{,}863.10^6}{252{,}512.10^3}\right)^2}}$$

$= 144{,}320$ KN

Geser yang ditahan tulangan geser :

$$Vs = \frac{Vu}{\phi} - Vc = 420{,}853 - 144{,}320 = 276{,}532 \text{ KN}$$

$$\frac{Av}{s} = \frac{Vs}{fy.d} = \frac{276{,}532.10^3}{400.600} = 1{,}15 \text{ mm}^2\text{/mm jarak/ dua kaki}$$

2. Perencanaan sengkang geser dan torsi (gabungan)

$$\frac{Avt}{s} = 2.\frac{At}{s} + \frac{Av}{s} = 2.0{,}34 + 1{,}15 = 1{,}83 \text{ mm}^2$$

dipakai sengkang D 10 mm, dengan luas dus kaki Av = 157 mm^2 sehingga jarak sengkang

s = As/Avt = 157 / 1,830 = 85,792 mm

Jarak sengkang maksimum:

S = ¼ .(x1 + y1) = ¼. (260 + 610) = 217,5 mm

Jadi pakai sengkang **P10 – 80**

4.4.2 Disain Tulangan Balok Anak

4.4.2.1 Disain Tulangan Lentur Balok Anak

A. Momen Rencana Balok

Momen rencana balok diambil yang terbesar dari hasil kombinasi beban sebagai berikut:

3. $1{,}2\ M_D + 1{,}6\ M_L$
4. $1{,}05\ (M_D + 0{,}9\ M_L \pm M_E)$

B. Desain balok anak ukuran 250/450

Berikut diberikan contoh perhitungan balok anak lantai 2 bentang 183-185

Dengan spesifikasi :

- f'c = 25 Mpa
- fy deform = 400 Mpa
- Ø tul pokok = 16 mm
- Ø tul sengkang= 8 mm
- B_1 = 0,85 (f'c ≤ 30 Mpa)

Perhitungan :

$$\rho_b = \frac{0{,}85.fc'}{fy}\beta_1\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25.0{,}85}{400}\left(\frac{600}{600+400}\right) = 0{,}0271$$

$\rho_{maks} = 0{,}75\ \rho b = 0{,}75.0{,}0271 = 0{,}0203$

rasio tulangan rencana = 0,5. $\rho_{maks} = 0{,}5.0{,}0203 = 0{,}01015$

$$\rho_{min} = {}^{1,4}\!/_{fy} = {}^{1,4}\!/_{400} = 0{,}0035$$

$$m = \frac{fy}{0{,}85.fc'} = \frac{400}{0{,}85.25} = 18{,}824$$

$$Rn = \rho.fy.\left(1-\frac{1}{2}.\rho.m\right) = 0{,}01015.400.\left(1-\frac{1}{2}.0{,}01015.18{,}824\right) = 3{,}675\,Mpa$$

1. Tulangan Tumpuan Kiri Mu = 108,89 KNm

$$\frac{Mu}{\phi} = \frac{108{,}89}{.0{,}8} = 136{,}113\ KNm$$

$$b\,.\,d^2 = \frac{Mu/\phi}{Rn}$$

dengan b = 250 mm, sehingga

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{136{,}113.10^6}{3{,}675.250}} = 384{,}903\ mm$$

d' = Pb + D sengkang + 0,5 D tulangan

= 40 + 10 + 0,5 . 16 = 58 (d' = 50-70 mm, anggap tulangan 1 lapis)

d pakai = h – d

= 450 – 58 = 392 mm

d pakai > d perlu , pakai tulangan sebelah

$$Rn_{ada} = \frac{Mu/\phi}{b.d^2} = \frac{136{,}133.10^6}{250.392^2} = 3{,}543\ Mpa$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn} = \frac{3{,}543}{3{,}675}.0{,}01015 = 0{,}0098 > \rho_{min} = 0{,}0035$$

$$< \rho_{maks} = 0{,}0203$$

$As_{perlu} = \rho_{ada}.b.d = 0{,}0098.250.392 = 960{,}4\ mm^2$

Dipakai tulangan Ø16 dengan A1Ø = 200,96 mm^2

$$\text{jumlah tulangan (n)} = \frac{As_{perlu}}{A1\phi} = \frac{960{,}4}{200{,}96} = 4{,}779\ \text{batang}$$

dipakai 5D16, maka $As_{ada} = 5 \cdot 200,96 = 1004,8\ mm^2 > As_{perlu}$

Kontrol kapasitas lentur yang terjadi :

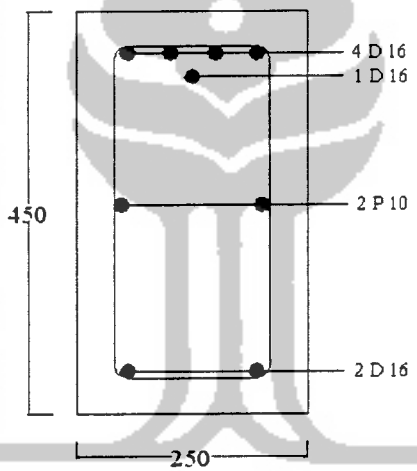
$$a = \frac{As_{ada}.fy}{0,85.fc'.b} = \frac{1004,8.400}{0,85.25.250} = 75,656\ mm$$

$$Mn = As_{ada}.fy.(d - \tfrac{a}{2})$$

$$= 1004,8.400.(392 - \tfrac{75,656}{2})$$

$$= 142,349\ KNm > \tfrac{Mu}{\phi} = 136,113\ KNm \rightarrow \text{OK!}$$

Jumlah tulangan 1lapis (n) $= \frac{b - 2.pb}{(\phi tul + 25)} = \frac{250 - 2.40}{(16 + 25)} = 4,15 \approx 4$



Gambar 4.22 tulangan pokok balok tumpuan kiri

2. Tulangan Lapangan Mu = 61,67 KNm

Mu = 61,67 KNm KNm

$$\frac{Mu}{\phi} = \frac{61,67}{0,8} = 77,0875\ KNm$$

$$b \cdot d^2 = \frac{Mu/\phi}{Rn}$$

dengan b = 250 mm, sehingga

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{77{,}0875.10^6}{3{,}675.250}} = 289{,}663 \text{ mm}$$

nilai h = 450 mm

d_{pakai} = Pb + D sengkang + 0,5 . D tulangan

= 40 + 10 + 0,5 . 16 = 58 (Anggap tulangan satu lapis, d'=50-70 mm)

d pakai = h – d' = 450 – 58 = 392 mm

d pakai > d perlu ,dipakai tulagan sebelah

$$Rn_{ada} = \frac{Mu/\phi}{b.d^2} = \frac{77{,}0875.10^6}{250.392^2} = 2{,}007 \text{ Mpa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn} = \frac{2{,}007}{3{,}675}.0{,}01015 = 0{,}0056 > \rho_{min} = 0{,}0035$$

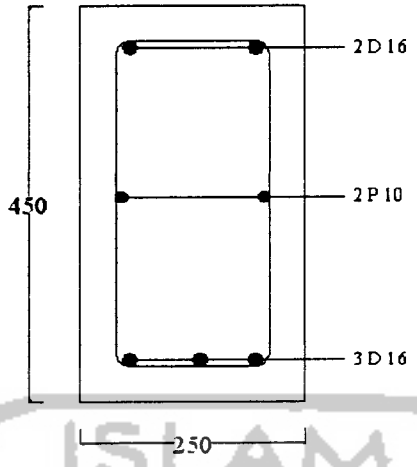
$$< \rho_{maks} = 0{,}0203$$

$As_{perlu} = \rho_{ada}.b.d = 0{,}0056.350.600 = 548{,}8 \text{ mm}^2$

Dipakai tulangan Ø16 dengan A1Ø = 200,96 mm^2

$$\text{jumlah tulangan (n)} = \frac{As_{perlu}}{A1\phi} = \frac{548{,}8}{200{,}96} = 2{,}731 \text{ batang}$$

dipakai 3D16, maka $As_{ada} = 3 \cdot 200{,}96 = 602{,}88 \text{ mm}^2 > As_{perlu}$



Gambar 4.23 tulangan pokok balok lapangan

Kontrol kapasitas momen nominal :

$$a = \frac{As_{ada}.fy}{0,85.fc'.b} = \frac{602,88.400}{0,85.25.350} = 45,393 \text{ mm}$$

$$Mn = As_{ada}.fy.(d - {}^{a}/_{2})$$

$$= 602,88.400.(392 - {}^{45,393}/_{2})$$

$$= 89,085 \text{ KNm} > {}^{Mu}/_{\phi} = 77,0875 \text{ KNm} \rightarrow \textbf{OK!}$$

D. Tulangan Tumpuan Kanan Mu = 97,8 KNm

Mu = 97,8 KNm

$$\frac{Mu}{\phi} = \frac{97,8}{0,8} = 122,25 \text{ KNm}$$

$$b \cdot d^2 = \frac{{}^{Mu}/_{\phi}}{Rn}$$

dengan b = 250 mm, sehingga

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{122,25.10^6}{3,675.250}} = 364,776 \text{ mm}$$

nilai h = 450 mm

d_{pakai} = Pb + D sengkang + 0,5 . D tulangan

= 40 + 10 + 0,5 . 16 = 58 (Anggap tulangan satu lapis, d'=50-70 mm)

d pakai = h – d' = 450 – 58 = 392 mm

d pakai > d perlu ,dipakai tulangan sebelah

$$Rn_{ada} = \frac{Mu/\phi}{b.d^2} = \frac{122,25.10^6}{250.392^2} = 3,182 \text{ Mpa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn} = \frac{3,182}{3,675}.0,01015 = 0,0088 > \rho_{min} = 0,0035$$

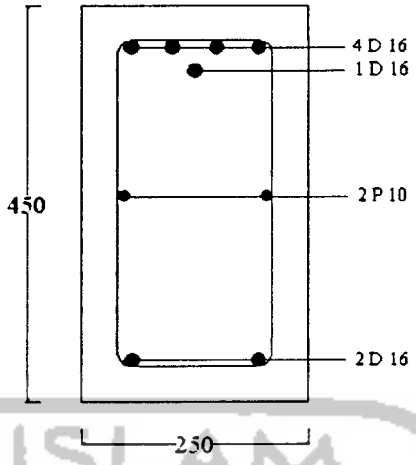
$$< \rho_{maks} = 0,0203$$

$As_{perlu} = \rho_{ada}.b.d = 0,0088.250.392 = 862,4 \text{ mm}^2$

Dipakai tulangan Ø16 dengan A1Ø = 200,96 mm^2

$$\text{jumlah tulangan (n)} = \frac{As_{perlu}}{A1\phi} = \frac{862,4}{200,96} = 4,291 \text{ batang}$$

dipakai 5D16, maka $As_{ada} = 5 . 200,96 = 1004,8 \text{ mm}^2 > As_{perlu}$



Gambar 4.24 tulangan pokok balok tumpuan kanan

Kontrol kapasitas momen nominal :

$$a = \frac{As_{ada}.fy}{0,85.fc'.b} = \frac{1004,8.400}{0,85.25.350} = 75,656 \text{ mm}$$

$$Mn = As_{ada}.fy.(d - {}^{a}/_{2})$$

$$= 1004,8.400.(392 - {}^{75,656}/_{2})$$

$$= 142,349 \text{ KNm} > {}^{Mu}/_{\phi} = 122,25 \text{ KNm} \rightarrow \textbf{OK!}$$

4.4.2.2 Disain Tulangan Geser Balok Anak

- Gaya geser dukung :

  Vu dukungan = 85,48 KN

  Maka ${}^{Vu}/_{\phi}$ = ${}^{85,48}/_{0,6}$ = 142,467 KN

- Gaya geser pada penampang kritis sejauh d dari tumpuan :

4.4.3 Disain Tulangan Balok Pengikat

- L = 6250 mm
- P1 = 1640.720 KN
- P2 = 1159.520 KN

$Pu_{pakai} = 0,1 . \left(\frac{1640,720 + 1159,520}{2}\right) = 140,012$ KN

$A_{tul\ tarik} = \frac{140,012.1000}{0,33.400} = 1060,697\ mm^2$

Dipakai tul D22, $A_1\phi = \frac{1}{4} . \pi . (22)^2 = 379,94 mm^2$

Jumlah tulangan $= \frac{1060,697}{379,94} = 2,792 \approx 4$ buah

As' ada = 4 . 379,94 = 1519,76 mm^2 > $A_{tul\ tarik}$ = 1060,697 mm^2

Pn = 0,8 . [0,85.f'c . (Ag – Ast) + Ast . fy]

2524,185.1000= 0,8 . [0,85 . 25 . 350 . 600 + 400 . Ast]

Ast = - 5127,5 mm^2

ρ_{min} = 1,4/fy = 1,4/400 = 0,0035

As min = 0,0035 . 350 . 600 = 735 mm^2

Dipakai tul D22, $A_1\phi = \frac{1}{4} . \pi . (22)^2 = 379,94 mm^2$

Jumlah tulangan $= \frac{735}{379,94} = 1,935 \approx$ 4 buah

As ada = 4 . 379,94 = 1519,76 mm^2 > As min = 735 mm^2

$$Vu/\phi \text{ d} = Vu/\phi Lap + \left(\frac{1/2\,L - d}{1/2.L}\right)\left(Vu/\phi tump - Vu/\phi Lap\right)$$

$$= 2{,}11/0{,}6 + \left(\frac{1/2.9{,}25 - 0{,}392}{1/2.9{,}25}\right)\left(85{,}48/0{,}6 - 2{,}11/0{,}6\right)$$

$$= 130{,}656 \text{ KN}$$

- Gaya geser beton (Vc)

$$Vc = 1/6.\sqrt{f'c}.b.d = 1/6.\sqrt{25}.250.392 = 81{,}667 KN$$

$0{,}5\ Vc = 0{,}5\ .\ 81{,}667 = 40{,}834$ KN

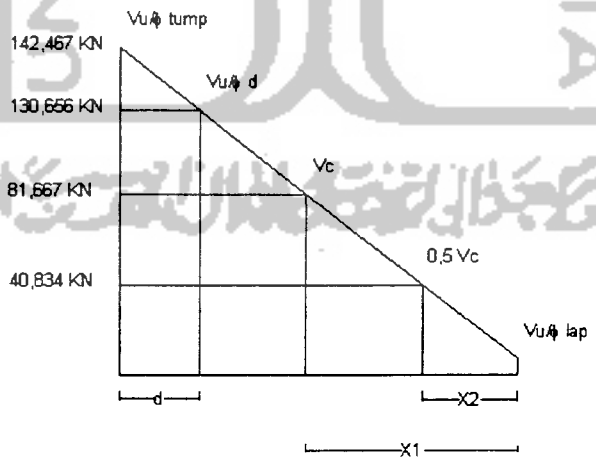
$3\ .\ Vc = 3\ .\ 81{,}667 = 245{,}001$ KN

$Vs\ min = 1/3\ b.d = 1/3.250.392 = 32{,}667$ KN

$Vc + Vs\ min = 114{,}334$ KN

$(Vc + Vs\ min) < Vu/\phi < 3\ Vc$

114,334 KN < 130,656 KN < 245,001 KN , maka diperlukan tulangan geser



Gambar 4.25 Diagram geser balok anak

Titik dimana gaya geser Vc = 81,667 KN

$X_1 = \frac{81,667 - 3,517}{142,467 - 3,517}.4,625 = 2,601$ m dari tengah bentang

Titik dimana gaya geser 0,5 Vc = 40,834 KN

$X_1 = \frac{40,834 - 3,517}{142,467 - 3,517}.4,625 = 1,242$ m dari tengah bentang

- ***Daerah I***

Dipakai sengkang P8, maka Av = 2 . ¼ . π . 8^2 = 100,48 mm^2

Vu/ϕ -Vc = 142,467 – 81,834 = 60,8 KN

Jarak sengkang, s $\leq \frac{Av.fy.d}{Vu/\phi - Vc} = \frac{100,48.240.392}{60,8} = 155,480mm$

$\leq$ d/2 = 392/2 =196 mm

$\leq$ 600 mm

Jadi dipakai sengkang P8 – 150 mm

- ***Daerah II***

Dipakai sengkang P8, maka Av = 2 . ¼ . π . 8^2 = 100,48 mm^2

Jarak sengkang, s $\leq \frac{Av.fy.d}{Vu/\phi - Vc} = \frac{100,48.240.392}{60,8} = 155,480mm$

$\leq$ d/2 = 392/2 =196 mm

$\leq$ 600 mm

Jadi dipakai sengkang P8 – 150 mm

4.5 Disain Kolom

4.5.1 Analisis Gaya Aksial dan Momen akibat balok

Perhitungan kolom K1-8 = K1-9 lantai 1

h = 4,36 m

hn = 3,66 m

Rv = 1 (jumlah lantai ; $1 < n \leq 4$)

ωd = 1,3

k = 1

a. Perhitungan Arah X

- Atas

$Mkap_{(kiri)}$ = 1,25. Mnak = 1,25. 599,77 = 749,7125 KNm

$Mkap_{(kanan)}$ = 1,25. Mnak = 1,25. 738,71 = 932,3875 KNm

menghitung gaya aksial rencana :

$$Pu,k_y = 0,7.\ Rv.\ \frac{Mkap_{kiri} + Mkap_{kanan}}{l} + 1,05.\ Ng$$

$$= 0,7.\ 1.\ \frac{749,7125}{4,5625} + \frac{932,3875}{8,750} + 1,05(1211,26 + 267,15)$$

$$= 1741,946\ KN$$

tidak perlu melebihi :

$$Pu,k_y = 1,05\ (N_D + N_L + 4.N_{Ey}.\ 0,3\ N_{Ex})$$

$$= 1,05\ (1211,26 + 276,150 + 4.12,25\ .\ 0,3\ .\ 103,87)$$

$$= 3165,014\ KN$$

- Bawah

$Mkap_{(kiri)}$ = 1,25. Mnak = 1,25. 352,50 = 440,625 KNm

$Mkap_{(kanan)}$= 1,25. Mnak = 1,25. 352,50 = 440,625 KNm

menghitung gaya aksial rencana :

$$Pu,k_y = 0,7.\ Rv.\ \frac{Mkap_{kiri} + Mkap_{kanan}}{l} + 1,05.\ Ng$$

$$= 0,7.\ 1.\ \frac{440,625}{4,5625} + \frac{440,625}{8,750} + 1,05(1221,63 + 276,15)$$

$$= 1666,072\ KN$$

tidak perlu melebihi :

$$Pu,k_y = 1,05\ (N_D + N_L + 4.N_{Ey}.\ 0,3\ N_{Ex})$$

$$= 1,05\ (1211,26 + 276,150 + 4.12,25\ .\ 0,3\ .\ 103,87)$$

$$= 3165,014\ KN$$

menghitung α :

$M_{E,K\ atas}$ = 132,45 KNm

$M_{E,K\ bawah}$ = 183,63 KNm

$$\alpha\ ka = \frac{M_{E,k(lti+1atas)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = \frac{132,45}{132,45 + 183,63} = 0,419$$

$$\alpha\ kb = \frac{M_{E,k(ltibawah)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = \frac{183,63}{183,63 + 132,45} = 0,581$$

menghitung momen rancang kolom :

$$Mu,k_y\ atas = \frac{hn}{h}\ \omega d.\alpha.0,7.\left(\frac{l_{ki}}{l'_{ki}} Mkap,ki + \frac{l_{ka}}{l'_{ka}} Mkap,ka\right)$$

$$= \frac{3,66}{4,36}.1,3\ .\ 0,419.0,7.\left(\frac{4,5625}{3,9625}.749,7125+\frac{8,75}{8,15}.932,3875\right)$$

$$= 596,702\ \text{KNm}$$

$$\text{Mu,k}_y\ \text{bawah} = \frac{hn}{h}\omega d.\alpha.0,7.\left(\frac{l_{ki}}{l'_{ki}}Mkap,ki+\frac{l_{ka}}{l'_{ka}}Mkap,ka\right)$$

$$= \frac{3,66}{4,36}.1,3.0,581\ 0,7.\left(\frac{4,5625}{3,9625}.440,625+\frac{8,75}{8,15}.440,625\right)$$

$$= 435,130\ \text{kNm}$$

tidak perlu melebihi :

$$\text{Mu,k} = 1,05(M_{Dy} + M_{Ly} + \frac{4}{k}\ M_{Ey})$$

$$= 1,05\ (67,2 + 23,81\ + \frac{4}{1}(132,45\ ).0,3\ .\ 120,31)$$

$$= 20173,735\ \text{KNm}$$

b. .Perhitungan Arah Y

- Atas

$\text{Mkap}_{(\text{kiri})}$ = 1,25. Mnak = 1,25. 677,01 = 846,263 KNm

$\text{Mkap}_{(\text{kanan})}$= 1,25. Mnak = 1,25. 258,66 = 323,325 KNm

menghitung gaya aksial rencana :

$$\text{Pu,k}_x = 0,7.\ \text{Rv.}\ \frac{\text{Mkap}_{\text{kiri}} + \text{Mkap}_{\text{kanan}}}{l} + 1,05.\ \text{Ng}$$

$$= 0,7.\ 1.\ \frac{323,325}{6,250} + \frac{846,263}{2,75} + 1,05(1211,26 + 267,15\ )$$

$$= 1803.927\ \text{KN}$$

tidak perlu melebihi :

Pu,k_x $= 1,05\ (N_D + N_L + 4.N_{Ex}.\ 0,3\ N_{Ey})$

$= 1,05\ (1211,26 + 276,150 + 4.\ 103,87\ .\ 0,3\ .\ 12,25)$

$= 3165,014$ KN

- Bawah

$Mkap_{(kiri)}$ $= 1,25.\ Mnak = 1,25.\ 352,50 =$ 440,625 KNm

$Mkap_{(kanan)} = 1,25.\ Mnak = 1,25.\ 352,50 =$ 440,625 KNm

menghitung gaya aksial rencana :

$$Pu,k_x = 0,7.\ Rv.\ \frac{Mkap_{kiri} + Mkap_{kanan}}{l} + 1,05.\ Ng$$

$$= 0,7.\ 1.\ \frac{440,625}{6,25} + \frac{440,625}{2,75} + 1,05(1221,63 + 276,15)$$

$= 1734,178$ KN

tidak perlu melebihi :

Pu,k_x $= 1,05\ (N_D + N_L + 4.N_{Ex}.\ 0,3\ N_{Ey})$

$= 1,05\ (1211,26 + 276,150 + 4.103,87\ .\ 0,3\ .\ 12,25)$

$= 3165,014$ KN

menghitung α :

$M_{E,K\ atas}$ $= 120,310$ KNm

$M_{E,K\ bawah}$ $= 146,540$ KNm

$$\alpha\ ka = \frac{M_{E,k(lti+1atas)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = \frac{120,310}{120,310 + 146,540} = 0,451$$

$$\alpha\ kb = \frac{M_{E,k(ltibawah)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = \frac{146,540}{146,540 + 120,310} = 0,549$$

menghitung momen rancang kolom :

$$Mu,k_x \text{ atas} = \frac{hn}{h}\omega d.\alpha.0,7.\left(\frac{l_{ki}}{l'_{ki}}Mkap,ki + \frac{l_{ka}}{l'_{ka}}Mkap,ka\right)$$

$$= \frac{3,66}{4,36}.1,3\ .\ 0,451.0,7.\left(\frac{6,25}{5,65}.846,263 + \frac{2,75}{2,15}.323,325\right)$$

$$= 464,992 \text{ KNm}$$

$$Mu,k_x \text{ bawah} = \frac{hn}{h}\omega d.\alpha.0,7.\left(\frac{l_{ki}}{l'_{ki}}Mkap,ki + \frac{l_{ka}}{l'_{ka}}Mkap,ka\right)$$

$$= \frac{3,66}{4,36}.1,3.0,549\ 0,7.\left(\frac{6,25}{5,65}.440,625 + \frac{2,75}{2,15}.440,625\right)$$

$$= 443,826 \text{ KNm}$$

tidak perlu melebihi :

$$Mu,k = 1,05(M_{Dx} + M_{Lx} + \frac{4}{k} M_{Ex})$$

$$= 1,05\ (67,2 + 23,81\ + \frac{4}{1}(132,45\ ).0,3\ .\ 120,31)$$

$$= 20173,735 \text{ KNm}$$

4.5.2 Perencanaan Tulangan Lentur Kolom

Untuk perencanaan penulangan kolom dipakai nilai terbesar dari hasil analisis SAP200 dan momen akibat momen kapasitas balok, maka :

Pu,k_x = 1741,946 KN

Pu,k_y = 1803,927 KN

Mu,k_x = 596,702 KNm

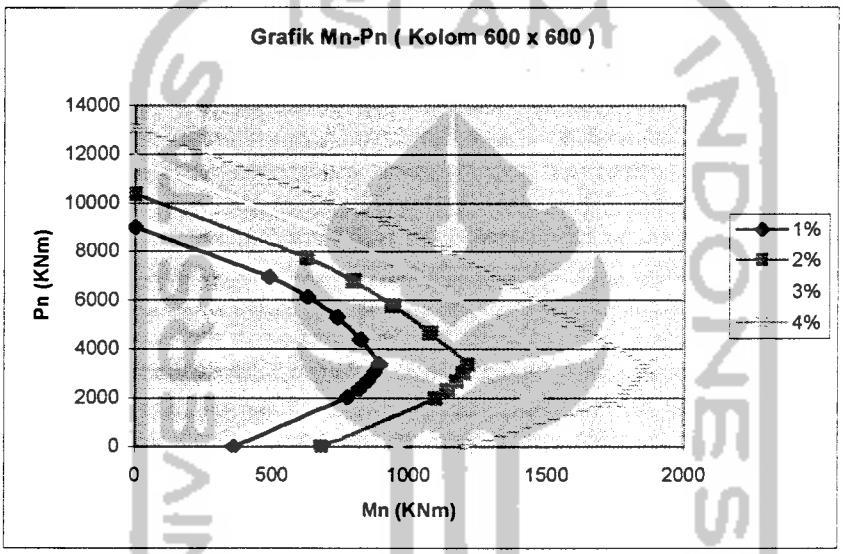
Mu,k_y = 464,992 KNm

$$\frac{Pu,k_x}{\phi} = \frac{1741{,}946}{0{,}65} = 2678{,}378 \text{ KN}$$

$$\frac{Pu,k_y}{\phi} = \frac{1803{,}927}{0{,}65} = 2775{,}272 \text{ KN}$$

$$\frac{Mu,k_x}{\phi} = \frac{596{,}702}{0{,}65} = 918{,}003 \text{ KNm}$$

$$\frac{Mu,k_y}{\phi} = \frac{464{,}992}{0{,}65} = 715{,}372 \text{ KNm}$$



Gambar 4.26 Grafik Mn - Pn kolom

a. Arah x

$$\frac{Pu,k_x}{\phi} = \frac{1741{,}946}{0{,}65} = 2678{,}378 \text{ KN}$$

$$\frac{Mu,k_x}{\phi} = \frac{596{,}702}{0{,}65} = 918{,}003 \text{ KNm}$$

Dari grafik Mn vs Pn didapat $\rho_g = 1{,}45\ \%$

Ast = 0,0145.600.600 = 5220 mm^2

$As = As' = 0{,}5.Ast = 2610\ mm^2$

dipakai 7D22 dengan $As_{ada} = As'_{ada} = 2659{,}58\ mm^2$

Cek eksentrisitas balance (e_b)

$$Xb = \frac{600.d}{600+fy} = \frac{600.530}{600+400} = 318\ mm$$

$ab = \beta_1 . Xb = 0{,}85 . 318 = 270{,}3\ mm$

$$f's = 600\frac{(Xb-d')}{Xb} = 600\frac{(318-70)}{318} = 468\ MPa > fy = 400\ MPa$$

Dengan demikian digunakan $f's = fy = 400\ MPa$

$Ccb = 0{,}85.f'c.b.ab = 0{,}85.25.600.270{,}3 = 3446325\ N$

$Csb = As'(fs' - 0{,}85.f'c) = 2659{,}58.(400 - 0{,}85.25) = 1007315\ N$

$Tsb = As.fy = 2659{,}58.400 = 1063832\ N$

$Pnb = Ccb + Csb - Tsb = 3446325 + 1007315 - 1063832$

$= 3389808\ N = 3389{,}808\ KN$

$$Mnb = Ccb\left[\frac{h}{2} - \frac{ab}{2}\right] + Csb\left(\frac{h}{2} - d'\right) + Tsb\left(d - \frac{h}{2}\right)$$

$$= 3446325\left[\frac{600}{2} - \frac{270{,}3}{2}\right] + 1007315\left(\frac{600}{2} - 70\right)$$

$$+ 1063832\left(530 - \frac{600}{2}\right)$$

$= 1045\ KNm$

$$eb = \frac{Mnb}{Pnb} = \frac{1045}{3389{,}808} = 0{,}308\ m$$

$$e = \frac{Mu_k, y/\phi}{Pu_k/\phi} = \frac{918{,}003}{2678{,}378} = 0{,}343\ m$$

$$e' = e + \frac{d-d'}{2} = 343 + \frac{530-70}{2} = 573 \text{ mm.}$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.25} = 18{,}82$$

$$\rho = \frac{As}{b.d} = \frac{2610}{600.530} = 0{,}00821$$

karena $e > e_b$ ⟶ kolom mengalami patah tarik

Kontrol tegangan pada daerah tarik :

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

$$Pn = 0{,}85.25.600.530.\left[\left(1-\frac{573}{530}\right)+\sqrt{\left(1-\frac{573}{530}\right)^2+2.18{,}82.0{,}00821.\left(1-\frac{70}{530}\right)}\right]$$

$$=2994{,}074 \text{ KN} > \frac{Pu,k_x}{\phi} = \frac{1741{,}946}{0{,}65} = 2678{,}378 \text{ KN}$$

Mn = Pn . e

= 2994,074 . 0,343

$$= 1026{,}967 \text{ KNm} > \frac{Mu,k_x}{\phi} = \frac{596{,}702}{0{,}65} = 918{,}003 \text{ KNm}$$

b. Arah y

$$\frac{Pu,k_y}{\phi} = \frac{1803{,}927}{0{,}65} = 2775{,}272 \text{ KN}$$

$$\frac{Mu,k_y}{\phi} = \frac{464{,}992}{0{,}65} = 715{,}372 \text{ KNm}$$

Dari grafik Mn vs Pn didapat $\rho_g = 1$ %

Ast = 0,01.600.600 = 3600 mm^2

As = As' = 0,5.Ast = 1800 mm^2

dipakai 5D22 dengan As_{ada} = As'_{ada} = 1899,7 mm^2

Cek eksentrisitas balance (e_b)

$$cb = \frac{600.d}{600 + fy} = \frac{600.530}{600 + 400} = 318 \text{ mm}$$

$ab = \beta_1 . cb = 0,85 . 318 = 270,3$ mm

$$f'sb = 600\frac{(cb - d')}{cb} = 600\frac{(270,3 - 70)}{270,3} = 444,617 \text{ MPa} > fy = 400 \text{ MPa}$$

digunakan f'sb = fy = 400 MPa

Ccb = 0,85.f'c.b.ab = 0,85.25.600.270,3 = 3446325 N

Csb = As'(fs'- 0,85.f'c) = 1800.(400 – 0,85.25) = 681750 N

Tsb = As.fy = 1800.400 = 720000 N

Pnb = Ccb + Csb – Tsb = 3446325 + 681750 – 720000

= 3408075 N

$$Mnb = Ccb\left[\frac{h}{2} - \frac{ab}{2}\right] + Csb\left(\frac{h}{2} - d'\right) + Tsb\left(d - \frac{h}{2}\right)$$

$$= 3446325.\left[\frac{600}{2} - \frac{270,3}{2}\right] + 681750.\left(\frac{600}{2} - 70\right) + 720000\left(530 - \frac{600}{2}\right)$$

= 890,529 KNm

$$eb = \frac{Mnb}{Pnb} = \frac{890,529}{3408,075} = 0,261$$

$$e = \frac{Mu_{,kx}/\phi}{Pu_{,k}/\phi} = \frac{715,372}{2775,272} = 0,258$$

karena $e < e_b$,kolom mengalami patah desak

Kontrol kapasitas kolom terhadap patah desak :

$$Pn = \frac{As'.fy}{\frac{e}{(d-d')}+0,5} + \frac{b.h.fc'}{\frac{3.h.e}{d^2}+1,18}$$

$$= \frac{1800.400}{\frac{258}{(530-70)}+0,5} + \frac{600.600.25}{\frac{3.600.258}{530^2}+1,18}$$

$$= 678694,644 + 3176564,017 = 3855258,661 \text{ N}$$

$$Pn = 3855,258 \text{ KN} > \frac{Pu,_k}{\phi} = 2775,272 \text{ KN} \text{ ..... Ok !}$$

$$Mn = Pn \,.\, e$$

$$= 3855,258 \,.\, 0,258$$

$$= 994,657 \text{ KNm} > \frac{Mu,k_x}{\phi} = 715,372 \text{ KNm}$$

4.5.3 Perencanaan Tulangan Geser Kolom

$Mu,_k$ atas = 596,702 KNm

$Mu,_k$ bwh = 218,782 KNm

$V_{D,}k$ = 19,99 KN

$V_{L,}k$ = 6,740 KN

$V_{E,}k$ = 52,74 KN

h_n = 3,66 m

$$V_{u,k} = \frac{Mu,k_{atas} + Mu,k_{bawah}}{h_n} = \frac{596,702 + 218,782}{3,66} = 222,810 \text{ KN}$$

tetapi tidak perlu lebih besar dari :

$$V_{u,k} = 1,05\,(V_{D,k} + V_{L,k} + \frac{4}{k}(V_{E,k})$$

$$= 1{,}05\ (19{,}99 + 6{,}740 + \frac{4}{1}.(52{,}74))$$

$$= 249{,}575\ \text{KN}$$

$$\frac{V_U, k}{\phi} = \frac{222{,}810}{0{,}6} = 371{,}350\ \text{KN}$$

di daerah sejauh *lo*

kekuatan beton dalam menahan gaya geser dianggap 0 (Vc = 0)

$$\text{Vs} = \frac{V_U, k}{\phi} = 371{,}350\ \text{KN}$$

Dipakai tulangan geser P10 mm, maka :

$$\text{Av} = 2\ .\ ¼\ .\pi\ .\ 10^2 = 157\ \text{mm}^2$$

$$\text{Jarak (s)} < \frac{Av.fy.d}{Vs} = \frac{157.240.530}{371{,}350.10^3} = 53{,}778\ \text{mm}$$

$$< \text{d/4} = 132{,}5\ \text{mm}$$

$$< 16.\text{D} = 160\ \text{mm}$$

Digunakan sengkang $\mathbf{P_{10\text{-}50}}$ **mm**

di luar daerah *lo*

$$\text{Vc} = \left(1 + \frac{P_U.k}{14.A_g}\right).\frac{1}{6}\sqrt{fc'}.b.d = \left(1 + \frac{1880{,}95.10^3}{14.600.600}\right).\frac{1}{6}\sqrt{25}.600.530$$

$$= 363{,}900\ \text{KN} < \frac{V_U, k}{\phi} = 371{,}350\ \text{KN}, \text{ maka perlu tulangan geser.}$$

$$\text{Vs} = \frac{V_U, k}{\phi} - \text{Vc} = 371{,}350 - 363{,}900 = 7{,}45\ \text{KN}$$

Dipakai tulangan geser P10 mm, maka :

$$\text{Av} = 2\ .\ ¼\ .\pi\ .\ 10^2 = 157\ \text{mm}^2$$

Jarak (s) $< \frac{Av.fy.d}{Vs} = \frac{157.240.530}{7,45.10^3} = 2680,59$ mm

$< d/2 = 265$ mm

$< 16.D = 160$ mm

Digunakan sengkang **$P_{10\text{-}160}$ mm**

4.5.4 Perencanaan Tulangan Lentur Kolom Dengan Biaksial Momen

Perencanaan kolom biaksial momen ini hanya dijadikan cek. Sedangkan perencanaan sebenarnya pada kolom adalah dengan menghitung arah x dan arah y sebagaimana telah dijelaskan di muka. Adapun perhitungan kolom dengan cara biaksial momen adalah sebagai berikut :

Pu = 1803.927 KN

Mu_x = 596.702 KNm

Mu_y = 464.992 KNm

$Pn = \frac{Pu}{\phi} = \frac{1803,927}{0,7} = 2577.039$ KN

$Mn_x = \frac{Pu}{\phi} = \frac{596,702}{0,7} = 852.431$ KNm

$Mn_y = \frac{Pu}{\phi} = \frac{464,992}{0,7} = 664.274$ KNm

$\frac{Mnx}{Mny} = \frac{852,431}{664,274} = 1.283$

Gunakan Mox Untuk perencanaan

$Mox = Mnx + Mny \frac{h}{b} . \frac{1-\beta}{\beta}$

Digunakan $\beta = 0.65$ untuk perencanaan

b = 600 mm. h = 600 mm. d = 530 mm. d' = 70 mm

$$Mox = 852.431 + 664.274 \frac{0,6}{0,6} \cdot \frac{1-0,65}{0,65} = 1210.117 \text{ KNm}$$

Kontrol kapasitas Pn pada penampang yang diasumsikan

Dianggap $\rho = \rho' = 0.012$

$As = As' = \rho.b.d = 0.012.600.530 = 3816 \text{ mm}^2$

Pakai 16D25 dengan As ada = 7850 mm^2

Cek eksentrisitas balance (e_b)

$$Xb = \frac{600.d}{600+fy} = \frac{600.530}{600+400} = 318 \text{ mm}$$

$ab = \beta_1 . Xb = 0.85 . 318 = 270.3$ mm

$$f's = 600 . \frac{(Xb-d')}{Xb} = 600\frac{(318-70)}{318} = 467,925Mpa > fy = 400 \text{ Mpa}$$

Dengan demikian digunakan f's = 400 Mpa

Ccb = 0.85 . f'c . ab = 0.85 . 25 . 600 . 270.3 = 3446325 N

Csb = As' . (fs' – 0.85 . f'c) = 7850.(400 - 0.85.25) = 2973187.5 N

Tsb = As . fy = 7850 . 400 = 3140000 N

Pnb = Ccb + Csb – Tsb

= 3446325 + 2973187.5 – 3140000 = 3279.513 KN

Karena Pnb > Pnmaka patah yang terjadi adalah pata desak

$$\rho = \frac{As}{b.d} = \frac{7850}{600.530} = 0.025$$

$$e = \frac{Mox}{Pn} = \frac{1210,117}{2577,039} = 0.470 \text{ m} = 470 \text{ mm}$$

$$\frac{h-2e}{2d} = \frac{600-2.470}{2.530} = -0.321$$

$$1 - \frac{d'}{d} = 1 - \frac{70}{530} = 0.868$$

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.25} = 18,824$$

Kontrol tegangan pada daerah tarik :

$$Pn = 0.85.f'c.b.d.\left[\frac{h-2e}{2d} + \sqrt{\left(\frac{h-2e}{2d}\right)^2 + 2m\rho\left(1-\frac{d'}{d}\right)}\right]$$

$$= 0.85.25.600.530.\left[-0,321 + \sqrt{(-0,321)^2 + 2.18,824.0,025.0,868}\right]$$

$$= 4.313 \,.\, 10^6 \text{ N} > 2.578 \,.\, 10^6 \text{ N}$$

$$Pr = \phi.Pn = 0.7 \,.\, 4.313.10^6 \text{ N} = 3018.693 \text{ KN}$$

Pr > 0.1 . Ag . f'c = 0.1 . 600 . 600 . 25 = 900 KN. Maka tetap pakai 0.7

Cek apakah benar tegangan pada tulangan desak fs'>fy

$$a = \frac{Pn}{0,85.f'c.b} = \frac{4313000}{0,85.25.600} = 338,275mm$$

$$c = \frac{a}{0,85} = \frac{338,275}{0,85} = 397,970mm$$

$$fs' = 600.\frac{c-d'}{c} = 600.\frac{397,970-70}{397,970} = 494,464Mpa > fy = 400 \text{ Mpa}$$

menghitung momen tahanan nominal aktual Moxn Untuk lentur uniaksial ekuivalen terhadap sumbu x bila Moy = 0

$$a = \frac{Pn}{0,85.f'c.b} = \frac{2577039}{0,85.25.600} = 202,121 \text{ mm}$$

$$c = \frac{a}{\beta_1} = \frac{202{,}121}{0{,}85} = 237{,}789mm$$

$$f's = 600.\frac{c-d'}{c} = 600.\frac{237{,}789-70}{237{,}789} = 423{,}373Mpa$$

fs' > fy ; maka dipakai fs'= fy = 400 Mpa

Moxn = Pn . e

$$= 0.85.f'c.ab.\left(\frac{h}{2}-\frac{a}{2}\right) + As.fs'\left(\frac{h}{2}-d'\right) + As.fy\left(d-\frac{h}{2}\right)$$

$$= 0.85.25.202.121.600.\left(\frac{600}{2}-\frac{202{,}121}{2}\right) + 7850.400.\left(\frac{600}{2}-70\right)$$

$$+ \ 7850.400.\left(530-\frac{600}{2}\right)$$

= 1957.076 KNm > Mox = 1210.117 KNm

Menghitung momen tahanan aktual Moyn Untuk momen lentur uniaksial ekivalen terhadap sumbu y dimana Mox = 0

Dengan coba – coba dan penyesuaian. Menentukan tinggi blok tegangan a atau tinggi garis netral sedemikian rupa sehingga Pn yang dihitung mendekati Pn yang diperlukan.

Dicoba : a = 180 dan c = 180 / 0,85 = 211,765 mm

$$fs' = 600.\frac{c-d'}{c} = 600.\frac{211{,}765-70}{211{,}765} = 401{,}666 \text{ Mpa} > fy$$

$$fs = 600.\frac{d-c}{d} = 600.\frac{530-211{,}765}{530} = 360{,}266 \text{ Mpa} > fy$$

digunakan fs' = 400 Mpa dan fs = 360.266 Mpa

Pn = 0,85 . f'c . ab + As' . fs' – As . fs

$= 0.85 \cdot 25 \cdot 180 \cdot 600 + 7850 \cdot 400 - 7850 \cdot 360{,}266$

$= 2606{,}912$ KN

Dengan demikian dipakai a = 180 mm Untuk menghitung Moyn

$$\text{Moyn} = 0{,}85 \cdot f'c \cdot ab \cdot \left(\frac{h}{2} - \frac{a}{2}\right) + As.f's.\left(\frac{h}{2} - d'\right) + As.fy\left(d - \frac{h}{2}\right)$$

$= 0{,}85 \cdot 25 \cdot 180.600 \cdot$

$$\left(\frac{600}{2} - \frac{180}{2}\right) + 7850.400.\left(\frac{600}{2} - 70\right) + 7850.400\left(530 - \frac{600}{2}\right)$$

$= 1926{,}35$ KNm

Kemudian Untuk mencari Mny, diperlukan data Mnx/Moxn dan factor β pada grafik Untuk β = 0,65

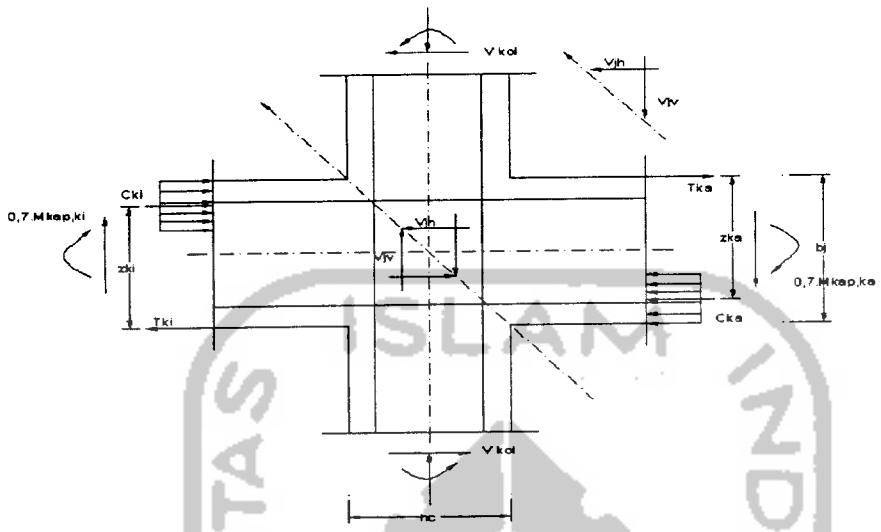
$$\frac{Mnx}{Moxn} = \frac{852{,}431}{1957{,}076} = 0{,}436$$

dari dua data diatas diperoleh

$$\frac{Mny}{Moyn} = 0{,}825$$

Mny $= 0{,}825 \cdot 1926{,}35 = 1589{,}239$ KNm > 664,740 KNm

4.5.5 Pertemuan Balok Kolom



Gambar 4.27 Joint Balok Kolom

a. Perhitungan gaya-gaya dalam

1). Sumbu X

$b_j = bc = 600$ mm

$= bb + 0{,}5.hc = 350 + 0{,}5.600 = 750$ mm

b_j pakai = 600 mm

$h_c = 600$ mm

$$V_{kol,x} = \frac{0{,}7.\phi_o.\left(\sum \frac{l_x}{l_{nx}}.M_{nak.bx} + 0{,}3.\sum \frac{l_y}{l_{ny}}.M_{nak.by}\right)}{\frac{1}{2}.(h_a + h_b)}$$

$$V_{kol,x} = 0,7.1,25.\left[\left(\frac{4,5625}{3,9625}.599,77 + \frac{8,75}{8,15}.738,711\right)\right.$$

$$\left. + 0,3.\left(\frac{6,25}{5,65}.677,01 + \frac{2,75}{2,15}.258,66\right)\right] \Big/ \frac{1}{2}.(4,36 + 3,9)$$

$$= 382,262 \text{ KN}$$

$z_{ki,x} = 0,9.d = 0,9.530 = 477 \text{ mm} = 0,477 \text{ m}$

$z_{ka,x} = 0,9.d = 0,9.530 = 477 \text{ mm} = 0,477 \text{ m}$

$C_{ki,x} = T_{ki,x} = 0,7.\phi_o.(M_{nak,bx\text{-}ki})/z_{ki,x}$

$= 0,7.1,25.(599,770)/0,477 = 1100,207 \text{ KN}$

$C_{ka,x} = T_{ka,x} = 0,7.\phi_o.(M_{nak,bx\text{-}ka})/z_{ka,x}$

$= 0,7.1,25.(738,711)/0,477 = 1355,078 \text{ KN}$

$V_{jh,x} = C_{ki,x} + T_{ka,x} - V_{kol,x} = 1100,207 + 1355,078 - 382,262$

$= 2073,023 \text{ KN}$

Kontrol tegangan geser horizontal :

$$v_{jh,x} = \frac{V_{jh,x}}{b_j.h_c} \leq 1,5\sqrt{f'c}$$

$$v_{jh,x} = \frac{2073,023}{0,6.0,6} = 5758,397 \text{ KN/m}^2$$

$= 5,758 \text{ N/mm}^2 < 1,5.\sqrt{25} = 7,5 \text{ N/mm}^2$ ..... **Ok !**

$$V_{ch,x} = 2/3 \ . \sqrt{\left\{\left(Pu,k \Big/ Ag\right) - 0,1.f'c\right\}}.b_j.h_c$$

$$V_{ch,x} = 2/3. \sqrt{\left\{\left(1741,946.10^3 \Big/ 600.600\right) - 0,1.25\right\}}.600.600$$

$= 329,154 \text{ KN}$

$V_{sh,x} = V_{jh,x} - V_{ch,x}$

$= 2073,023 - 329,154 = 1743,869$ KN

2). Arah Y

$b_j = bc = 600$ mm

$= bb + 0,5.hc = 300 + 0,5.450 = 525$ mm

b_j pakai = 600 mm

$h_c = 600$ mm

$$V_{kol,y} = \frac{0,7.\phi_o.\left(0,3.\sum \frac{l_x}{l_{nx}}.M_{nak,bx} + \sum \frac{l_y}{l_{ny}}.M_{nak,by}\right)}{\frac{1}{2}.(h_a + h_b)}$$

$$V_{kol,y} = 0,7.1,25.\left[0,3.\left(\frac{4,5625}{3,9625}.599,77 + \frac{8,75}{8,15}.738,711\right) + \left(\frac{6,25}{5,65}.677,01 + \frac{2,75}{2,15}.258,66\right)\right] \Big/ \frac{1}{2}.(4,36 + 3,9)$$

$= 323,0621$ KN

$z_{ki,y} = 0,9.d = 0,9.530 = 477$ mm $= 0,477$ m

$z_{ka,y} = 0,9.d = 0,9.530 = 477$ mm $= 0,477$ m

$C_{ki,y} = T_{ki,y} = 0,7.\phi_o.(M_{nak,by\text{-}ki})/z_{ki,y}$

$= 0,7.1,25.(677,01)/0,477 = 1241,895$ KN

$C_{ka,y} = T_{ka,y} = 0,7.\phi_o.(M_{nak,by\text{-}ka})/z_{ka,y}$

$= 0,7.1,25.(258,66)/0,477 = 474,481$ KN

$V_{jh,y} = C_{ki,y} + T_{ka,y} - V_{kol,y}$

$= 1241,895 + 474,481 - 323,0621 = 1393,314$ kN

Kontrol tegangan geser horizontal :

$$v_{jh,y} = \frac{V_{jh,y}}{b_j.h_c} \leq 1{,}5\sqrt{f'c}$$

$$v_{jh,y} = \frac{1393{,}314}{0{,}6.0{,}6} = 3870{,}317 \text{ KN/m}^2$$

$= 3{,}870 \text{ N/mm}^2 < 1{,}5.\sqrt{25} = 7{,}5 \text{ N/mm}^2$ .....**Ok !**

$$V_{ch,y} = 2/3 \cdot \sqrt{\left\{\left(Pu,k/Ag\right) - 0{,}1.f'c\right\}}.b_j.h_c$$

$$V_{ch,y} = 2/3.\sqrt{\left\{\left(1880{,}95.10^3/600.600 - 0{,}1.25\right)\right\}}.600.600$$

$= 396{,}167$ KN

$V_{sh,y} = V_{jh,y} - V_{ch,y}$

$= 1393{,}314 - 396{,}167 = 997{,}147$ KN

b. Penulangan Geser Horisontal

$V_{sh,mak} = V_{sh,x} = 1743{,}869$ KN

$$A_{jh} = \frac{V_{sh,mak}}{fy} = \frac{1743869}{400} = 4359{,}673 \text{ mm}^2$$

Digunakan sengkang 2P10 dengan Av = 314 mm^2

Jumlah lapis sengkang = $\frac{4359{,}673}{314} = 13{,}9$ lapis

Penulangan geser vertikal

$$V_{cv} = \frac{A_{sc'}}{A_{sc}} V_{jh,mak}\left(0{,}6 + Pu,k/Ag.f'c\right)$$

$$V_{cv} = 1.3870{,}317.10^3 \left(0{,}6 + {1880{,}95.10^3}/{600.600.25} \right)$$

$= 3131064{,}951$ N $= 3131{,}065$ KN

$V_{jv} = bj/h_c . V_{jh,mak}$

$= (0{,}6/0{,}6).\ 3870{,}317 = 3870{,}317$ KN

$V_{sv} = V_{jv} - V_{cv} = 3870{,}317 - 3131{,}065 = 739{,}252$ KN

$$A_{jv} = \frac{V_{sv}}{fy} = \frac{739252}{400} = 1848{,}130 \text{ mm}^2$$

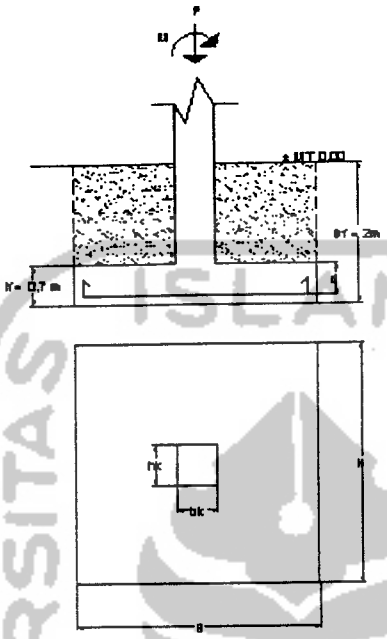
Digunakan sengkang 2P10 dengan Av = 314 mm^2

Jumlah lapis sengkang = $\frac{1848{,}130}{314} = 5{,}886$ lapis

4.6 Perencanaan Pondasi

4.6.1 Perencanaan Pondasi Telapak Setempat (PS1)

4.6.1.1 Perencanaan Dimensi Pondasi



Gambar 4.28 Pondasi telapak setempat

σ tanah	= 325 KN/m^2	γbtanah	= 15,100 KN/m^3
f'c	= 25 Mpa	γbeton	= 24 KN/m^3
fy	= 400 Mpa	Asumsi tebal pelat (tf) = 700 mm	
P tetap	= 3533,400 KN	Ukuran kolom :	
P sementara	= 3014,070 KN	hk = 600 mm	
Mx tetap	= 6,430 KNm	bk = 600 mm	
My tetap	= 28,870 KNm		
Mx sementara	= 1125,580 KNm		

My sementara = 200,720 KNm

$\sigma_{netto\ tanah} = \sigma_{tanah} - \Sigma(h.\gamma_{beton}) - \Sigma(h.\gamma_{tanah})$

$= 325 - (0,7.24) - (1,3.15,100)$

$= 288,570\ kN/m^2$

1. Tinjauan Terhadap Beban Tetap

Dicoba dengan nilai B = H = 3,5 m

$$A = \frac{P}{\sigma netto\ tanah - \left(\frac{Muky}{1/6\,H^2B} + \frac{Mukx}{1/6\,B^2H}\right)}$$

$$= \frac{3533,400}{288,570 - \left(\frac{6.6,430}{3,5^2.3,5} + \frac{6.28,820}{3,5^2.3,5}\right)}$$

$= 12,458\ m^2$

Digunakan penampang bjur sangkar dengan :

$B = H = \sqrt{12,458} = 3,529$ m $B_{ada} = H_{ada} = 3,6$ m

Luas Penampang Pelat Pondasi : $A_{ada} = B \times H = 3,6 \times 3,6 = 12,960\ m^2$

Kontrol Luas Pelat = $A_{ada} = 12,960\ m^2 > 12,458\ m^2$

Tegangan Kontak yang terjadi di dasar pondasi :

$$\sigma_{Kontak} = \frac{Pu}{Aada} + \frac{Mx}{1/6\,H^2.B} + \frac{My}{1/6\,H.B^2}$$

$$\sigma_{Kontak} = \frac{3533,400}{12,960} + \frac{6.6,430}{3,6^2.3,6} + \frac{6.28,870}{3,6^2.3,6}$$

$= 277,178\ KN/m^2 < \sigma_{nettotanah} = 288,570\ KN/m^2$ **.....Ok !**

$$\sigma_{Kontak} = \frac{Pu}{Aada} - \frac{Mx}{\frac{1}{6}H^2.B} - \frac{My}{\frac{1}{6}H.B^2}$$

$$\sigma_{Kontak} = \frac{3533,400}{12,960} - \frac{6.6,430}{3,6^2.3,6} - \frac{6.28,870}{3,6^2.3,6}$$

$= 258,209 \text{ KN/m}^2 > 0$ **.....Ok !**

jarak pusat tulangan tarik ke serat beton :

d = h – Pb – ½ D tul pokok = 700 – 75 – ½.19 = 615,5 mm

2. Tinjauan Terhadap Beban Sementara

P = 3014,07 KN

Mx = 112,580 KN

My = 200,720 KN

Eksentrisitas yang terjadi :

$$ex = \frac{Mx}{P} = \frac{112,580}{3014,070} = 0,037 \text{ m}$$

$$ey = \frac{My}{P} = \frac{200,720}{3014,070} = 0,067 \text{ m}$$

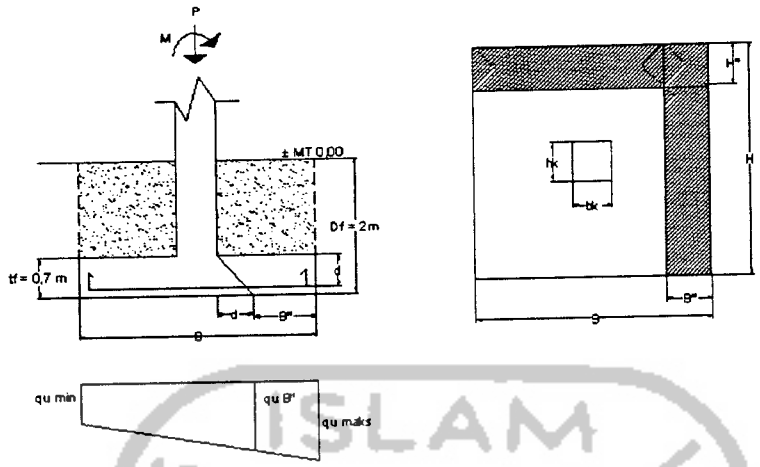
Kontrol tegangan yang terjadi :

$$\sigma_{terjadi} = \frac{P}{(H(B-2ex))+(B(H-2ey))} < 1,5\sigma netto$$

$$= \frac{3533,400}{(3,6(3,6-2.0,037))+(3,6(3,6-2.0,067))} < 1,5\sigma netto$$

$= 140,375 \text{ KN/m}^2 < 1,5.\sigma_{netto} = 1,5.\ 288,570 = 432,855 \text{ KN/m}^2$ **.....Ok !**

4.6.1.2 Perencananaan Geser Satu Arah



Gambar 4.29 Pondasi dengan geser satu arah

→ Ditinjau pada arah momen terbesar.

P = 3533,40 KN

Mx = 6,43 KNm

My = 28,87 kNm

$$H'' = \frac{H - hk - 2.d}{2} = \frac{3,6 - 0,6 - 2.0,6155}{2} = 0,885 \text{ m}$$

$$B'' = \frac{B - bk - 2.d}{2} = \frac{3,6 - 0,6 - 2.0,6155}{2} = 0,885 \text{ m}$$

• Tegangan kontak yang terjadi :

$$qu = \frac{P}{A} \pm \frac{6.Mx}{B^2.H} \pm \frac{6.My}{B.H^2}$$

$$q_A = \frac{P}{A} + \frac{6.Mx}{B^2.H} + \frac{6.My}{B.H^2}$$

$$= \frac{3533,400}{12,96} + \frac{6.6,43}{3,6^2.3,6} + \frac{6.28,87}{3,6.3,6^2}$$

$= 277{,}178 \text{ KN/m}^2$

$$q_B = \frac{P}{A} + \frac{6.Mx}{B^2.H} - \frac{6.My}{B.H^2}$$

$$= \frac{3533{,}400}{12{,}96} + \frac{6.6{,}43}{3{,}6^2.3{,}6} - \frac{6.28{,}87}{3{,}6.3{,}6^2}$$

$= 269{,}753 \text{ KN/m}^2$

$$q_C = \frac{P}{A} - \frac{6.Mx}{B^2.H} + \frac{6.My}{B.H^2}$$

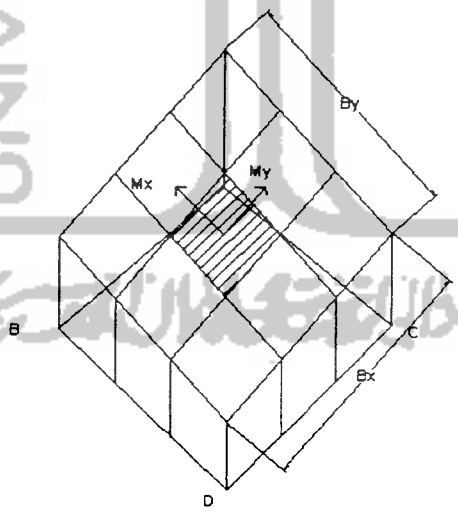
$$= \frac{3533{,}400}{12{,}96} - \frac{6.6{,}43}{3{,}6^2.3{,}6} + \frac{6.28{,}87}{3{,}6.3{,}6^2}$$

$= 275{,}525 \text{ KN/m}^2$

$$q_D = \frac{P}{A} - \frac{6.Mx}{B^2.H} - \frac{6.My}{B.H^2}$$

$$= \frac{3533{,}400}{12{,}96} - \frac{6.6{,}43}{3{,}6^2.3{,}6} - \frac{6.28{,}87}{3{,}6.3{,}6^2}$$

$= 258{,}209 \text{ KN/m}^2$



Gambar 4.30 Diagram tegangan tanah

Arah B

$$q\ max = \frac{277{,}178 + 269{,}753}{2} = 273{,}466 KN/m^2$$

$$qu_B'' = \frac{276{,}772 + 266{,}915}{2} = 271{,}844 KN/m^2$$

qux_{pakai} = 1/2. (qux_{max} + $qu_{B''}$)

= ½. (273,466 + 271,844)

= 272,655 KN/m^2

- Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

Vu = qux_{pakai} .H".B = 272,655 . 0,885 . 3,6 = 868,678 KN

Vu/ϕ = $868{,}678/0{,}6$ = 1447,798 KN

- Kekuatan beton menahan geser:

Vc = 1/6. $\sqrt{f'c}$. Bx. d = 1/6. $\sqrt{25}$ 3,6. 0,6155 . 10^3 = 1846,5 KN

- Kontrol gaya geser :

Vc = 1846,5 KN ≥ Vu/ϕ = 1447,798 KN **.....Ok !**

Arah H

$$q\ max = \frac{277{,}178 + 275{,}525}{2} = 276{,}352 KN/m^2$$

$$qu_H'' = \frac{275{,}353 + 271{,}268}{2} = 273{,}321 KN/m^2$$

qux_{pakai} = 1/2. (qux_{max} + $qu_{B''}$)

= ½. (276,352 + 273,321)

= 274,832 KN/m^2

- Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

Vu = qux_{pakai} .H".B = 274,832 . 0,885 . 3,6 = 875,615 KN

$Vu/\phi = 875,615/0,6 = 1459,358$ KN

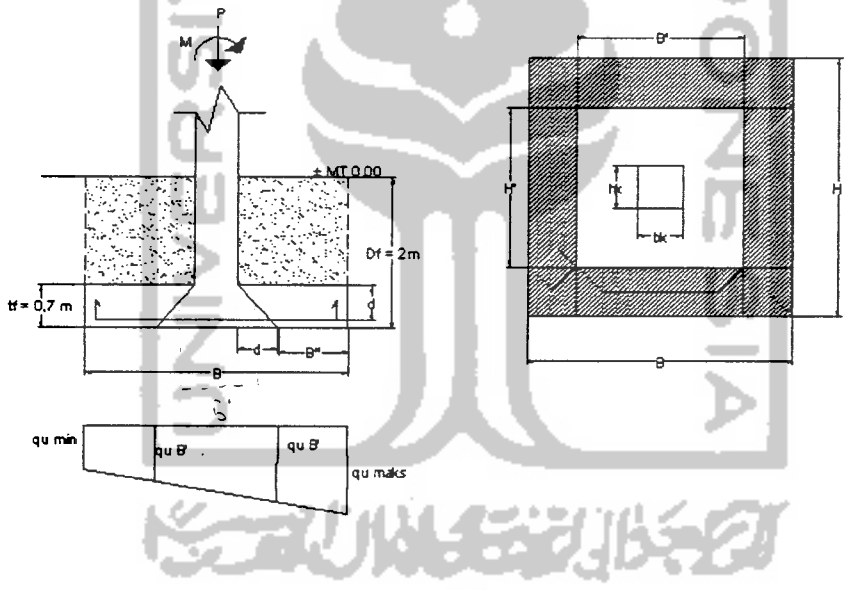
- Kekuatan beton menahan geser:

Vc = 1/6. $\sqrt{f'c}$. Bx. d = 1/6. $\sqrt{25}$ 3,6. 0,6155 . 10^3 = 1846,5 KN

- Kontrol gaya geser :

Vc = 1846,5 KN ≥ Vu/ϕ = 1459,358 KN .....**Ok !**

4.6.1.3 Perencananaan Geser Dua Arah



Gambar 4.31 Pondasi dengan geser dua arah

Ditinjau pada arah momen terbesar :

H' = hk + d

= 600 + 615,5 = 1215,5 mm

B' = bk + d

= 600 + 615,5 = 1215,5 mm

- Tegangan kontak yang terjadi :

$$qu = \frac{P}{Aperlu}$$

$$= \frac{3533,400}{12,96} = 272,639 KN/m^2$$

- Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi

$Vu = qu_{pakai}.((H.B)-(H'.B'))$

$= 272,639.((3,6 . 3,6)-(1,216 . 1,216))$

$= 3130,262$ KN

$Vu/\phi = 3130,262/0,6 = 5217,104$ KN

- Kekuatan beton menahan geser :

$$\beta c = \frac{sisipanjang}{sisipendek} = \frac{bk}{hk} = \frac{3,6}{3,6} = 1$$

$bo = 2 . (H' + B') = 2 . (1,216 + 1,216) = 4,862$ m = 4862 mm

$Vc_1 = (1 + 2/\beta c).(2.\sqrt{f'c}).bo.d$

$= (1 + 2/1,173).(2.\sqrt{25}).4862.615,5.10^{-3} = 89776,8$ KN

$Vc_2 = 4.\sqrt{f'c}.bo.d$

$= 4.\sqrt{25}.4862.615,5.10^{-3} = 59851,22$ KN

- Kontrol gaya geser :

Digunakan nilai yang terkecil dari Vc1 dan Vc2

$Vc = 59851,22$ KN $\geq Vu/\phi = 5217,104$ KN.....**Ok !**

4.6.1.4 Perencanaan Tulangan Lentur Pondasi

Arah B

$$l = \frac{B - bk}{2} = \frac{3{,}6 - 0{,}6}{2} = 1{,}5$$

qu mak= 277,178 KN/m^2

Mu = 0,5 qu mak.l^2 = 0,5 . 277,178 . 1,5^2 = 311,825 KNm

$$Mu/\phi = 311{,}825/0{,}8 = 389{,}782 \text{ KNm}$$

Digunakan tulangan pokok D 19 mm, dengan luas tampang 1 tulangan :

A1 = ¼. π . D^2 = ¼. π . 19^2 = 283,385 mm^2

Tebal pelat pondasi : h = 700 mm, selimut beton (Pb) = 75 mm

d = h – Pb – 0,5 . 19 = 700 – 75 – 0,5 . 19 = 615,5 mm

$$m = \frac{fy}{b.d^2} = \frac{400}{0{,}85.25} = 18{,}824$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm :

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{389{,}782.10^6}{1000.615{,}5^2} = 1{,}029 \text{ MPa}$$

Rasio Tulangan :

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

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25.0{,}85}{400}\left(\frac{600}{600+400}\right) = 0{,}0271$$

$$\rho_{mak} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}027 = 0{,}0203$$

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

$$= \frac{1}{18,824}\left(1-\sqrt{1-\frac{2.18,824.1,029}{400}}\right) = 0,00264 < \rho_{mak} = 0,020$$

$$< \rho_{min} = 0,0035$$

$1,33 \; . \; 0,00264 = 0,0035$

$As_{perlu} = \rho_{perlu} \; . \; b \; . \; d = 0,0035 \; . \; 1000 \; . \; 615,5 = 2161,144 \; mm^2$

$As_{tul\ susut} = 0,002 \; . \; b \; . \; h = 0,002 \; . \; 1000 \; . \; 700 = 1400 \; mm^2$

$As_{perlu} > As_{tul\ susut}$ sehingga $As_{pakai} = 2161,144 \; mm^2$

Jarak antar tulangan :

$$s \leq \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{283,529.1000}{2161,144} = 131,127 \text{ mm}$$

$$s \leq 2.h = 2 \; .500 = 1000 \text{ mm}$$

$$s \leq 250 \text{ mm}$$

→ **Dipakai Tulangan Pokok : D_{19} – 130 mm**

$$As_{ada} = \frac{A_{1\theta}.1000}{s} = \frac{283,529.1000}{130} = 2179,85 \; mm^2$$

• Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0,85.f'c.b} = \frac{2179,850.400}{0,85.25.1000} = 41,032 \text{ mm}$$

$$Mn = As_{ada}.fy.\left(d - {}^{a}/_{2}\right)$$

$$= 2179,850. \; 400 \; (615,5 - {}^{41,032}/_{2})$$

$$= 518,791 \text{ KNm} \geq {}^{Mu}/_{\phi} = 389,782 \text{ KNm} \; \textbf{.....Ok !}$$

4.6.1.5 Perencanaan Tulangan Susut Pondasi

As_{susut} = 0,002. b. h = 0,002. 1000. 700 = 1400 mm^2

- Digunakan tulangan bagi ∅13 mm, maka: $A_{1\varnothing}$ = 132,67 mm^2

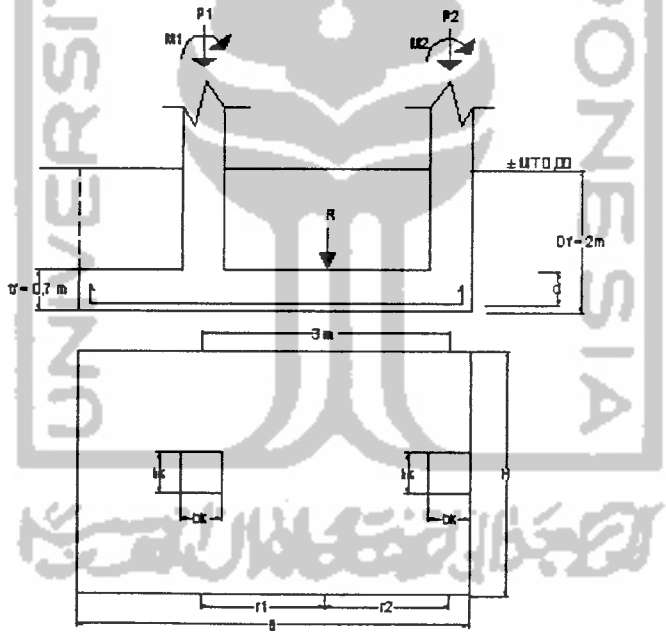
Jarak antar tulangan susut :

$$s \leq \frac{A_{\theta 1}.b}{As_{susut}} = \frac{132{,}670.1000}{1400} = 94{,}76 \text{ mm} \approx 90 \text{ mm}$$

→ **Dipakai Tulangan Susut : P_{13} – 90 mm**

4.6.2 Perencanaan Pondasi Gabungan

4.6.2.1 Perencanaan Dimensi Pondasi



Gambar 4.32 Pondasi telapak gabungan

σ tanah = 325 KN/m^2 γbtanah = 15,100 KN/m^3

f'c = 25 Mpa

fy = 400 Mpa

P_1 = 2298,200 KN

Mx_1 = 17,230 KNm

My_1 = 3,18 KNm

P_2 = 1803,500 KN

Mx_2 = 21,650 KNm

My_2 = 15,740 KNm

γbeton = 24 KN/m^3

Asumsi tebal pelat (tf) = 800 mm

Ukuran kolom :

P1 : 600/600

P2 : 600/600

$\sigma_{netto\ tanah} = \sigma_{tanah} - \Sigma(h.\gamma_{beton}) - \Sigma(h.\gamma_{tanah})$

$= 325 - (0{,}8.24) - (1{,}2.15{,}10)$

$= 287{,}68$ KN/m^2

1. Tinjauan Terhadap Beban Tetap

P total = $P_1 + P_2$ = 2298,200 + 1803,500 = 4101,700 KN

$e1 = \frac{M1}{P1} = \frac{17{,}23}{2298{,}200} = 0{,}01$

$e2 = \frac{M2}{P2} = \frac{21{,}65}{1803{,}500} = 0{,}01$

$e\ tot = \frac{Mtot}{Ptot} = \frac{38{,}88}{4101{,}700} = 0{,}01$

$\Sigma m = 0$

P1 . 3 – M1 – M2 + P tot . r2 = 0

(2298,2 . 3) – (17,23) – (21,65) + (4101,7 . r2) = 0

r2 = 1,671 m

x1 = r2 + 0,3

= 1,671 + 0,3 = 1,971 m

B = 2. x1 = 2 . 1,971 = 3,942 m

Ambil B = 5 m.

Agar P total berada ditengah mk ;

r1 + r2 = 3 m

r1 +1,671 = 3, r1 = 1,329 m

B = 2 . x1

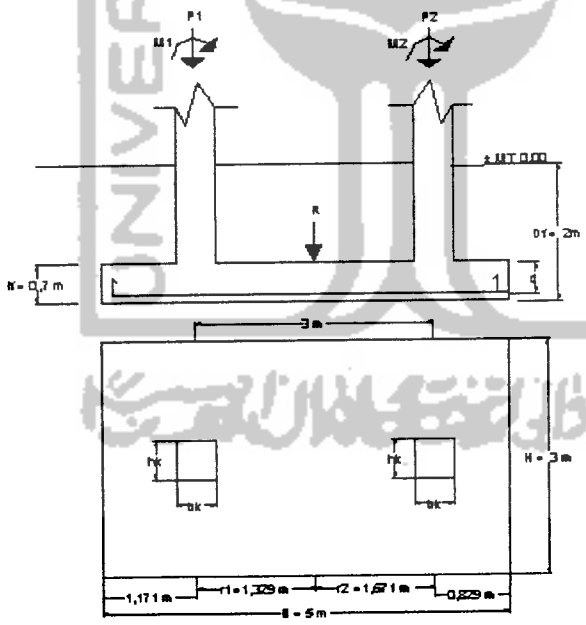
5 =2 . x1, x1 = 5/2 =2,5 m

x1 (kanan) = r2 + Δ

2,5 m = 1,671 + Δ, Δ= 2,5 – 1,671 = 0,829 m

x1 (kiri) = r1 + Δ, Δ= 2,5 – 1,329 = 1,171 m

sehingga pondasi terlihat seperti gambar berikut ini :



Gambar 4.33 Pondasi telapak gabungan

Mx_{tot} = 17,23 + 21,65 = 38,880 KNm

My_{tot} = 3,18 + 15,740 = 18,920 KNm

Dimensi luas pelat pondasi : (terdapat momen yang bekerja pada arah x dan y)

$$\sigma_{netto\ tanah} = \frac{P}{B.H} + \frac{6.My}{H^2.B} + \frac{6.Mx}{B^2.H}$$

dicoba dengan nilai B = 5 m dan H = 3 m

$$\sigma_{kontak\ max} = \frac{4101,7}{5.3} + \frac{6.38,88}{3^2.5} + \frac{6.18,92}{5^2.3}$$

$$= 280,144\ KN/m^2 \leq 287,68\ KN/m^2 \ldots\ldots\ \textbf{Ok !}$$

$$\sigma_{kontak\ max} = \frac{4101,7}{5.3} - \frac{6.38,88}{3^2.5} - \frac{6.18,92}{5^2.3}$$

$$= 266,750\ KN/m^2 \leq 287,68\ KN/m^2 \ldots\ldots\ \textbf{Ok !}$$

jarak pusat tulangan tarik ke serat tekan beton :

d = h – Pb – ½.D tul pokok = 800 – 75 – ½.19 = 715,5 m

2. Tinjauan Terhadap Beban Sementara

Mx_{tot} = 17,23 + 21,65 = 38,880 KNm

My_{tot} = 3,18 + 15,74 = 18,920 KNm

P total= 2298,2 + 1803,5 = 4101,7 KN

Eksentrisitas yang terjadi :

$$ex = \frac{Mx_{tot}}{R_{total}} = \frac{38,88}{4101,7} = 0,01\ m$$

$$ey = \frac{My_{tot}}{P_{total}} = \frac{18,92}{4101,7} = 0,01\ m$$

Kontrol tegangan yang terjadi :

$$\sigma_{terjadi} = \frac{P}{(H.(B-2.ex))+(B(H-2.ey))}$$

$$= \frac{4101,700}{(3.(5-2.0,01))+(5(3-2.0,01))}$$

$= 137,456 \text{ KN/m}^2 < 1,5.\sigma_{netto} = 1,5.\ 287,68 = 431,52 \text{ KN/m}^2$ **.....Ok !**

4.6.2.2 Perencananaan Geser Satu Arah

→ Ditinjau pada arah memanjang

P1 = 2298,2 KN P2 = 1803,5 KN

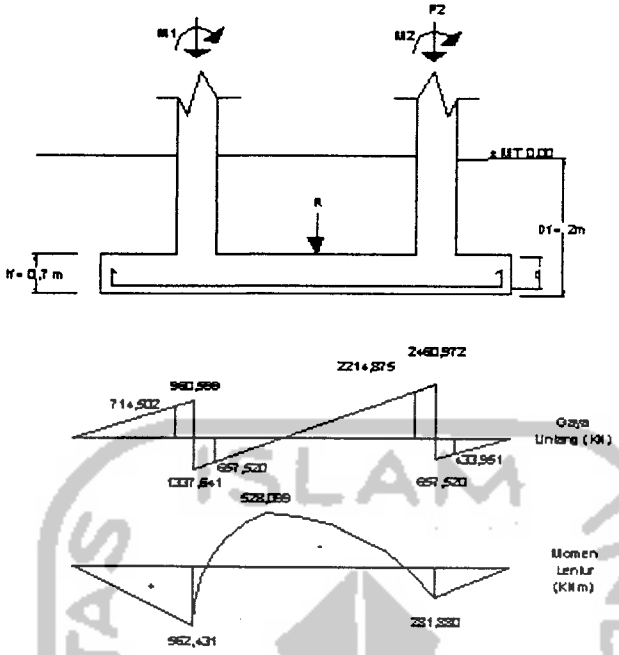
- Tegangan kontak yang terjadi :

$$qu = \frac{P_{tot}}{A}$$

$$= \frac{4101,7}{5.3} = 273,447 \text{ KN/m}^2$$

$qu \cdot H = 273,447 \cdot 3 = 820,340 \text{ KN/m}^2$

Pada gambar dibawah ini tampak bahwa kedudukan kolom relative dekat dengan ujung pondasi. dengan demikian pada arah memanjang struktur pondasi gabungan dapat berlaku senagai balok persegi lebar. Dengan menganggap kolom – kolom sebagai penopang dan pondasi akan menerima beban merata keatas yang berasal dari tekanan tanah. Dan untuk analisis geser dan momen yang terjadi dapat dianalisis dengan analisis balok sederhana.



Gambar 4.34 Diagram Geser dan Momen

- Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

Vu_d = Vmak – qu. d = 2214,875 – 820,34 . 0,7155 = 1627,933 KN

$Vu/\phi = 1627,933/0,6 = 2713,222$ KN

- Kekuatan beton menahan geser:

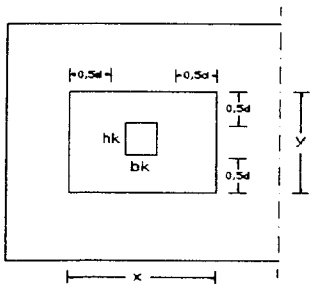
Vc = 1/6. $\sqrt{f'c}$. B. d = 1/6. $\sqrt{25}$. 5 . 0,7155 . 10^3

= 2981,25 KN ≥ Vu/ϕ = 2713,222 KN .....**Ok !**

4.6.2.3 Perencananaan Geser Dua Arah

→ Ditinjau masing-masing kolom pada arah pendek.

1. Kolom P1



Gambar 4.35 Bidang Geser 2 Arah kolom P1

x = bk + d = 600 + 715,5

= 1315,5 mm = 1,316 m

y = hk + d = 600 + 715,5

= 1315,5 mm = 1,316 m

• Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

$Vu = P_1 - qu\,(x \,.\, y)$

$=2298{,}2 - 273{,}447(1{,}316 \,.1{,}316) = 1824{,}629$ KN

$Vu/\phi = 1824{,}629/0{,}6 = 3041{,}049$ KN

• Kekuatan beton menahan geser :

$\beta c = \frac{sisipanjang}{sisipendek} = \frac{5}{3} = 1{,}667$

bo = 2 . (x + y) = 2 . (1315,5 + 1315,5) = 5262 mm

$Vc_1 = (1 + 2/\beta c).(2.\sqrt{f'c}).\ bo.\ d$

$= (1 + 2/1{,}667).(2.\sqrt{25}).\ 5262\ .\ 715{,}5.\ 10^{-3}$

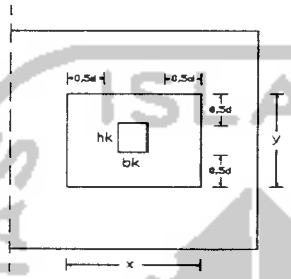
= 82829,142 KN

$Vc_2 = 4.\ \sqrt{f'c}\ .\ bo.\ d$

$= 4.\ \sqrt{25}\ .\ 5262.\ 715{,}5.\ 10^{-3} = 75299{,}220$ KN

Jadi Vc = 75299,220 KN ≥ Vu/ϕ = 3041,049 KN **.....Ok !**

2. Kolom P2



Gambar 4.36 Bidang Geser 2 Arah kolom P2

x = bk + d = 600 + 715,5

= 1315,5 mm = 1,316 m

y = hk + d = 600 + 715,5

= 1315,5 mm = 1,316 m

- Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

$Vu = P_2 - \sigma_{terjadi}\ (x\ .\ y)$

= 1803,5 – 273,447(1,316 .1,316) = 1329,929 KN

$Vu/\phi = 1329{,}929/0{,}6 = 2216{,}548$ KN

- Kekuatan beton menahan geser :

$\beta c = \frac{sisipanjang}{sisipendek} = \frac{5}{3} = 1{,}667$

bo $= 2 . (x + y) = 2 . (1315{,}5 + 1315{,}5) = 5262$ mm

$Vc_1 = (1 + \frac{2}{\beta c}).(2.\sqrt{f'c}).\ bo.\ d$

$= (1 + \frac{2}{1{,}667}).\ (2.\sqrt{25}).\ 5262 . 715{,}5.\ 10^{-3}$

$= 82829{,}142$ KN

$Vc_2 = 4.\ \sqrt{f'c}\ .\ bo.\ d$

$= 4.\ \sqrt{25}\ .\ 5262.\ 715{,}5.\ 10^{-3} = 75299{,}220$ KN

Jadi Vc = 75299,220 KN $\geq \frac{Vu}{\phi} = 2216{,}548$ KN .....**Ok !**

4.6.2.4 Perencanaan Tulangan Lentur Pondasi Gabungan

1. Arah Memanjang (B)

$M^{+}_{mak} = 562{,}431$ KNm

$M^{-}_{mak} = 528{,}099$ KNm

a. Momen Positif

Mu = 562,431 KNm

$\frac{Mu}{\phi} = \frac{562{,}431}{0{,}8} = 658{,}039$ KNm

- Digunakan tulangan pokok $\varnothing_{19}$ mm, maka : $A_{1\varnothing} = 283{,}529\ mm^2$
- Tebal pelat pondasi : tf = 800 mm, selimut beton (Pb) = 75 mm

d $= tf - Pb - 0{,}5.\ \varnothing_{tul.\ pokok} = 800 - 75 - 0{,}5.\ 19 = 715{,}5$ mm

$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.25} = 18{,}824$

Koefisien ketahanan (Rn), b = 3000 mm

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{658{,}039.10^6}{3000.715{,}5^2} = 0{,}428 \text{ MPa}$$

Rasio Tulangan :

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

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25.0{,}85}{400}.\left(\frac{600}{600+400}\right) = 0{,}0271$$

$$\rho_{mak} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}0271 = 0{,}0203$$

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

$$= \frac{1}{18{,}824}.\left(1-\sqrt{1-\frac{2.18{,}824.0{,}428}{400}}\right) = 0{,}0011 < \rho_{mak} = 0{,}020$$

$$< \rho_{min} = 0{,}0035$$

$1{,}33\ \rho_{ada} = 0{,}0014 < \rho_{min} = 0{,}0035$, maka :

$\rho_{perlu} = 0{,}0014$

$As_{perlu} = \rho_{perlu}\ .\ b\ .\ d = 0{,}0014.\ 3000\ .\ 715{,}5 = 3089{,}439\ mm^2$

$0{,}002.b.h = 0{,}002.3000.800 = 4800\ mm^2 > As_{perlu}$, maka :

$As_{perlu} = 4800\ mm^2$

Jarak antar tulangan :

$$s \le \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{283{,}385.3000}{4800} = 177{,}116 \text{ mm}$$

$s \le 2.h = 2\ .800 = 1600$ mm

$s \le 250$ mm

→ Dipakai Tulangan Pokok : D_{19} – 170 mm

$$As_{ada} = \frac{A_{1\emptyset}.b}{s} = \frac{283,529.3000}{170} = 5000,912 \text{ mm}^2$$

- Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0,85.f'c.b} = \frac{5000,912.400}{0,85.25.3000} = 31,378 \text{ mm}$$

$$Mn = As_{ada}.fy.\ (d - {}^{a}/_{2})$$

$$= 5000,912.\ 400\ (715,5 - {}^{31,378}/_{2})$$

$$= 1399,877 \text{ KNm} \geq 1,33\ .\ {}^{Mu}/_{\phi} = 875,192 \text{ KNm} \quad \textbf{.....Ok !}$$

b. Momen Negatif

$$Mu = 528,099 \text{ KNm}$$

$${}^{Mu}/_{\phi} = \frac{528,099}{0,8} = 660,124 \text{ KNm}$$

- Digunakan tulangan pokok $\emptyset_{19}$ mm, maka : $A_{1\emptyset} = 283,529 \text{ mm}^2$
- Tebal pelat pondasi : tf = 800 mm, selimut beton (Pb) = 75 mm

$$d = tf - Pb - 0,5.\ \emptyset_{tul.\ pokok} = 800 - 75 - 0,5.\ 19 = 715,5 \text{ mm}$$

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.25} = 18,824$$

Koefisien ketahanan (Rn), b = 3000 mm

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{660,124.10^6}{3000.715,5^2} = 0,430 \text{ MPa}$$

Rasio Tulangan :

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

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25.0{,}85}{400}.\left(\frac{600}{600+400}\right) = 0{,}0271$$

$$\rho_{mak} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}027 = 0{,}0203$$

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

$$= \frac{1}{18{,}824}.\left(1-\sqrt{1-\frac{2.18{,}824.0{,}430}{400}}\right) = 0{,}0011 < \rho_{mak} = 0{,}020$$

$$< \rho_{min} = 0{,}0035$$

$1{,}33\ \rho_{ada} = 0{,}0014 ,< \rho_{min} = 0{,}0035$, maka :

$\rho_{perlu} = 0{,}0014$

$As_{perlu} = \rho_{perlu}\ .\ b\ .\ d = 0{,}0014.\ 3000\ .\ 715{,}5 = 3099{,}330\ mm^2$

$0{,}002.b.h = 0{,}002.3000.800 = 4800\ mm^2 > As_{perlu}$, maka :

$As_{perlu} = 4800\ mm^2$

Jarak antar tulangan :

$$s \le \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{283{,}529.3000}{4800} = 177{,}116\ mm$$

$s \le 2.h = 2\ .800 = 1600\ mm$

$s \le 250\ mm$

→ **Dipakai Tulangan Pokok : D_{19} – 170 mm**

$$As_{ada} = \frac{A_{1\theta}.b}{s} = \frac{283{,}529.3000}{170} = 5000{,}912\ mm^2$$

• Kontrol Kapasitas Lentur Pelat pondasi :

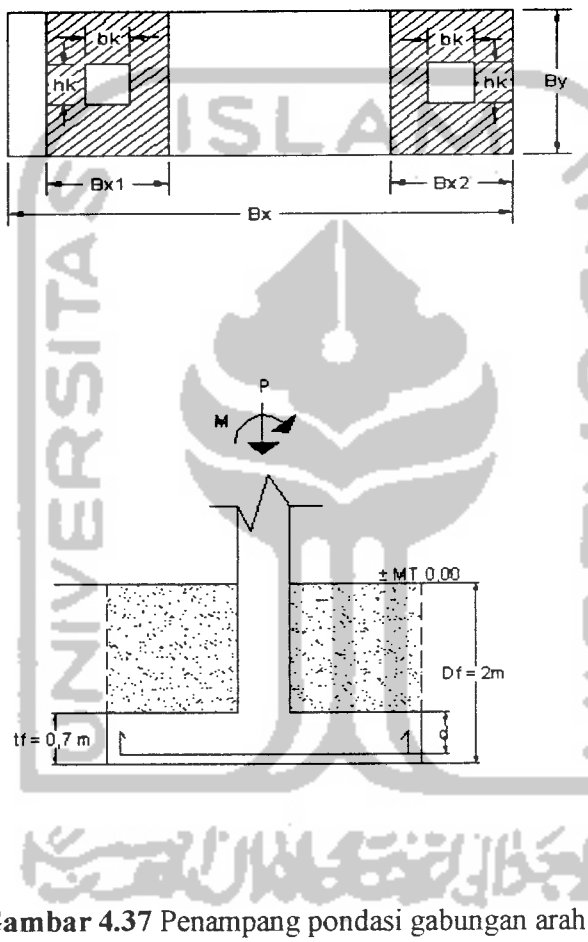
$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{5000{,}912.400}{0{,}85.25.3000} = 31{,}378\ mm$$

$Mn = As_{ada}.fy.(d - a/2)$

$= 5000{,}912.400\,(715{,}5 - 31{,}378/2)$

$= 1399{,}877$ KNm $\geq 1{,}33 . Mu/\phi = 877{,}965$ KNm **.....Ok !**

2. Arah Memendek (y)



Gambar 4.37 Penampang pondasi gabungan arah y

Bx1 = Bx2 = bk + d = 0,600 + 0,7155 = 1,316 m

a. Kolom P1

$$qu = \frac{P1}{By.Bx1} + \frac{6.Mx}{By^2.Bx1}$$

$$= \frac{2298,2}{3.1,316} + \frac{6.17,23}{3^2.1,316}$$

$qu = 590,846\ KN/m^2$

$$l = \frac{By - hk}{2} = \frac{3 - 0,6}{2} = 1,2\ m$$

$Mu = 0,5\ .\ qu\ .\ L^2 = 0,5\ .\ 590,846\ .\ 1,2^2 = 425,409\ KNm$

$$Mu/\phi = \frac{425,409}{0,8} = 531,761\ KNm$$

- Digunakan tulangan pokok $\varnothing_{19}$ **mm**, maka : $A_{1\varnothing} = 283,529mm^2$
- Tebal pelat pondasi : tf = 800 mm, selimut beton (Pb) = 75 mm

$d = tf - Pb - 0,5.\ \varnothing_{tul.\ pokok} = 800 - 75 - 0,5.\ 19 = 715,5\ mm$

$$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.25} = 18,824$$

Koefisien ketahanan (Rn), b = 1000 mm

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{531,761.10^6}{1000.715,5^2} = 1,039\ MPa$$

Rasio Tulangan :

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

$$\rho b = \frac{0,85.f'c.\beta_1}{fy}\left(\frac{600}{600+fy}\right) = \frac{0,85.25.0,85}{400}.\left(\frac{600}{600+400}\right) = 0,0271$$

$\rho_{mak} = 0,75.\ \rho b = 0,75.\ 0,027 = 0,0203$

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

$$= \frac{1}{18,824}\left(1-\sqrt{1-\frac{2.18,824.1,039}{400}}\right) = 0,00266 < \rho_{mak} = 0,020$$

$$< \rho_{min} = 0,0035$$

$1,33\ \rho_{ada} = 0,00354 > \rho_{min} = 0,0035$, maka : $\rho_{perlu} = 0,0035$

$As_{perlu} = \rho_{perlu} . b . d = 0,0035 . 1000 . 715,5 = 2504,250\ mm^2$

$0,002.b.h = 0,002.1000.800 = 1600\ mm^2 < As_{perlu}$, maka :

$As_{perlu} = 2504,250\ mm^2$

Jarak antar tulangan :

$$s \leq \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{283,529.1000}{2504,250} = 113,219\ mm$$

$s \leq 2.h = 2 .800 = 1600\ mm$

$s \leq 250\ mm$

→ **Dipakai Tulangan Pokok : D_{19} – 110 mm**

$$As_{ada} = \frac{A_{19}.b}{s} = \frac{283,529.1000}{110} = 2577,536\ mm^2$$

- Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0,85.f'c.b} = \frac{2577,536.400}{0,85.25.1000} = 48,518\ mm$$

$Mn = As_{ada}.fy. (d - a/2)$

$= 2577,536. 400 (715,5 - 48,518/2)$

$= 707,149\ KNm \geq Mu/\phi = 531,761\ KNm$ .....**Ok !**

b. Kolom P2

$$qu = \frac{P2}{By.Bx2} + \frac{6.Mx2}{By^2.Bx2}$$

$$= \frac{1803,5}{3.1,316} + \frac{6.21,65}{3^2.1,316}$$

$qu = 467,781\ KN/m^2$

$$L = \frac{By - hk}{2} = \frac{3 - 0,6}{2} = 1,2\ m$$

$Mu = 0,5\ .\ qu\ .\ L^2 = 0,5\ .\ 467,781\ .\ 1,2^2 = 336,802\ KNm$

$$Mu/\phi = \frac{336,802}{0,8} = 421,003\ KNm$$

- Digunakan tulangan pokok $\varnothing_{19}$ **mm**, maka : $A_{1\varnothing} = 283,529mm^2$
- Tebal pelat pondasi : tf = 800 mm, selimut beton (Pb) = 75 mm

  $d = tf - Pb - 0,5.\ \varnothing_{tul.\ pokok} = 800 - 75 - 0,5.\ 19 = 715,5\ mm$

  $$m = \frac{fy}{0,85.f'c} = \frac{400}{0,85.25} = 18,824$$

Koefisien ketahanan (Rn), b = 1000 mm

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{421,003.10^6}{1000.715,5^2} = 0,822\ MPa$$

Rasio Tulangan :

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

$$\rho b = \frac{0,85.f'c.\beta_1}{fy}\left(\frac{600}{600+fy}\right) = \frac{0,85.25.0,85}{400}\left(\frac{600}{600+400}\right) = 0,027$$

$\rho_{mak} = 0,75.\ \rho b = 0,75.\ 0,027 = 0,020$

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

$$= \frac{1}{18,824}\left(1-\sqrt{1-\frac{2.18,824.0,822}{400}}\right) = 0,0021 < \rho_{mak} = 0,020$$

$$< \rho_{min} = 0,0035$$

$1,33\ \rho_{ada} = 0,0028 < \rho_{min} = 0,0035$, maka : $\rho_{perlu} = 0,0028$

$As_{perlu} = \rho_{perlu} . b . d = 0,0028. 1000 . 715,5 = 1995,841\ mm^2$

$0,002.b.h = 0,002.1000.800 = 1600\ mm^2 < As_{perlu}$, maka :

$As_{perlu} = 1995,841\ mm^2$

Jarak antar tulangan :

$$s \leq \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{283,529.1000}{1995,841} = 142,060\ mm$$

$s \leq 2.h = 2 .800 = 1600\ mm$

$s \leq 250\ mm$

→ **Dipakai Tulangan Pokok : D_{19} – 140 mm**

$$As_{ada} = \frac{A_{1\theta}.b}{s} = \frac{283,529.1000}{140} = 2025,207\ mm^2$$

• Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0,85.f'c.b} = \frac{2025,207.400}{0,85.25.1000} = 38,122\ mm$$

$Mn = As_{ada}.fy. (d - a/2)$

$= 2025,207. 400 (715,5 - 38,122/2)$

$= 564,173\ KNm \geq Mu/\phi = 421,003\ KNm$ .....**Ok !**

4.6.2.5 Perencanaan Tulangan Susut Pondasi

$As_{susut} = 0{,}002.\ b.\ h = 0{,}002.\ 1000.\ 800 = 1600\ mm^2$

- Digunakan tulangan bagi ∅13 mm, maka: $A_{1\varnothing} = 132{,}67\ mm^2$

Jarak antar tulangan susut :

$$s \leq \frac{A_{\theta 1}.b}{As_{susut}} = \frac{132{,}670.1000}{1600} = 82{,}92\ mm \approx 80\ mm$$

→ **Dipakai Tulangan Susut : P_{13} – 80 mm**