BAB IV

PERENCANAAN STRUKTUR

4.1 Rangka Atap Kuda-kuda Baja

4.1.1 Data Konstruksi Rangka Atap

- Jarak antar kuda-kuda maksimum (b) = 5 m
- Panjang bentang (L) = 15 m
- Mutu baja profil :

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

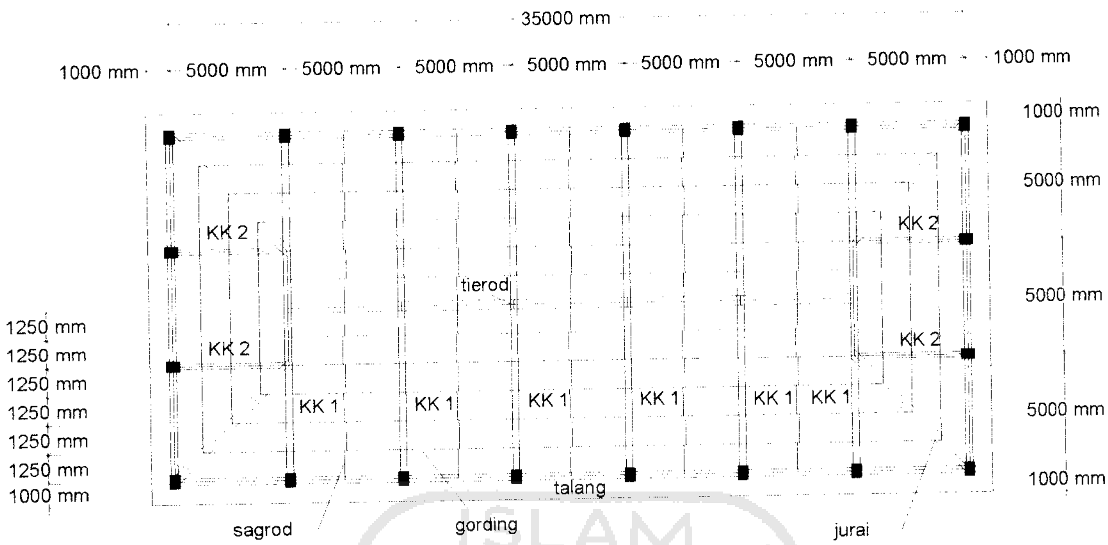
  Kuat tarik (Fu) = 58 Ksi = 4077 kg/cm^2
- Mutu baut A325X (Non Full Draat) :

  Tegangan tarik (Ft) = 44 Ksi = 3093 kg/cm^2

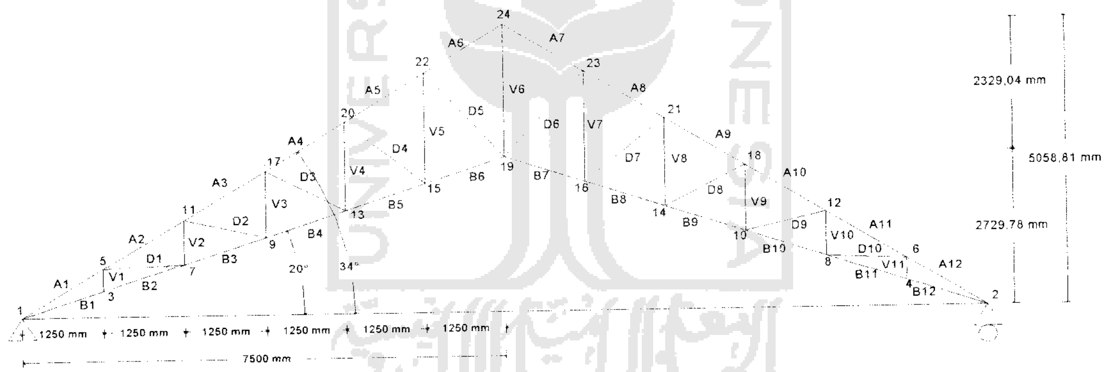
  Tegangan geser (Fv) = 30 Ksi = 2109 kg/cm^2
- Untuk atap genteng $\alpha \geq 22,5°$, sedangkan untuk atap asbes, seng $\alpha \geq 10°$. Pada perencanaan ini dipakai atap genteng dengan $\alpha_1 = 34°$ dan $\alpha_2 = 20°$.
- Usuk dan reng dipakai kayu sedangkan gording dipakai baja jenis Light Lip Channel
- Jurai menggunakan profil Double Light Lip Channel dan rangka kuda-kuda menggunakan profil Double Angel.

4.1.2 Jumlah dan Jarak Antar Gording

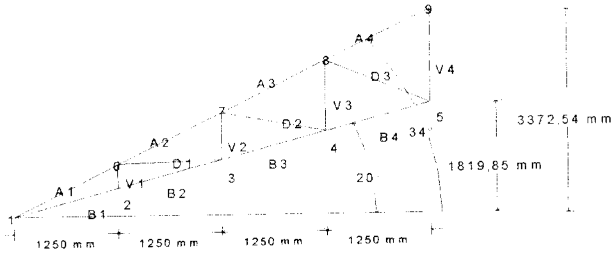
- Jarak gording maksimum (Atap Genteng) = 2,5 m
- Panjang sisi miring kuda-kuda (M) $= \dfrac{0,5L}{\cos\alpha} = \dfrac{0,5.15}{\cos 34} = 9,0466$ m
- Jumlah gording setengah bentang (n) = 7 buah
- Jarak antar gording (Lg) = $\dfrac{1,25}{\cos 34} = 1,5078 \text{ m} < 2,5 \text{ m}$



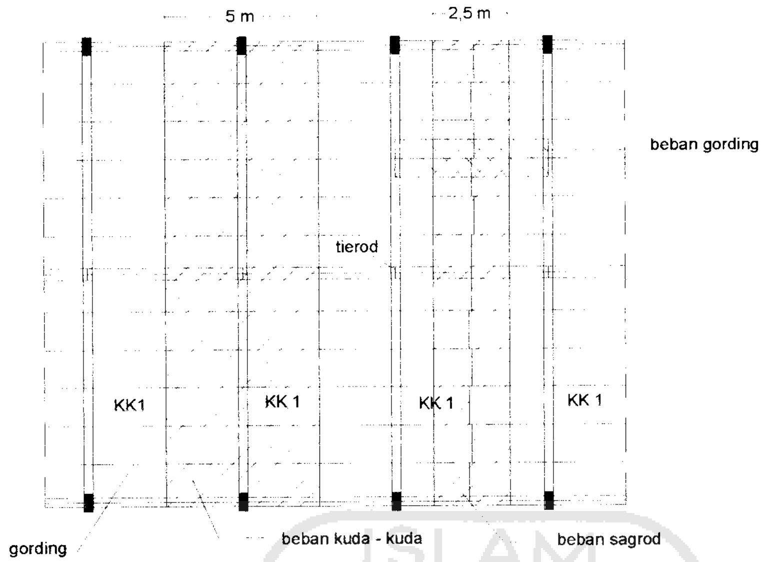
Gambar 4.1 Denah rencana kuda-kuda



Gambar 4.2 Rencana rangka kuda-kuda (KK1)



Gambar 4.3 Rencana rangka kuda-kuda (KK2)



Gambar 4.4 Pembebanan Kuda-kuda

4.1.3 Perencanaan Gording

a. Pembebanan Gording

1. Beban Tetap

Berat genteng (tabel 2.1.PPIUG'83) = 50 kg/m^2 x 1,5078 m = 75,39 kg/m'

- Beban hidup (Pasal 3.2.2.b.PPIUG'83) = 20 kg/m^2 x 1,5078 m = 30,15 kg/m'
- Berat gording taksiran (7 s/d 10 kg/m') = 10 kg/m'

q_{total} = 115,54 kg/m'

Mekanika gording

$q\perp = q_{total} \cdot \cos\alpha = 115{,}54 \cdot \cos 34° = 95{,}79$ kg/m'

$q\,// = q_{total} \cdot \sin\alpha = 115{,}54 \cdot \sin 34° = 64{,}61$ kg/m'

2. Beban Angin

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

- Angin Tekan (Wt)

$C_1 = 0{,}02\,\alpha - 0{,}4 = 0{,}02 \cdot 34 - 0{,}4 = +\,0{,}28$ (tekan)

$Wt = C_1 \cdot Wa \cdot Lg = 0{,}28 \cdot 25 \cdot 1{,}5078 = +\,10{,}554$ kg/m' (tekan)

- Angin Hisap (Wh)

  $C_2 = -0,4$

  $Wh = C_2 \cdot Wa \cdot Lg = -0,4 \cdot 25 \cdot 1,5078 = -15,078$ kg/m'(hisap)

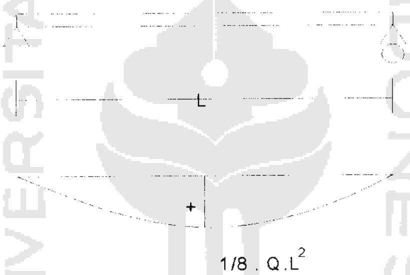
  $W\perp = +10,554$ kg/m' (tekan)

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

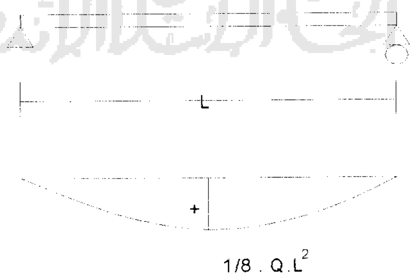
b. Momen yang terjadi

- akibat beban tetap

  $$M\perp \text{ maks} = \frac{1}{8} q \perp L^2 = \frac{1}{8} 95,79.5^2 = 299,345 \text{ kgm'}$$



  $$M // \text{ maks} = \frac{1}{8} q // L^2 = \frac{1}{8} 64,61.5^2 = 201,906 \text{ kgm'}$$



- akibat beban angin

  $$M \perp \text{ maks} = \frac{1}{8} W \perp L^2 = \frac{1}{8} 10,554.5^2 = 32,983 \text{ kgm'}$$

c. Penentuan Profil Baja

Dicoba profil Light Lip Channel (Ir.Morisco, hal 52) **C 150x50x20x3,2**

d = 150 mm = 5,906 in

bf = 50 mm = 1,969 in

tf = tw = 3,2 mm = 0,126 in

Sx = 37,4 cm^3	fy = 2531 kg/cm^2
Sy = 8,19 cm^3	E = 2,1 x 10^6 kg/cm^2
Ix = 280 cm^4	Fu = 4078 kg/cm^2
Iy = 28,3 cm^4	W = 6,76 kg/m

d. Kontrol Tegangan

- Pembebanan Sementara

$$fbx = \frac{M \perp maks}{Sx} = \frac{(299{,}345 + 32{,}983)x100}{37{,}4} = 888{,}576 \text{ kg/cm}^2 \qquad \textbf{(3.1.2)}$$

$$fby = \frac{M // maks}{Sy} = \frac{201{,}906x100}{8{,}19} = 2465{,}278 \text{ kg/cm}^2 \qquad \textbf{(3.1.3)}$$

cek kompak:

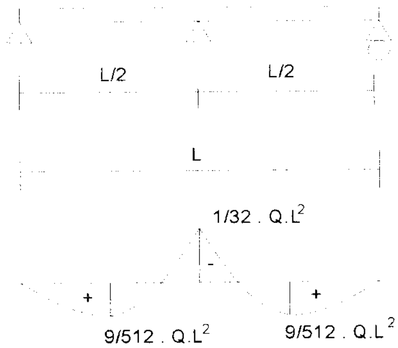
$$\frac{bf}{2tf} = \frac{1{,}969}{2.0{,}126} = 7{,}813 < \frac{65}{\sqrt{Fy}} = \frac{65}{\sqrt{36}} = 10{,}83$$

$$\frac{d}{tw} = \frac{5{,}906}{0{,}126} = 46{,}873 < \frac{640}{\sqrt{Fy}} = \frac{640}{\sqrt{36}} = 106{,}67$$

Profil kompak, maka Fbx = 0,66 Fy dan Fby = 0,75 Fy

$$\frac{fbx}{0{,}66fy} + \frac{fby}{0{,}75fy} = \frac{888{,}576}{0{,}66.2531} + \frac{2465{,}278}{0{,}75.2531} = 1{,}831 \leq 1{,}0 \textbf{ (tidak aman!)} \qquad \textbf{(3.1.1)}$$

sagrod dipasang di tengah bentang :



$$M\,//\,\text{maks} = \frac{1}{32} q\,//\,L^2 = \frac{1}{32} 64{,}61.5^2 = 50{,}478 \text{ kgm'}$$

Kontrol tegangan dengan profil yang sama di atas :

- Pembebanan Sementara

$$\text{fbx} = \frac{M \perp \text{maks}}{Sx} = \frac{(299{,}345 + 32{,}983)x100}{37{,}4} = 888{,}576 \text{ kg/cm}^2$$

$$\text{fby} = \frac{M\,//\,\text{maks}}{Sy} = \frac{50{,}478x100}{8{,}19} = 616{,}337 \text{ kg/cm}^2$$

$$\frac{\text{fbx}}{0{,}66\text{fy}} + \frac{\text{fby}}{0{,}75\text{fy}} = \frac{888{,}576}{0{,}66.2531} + \frac{616{,}337}{0{,}75.2531} = 0{,}857 \leq 1{,}0 \ \textbf{(Ok!)}$$

- Pembebanan Tetap

$$\text{fbx} = \frac{M \perp \text{maks}}{Sx} = \frac{(299{,}345)x100}{37{,}4} = 800{,}388 \text{ kg/cm}^2$$

$$\text{fby} = \frac{M\,//\,\text{maks}}{Sy} = \frac{50{,}478x100}{8{,}19} = 616{,}337 \text{ kg/cm}^2$$

$$\frac{\text{fbx}}{0{,}66\text{fy}} + \frac{\text{fby}}{0{,}75\text{fy}} = \frac{800{,}388}{0{,}66.2531} + \frac{616{,}337}{0{,}75.2531} = 0{,}804 \leq 1{,}0 \quad \textbf{(Ok!)}$$

e. Kontrol Lendutan

$$\delta \perp = \frac{5}{384} \cdot \frac{q \perp . L^4}{E.Ix} = \frac{5}{384} \frac{1{,}0(95{,}79).5^4.10^6}{2{,}1.10^6.280} \qquad \textbf{(3.1.4)}$$

$$= 1{,}226 \text{ cm} \leq \frac{L}{360} = \frac{5.100}{360} = 1{,}389 \text{ cm } \textbf{(Ok!)}$$

$$\delta // = \frac{5}{384} \frac{q//(L/2)^4}{E.Iy} = \frac{5}{384} \frac{64{,}61(2{,}5)^4.10^6}{2{,}1.10^6.28{,}3} \qquad \textbf{(3.1.5)}$$

$$= 0{,}453 \text{ cm} \leq \frac{L}{360} = \frac{5.100}{360} = 1{,}389 \text{ cm} \quad \textbf{(Ok!)}$$

jadi lendutan maksimum : $\sqrt{(1{,}226^2 + 0{,}453^2)} = 1{,}307$ cm **(Ok!)** **(3.1.6)**

Jadi profil **C 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 9,0466 = 452,3317 kg/m'

   - Beban hidup x sisi miring (M)

     = 20 x 9,0466 = 180,9327 kg/m'

   - Beban gording = berat gording x jml gording = 6,76 x 7 = 47,320 kg/m'

     P = 680,5844 kg/m'

Ss = L/2 = 5/2 = 2,5 m

P// = P.sin α . Ss = 680,5844 . sin 34 . 2,5 = 951,4449 kg **(3.1.8)**

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

$$D = \sqrt{\frac{P//.4}{0,33.Fu.\pi}} = \sqrt{\frac{951,4449.4}{0,33.4078.\pi}} = 0,9490 \text{ cm} = 9,490 \text{ mm} \qquad \textbf{(3.1.10)}$$

dipakai sagrod = D + 3 = 9,490 + 3 = 12,490 mm

2. Tierod

Beban Tierod = T = P// . cos α = 951,4449 . cos 34 = 788,7836 kg

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

$$D = \sqrt{\frac{T.4}{0,33.Fu.\pi}} = \sqrt{\frac{788,7836.4}{0,33.4078.\pi}} = 0,8641 \text{ cm} = 8,641 \text{ mm} \qquad \textbf{(3.1.15)}$$

dipakai tierod = D + 3 = 8,641 + 3 = 11,641 mm

Sagrod dan Tierod dipakai diameter = 12 mm (P12)

4.1.5 Perencanaan Kuda – Kuda

4.1.5.1 Pembebanan dan Gaya batang Rencana Kuda – Kuda

Beban Tetap :

- Berat gording (Light Lip Channel) = 6,76 kg/m’
- Berat eternit (Tabel 2.1,PPIUG ’83) = 11 kg/m^2
- Penggantung langit – langit (dari kayu) = 7 kg/m^2
- Berat penutup atap (genteng) = 50 kg/m^2
- Beban hidup = 20 kg/m^2

- Berat kuda-kuda taksiran :

$$W \text{ taksiran} = \left(10 \pm \left(\frac{L-12}{3}\right).5\right).\text{jarak kuda} - \text{kuda}$$

$$= \left(10 + \left(\frac{15-12}{3}\right).5\right).5 = 75 \text{ kg/m'}$$

Beban – beban pada joint :

a. $P_1 = P_{13}$

Beban gording	= 6,76 x 5		= 33,80 kg
Berat penutup atap	= 50 x 5 x ½ (1,25/cos 34)		= 188,472 kg
		qD	= 222,272 kg
			= 222 kg
Beban hidup (qL)	= 20 x 5 x ½(1,25/cos 34)		= 75,389 kg
			= 75 kg

b. P_2 s/d P_{12}

Beban gording	= 6,76 x 5		= 33,80 kg
Berat penutup atap	= 50 x 5 x (1,25/cos34)		= 376,943 kg
		qD	= 410,743 kg
			= 410 kg
Beban hidup (qL)	= 20 x 5 x (1,25/cos 34)		= 150,772 kg
			= 150 kg

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

Berat eternit dan penggantung	= 18 x 5 x 0.5(1,25/cos 20)		= 60 kg
Berat kuda-kuda	= 75 x 0.5(1,25/cos 20)		= 56,541 kg
		qD	= 116,541 kg
			= 116 kg

d. P_2' s/d P_{12}'

Berat eternit dan penggantung	= 18 x 5 x (1,25/cos 20)	= 120 kg
Berat kuda-kuda	= 75 x (1,25/cos 20)	= 118,083 kg
	qD	= 238,083 kg
		= 238 kg

Beban angin :

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

Koefisien angin :

- Angin Tekan (Wt)

  $C_1 = 0{,}02\ \alpha - 0{,}4 = 0{,}02\ .\ 34 - 0{,}4 = +0{,}28$ (tekan)

- Angin Hisap (Wh)

  $C_2 = -\ 0{,}4$

Beban-beban Angin :

$Wt = C_1\ .\ Wa = +\ 0{,}28\ .\ 25 = +\ 7$ kg/m^2 (tekan)

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

a. Angin kiri

akibat angin tekan :

$Wt_1 = Wt_7$	= 7 x 5 x ½(1,25/cos 34)	= + 26 kg
Wt_2 s/d Wt_6	= 7 x 5 x (1,25/cos 34)	= + 53 kg

Akibat angin hisap :

$Wh_1 = Wh_7$	= -10 x 5 x ½(1,25/cos 34)	= - 38 kg
Wh_2 s/d Wh_6	= -10 x 5 x (1,25/cos 34)	= - 75 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 tabel 4.5

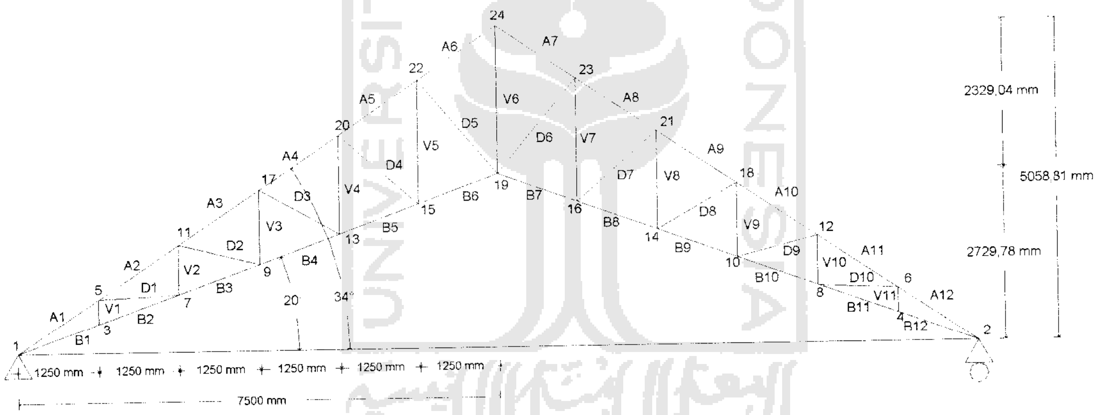
1. Data profil baja yang digunakan

   Modulus of Elasticity (Es) = $2{,}1.10^{10}$ kg/m^2 = $2{,}1.10^{6}$ kg/cm^2

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

   Asumsi profil 2L 80x80x10 dengan:

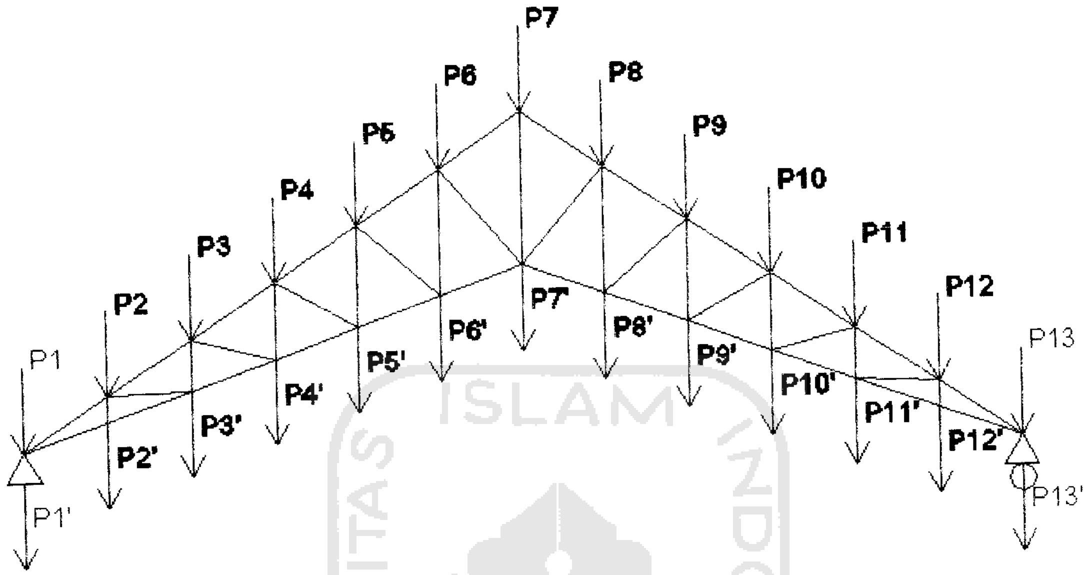
   b = 80 mm   tf = 10 mm   tw = 10 mm



Gambar 4.5 Rangka Kuda-kuda KK1

2. Data – data pembebanan yang dimasukkan pada SAP2000

a. Akibat beban tetap



Gambar 4.6 Gaya Akibat Beban Tetap

Tabel 4.1 Gaya P_1 sampai dengan P_{13}

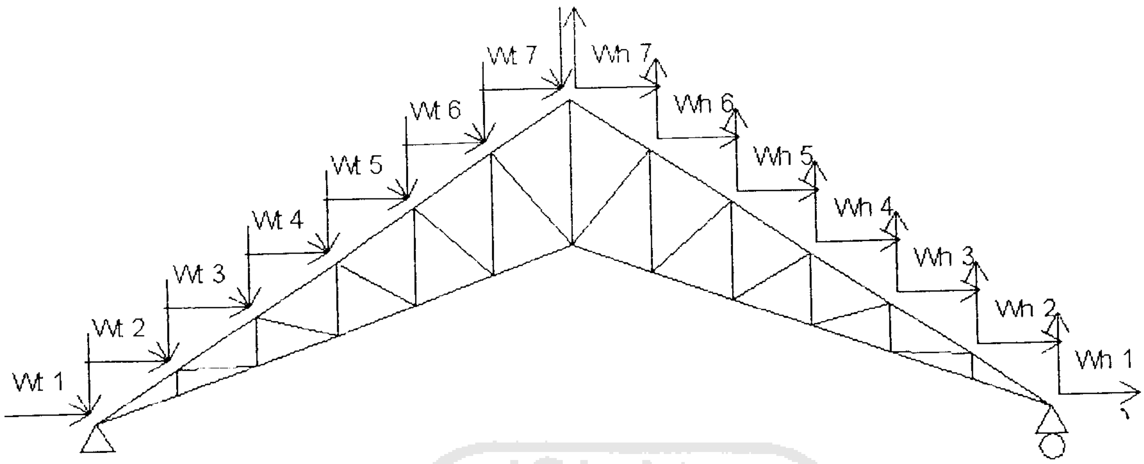
Nama Gaya	Beban Mati (qD) kg	Beban Hidup (qL) kg
$P_1 = P_{13}$	- 222	- 75
P_2 s/d P_{12}	- 410	- 150

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
$P_1' = P_{13}'$	- 60
P_2' s/d P_{12}'	- 120

b. Akibat beban angin kiri

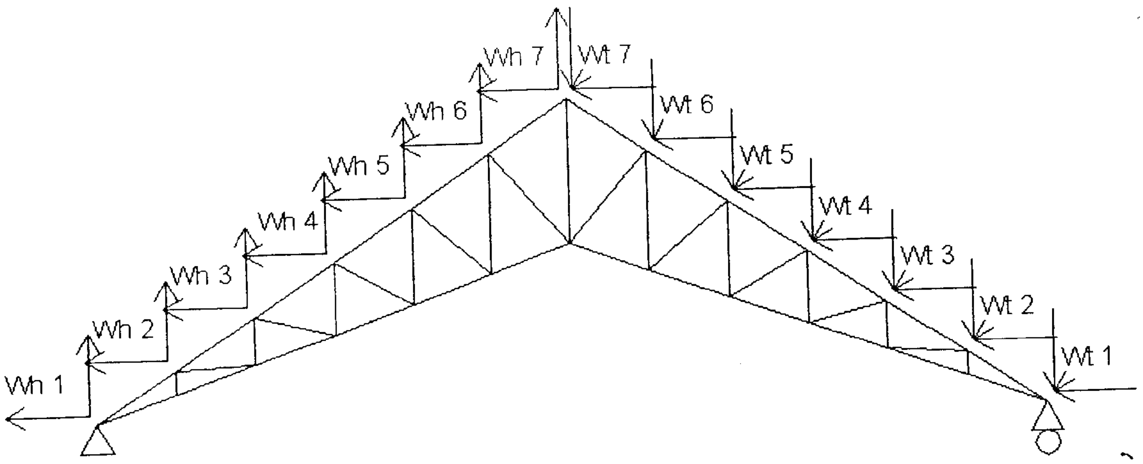


Gambar 4.7 Gaya Akibat Angin Kiri

Tabel 4.3 Gaya Tekan dan Hisap Angin Kiri

Nama Gaya	Gaya akibat Beban Angin Kiri (Wki) kg	Gaya Vertikal = Wki x cos 34° (kg)	Gaya Horisontal = Wki x sin 34° (kg)
$Wt_1 = Wt_7$	26	+22	- 15
$Wt_{2\ s/d}\ Wt_6$	53	+ 44	- 30
$Wh_1 = Wh_7$	-38	+31	+ 21
$Wh_{2\ s/d}\ Wh_6$	-75	+ 63	+ 42

c. Akibat beban angin kanan



Gambar 4.8 Gaya Akibat Angin Kanan

Tabel 4.4 Gaya Hisap dan Tekan Angin Kanan

Nama Gaya	**Gaya akibat Beban Angin Kanan (Wka) kg**	**Gaya Vertikal = Wka x cos 34° (kg)**	**Gaya Horisontal = Wka x sin 34° (kg)**
$Wt_1 = Wt_7$	26	- 22	- 15
$Wt_{2\ s/d}\ Wt_6$	53	- 44	- 30
$Wh_1 = Wh_7$	-38	- 31	+ 21
$Wh_{2\ s/d}\ Wh_6$	-75	- 63	+ 42

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Tabel 4.5 Gaya Batang Yang Terjadi Pada Rangka Atap

4.6.

Nama Batang	Panjang Batang (m)	Gaya Batang Yang Terjadi			Kombinasi Pembebanan			Beban Rencana (kg)
		P (kg)	Wki (kg)	Wkn (kg)	P + Wki (kg)	P + Wkn (kg)	1,33 . P (kg)	
Atas								
A1	1.5078	-16386.970	-576.415	947.960	-16963.385	-15439.010	-21794.670	-16386.970
A2	1.5078	-15691.654	-532.184	884.510	-16223.838	-14807.144	-20869.900	-15691.654
A3	1.5078	-14192.741	-443.415	758.593	-14636.156	-13434.149	-18876.346	-14192.741
A4	1.5078	-12667.729	-356.036	633.936	-13023.765	-12033.793	-16848.080	-12667.729
A5	1.5078	-11116.771	-268.318	508.715	-11385.089	-10608.057	-14785.305	-11116.771
A6	1.5078	-9518.212	-179.881	382.248	-9698.093	-9135.964	-12659.222	-9518.212
A7	1.5078	-9518.208	-223.065	425.433	-9741.273	-9092.776	-12659.217	-9518.208
A8	1.5078	-11116.773	-98.693	339.089	-11215.466	-10777.684	-14785.308	-11116.773
A9	1.5078	-12667.729	27.294	250.606	-12640.435	-12417.123	-16848.080	-12667.729
A10	1.5078	-14192.741	152.798	162.380	-14039.943	-14030.361	-18876.346	-14192.741
A11	1.5078	-15691.654	278.964	73.363	-15412.690	-15618.291	-20869.900	-15691.654
A12	1.5078	-16386.970	383.883	-12.337	-16003.087	-16399.307	-21794.670	-16386.970
Bawah								
B1	1.3302	14357.929	945.388	-1264.210	15303.317	13093.719	19096.046	14357.929
B2	1.3302	14660.236	951.829	-1277.132	15612.065	13383.104	19498.114	14660.236
B3	1.3302	13819.204	875.522	-1165.603	14694.726	12653.601	18379.541	13819.204
B4	1.3302	12521.387	769.586	-1014.081	13290.973	11507.306	16653.445	12521.387
B5	1.3302	11178.731	661.662	-859.876	11840.393	10318.855	14867.712	11178.731
B6	1.3302	9796.870	552.881	-704.308	10349.751	9092.562	13029.837	9796.870
B7	1.3302	9796.870	288.016	-439.443	10084.886	9357.427	13029.837	9796.870
B8	1.3302	11178.731	132.934	-331.149	11311.665	10847.582	14867.712	11178.731
B9	1.3302	12521.387	-22.546	-221.950	12498.841	12299.438	16653.445	12521.387
B10	1.3302	13819.204	-177.140	-112.942	13642.064	13706.263	18379.541	13819.204
B11	1.3302	14660.236	-317.572	-7.730	14342.664	14652.506	19498.114	14660.236
B12	1.3302	14357.929	-313.906	-4.916	14044.024	14353.013	19096.046	14357.929

Lanjutan tabel 4.6

Nama Batang	Panjang Batang (m)	Gaya Batang Yang Terjadi			Kombinasi Pembebanan			Beban Rencana (kg)
		P (kg)	Wki (kg)	Wkn (kg)	P + Wki (kg)	P + Wkn (kg)	1,33 . P (kg)	
Diagonal								
D1	1.2518	-726.588	-64.188	95.743	-790.776	-630.845	-966.362	-726.588
D2	1.2907	-1255.789	-101.623	145.684	-1357.413	-1110.105	-1670.200	-1255.789
D3	1.4373	-1450.146	-115.835	165.603	-1565.981	-1284.543	-1928.694	-1450.146
D4	1.6636	-1719.325	-135.050	193.041	-1854.376	-1526.285	-2286.703	-1719.325
D5	1.9418	-1995.290	-157.366	224.217	-2152.656	-1771.073	-2653.735	-1995.290
D6	1.9418	-1995.290	226.888	-160.037	-1768.402	-2155.327	-2653.735	-1995.290
D7	1.6636	-1719.325	193.481	-135.491	-1525.844	-1854.816	-2286.703	-1719.325
D8	1.4373	-1450.146	167.276	-117.507	-1282.870	-1567.653	-1928.694	-1450.146
D9	1.2907	-1255.789	149.481	-105.420	-1106.309	-1361.209	-1670.200	-1255.789
D10	1.2518	-726.588	128.902	-97.347	-597.686	-823.935	-966.362	-726.588
Vertikal								
V1	0.3882	-118.579	-12.347	18.653	-130.926	-99.926	-157.710	-118.579
V2	0.7763	515.331	30.0586	-42.4539	545.390	472.877	685.391	515.331
V3	1.1645	947.722	62.225	-88.745	1009.947	858.977	1260.470	947.722
V4	1.5527	1367.909	94.185	-134.401	1462.094	1233.508	1819.320	1367.909
V5	1.9409	1818.225	126.434	-180.649	1944.660	1637.576	2418.240	1818.225
V6	2.3290	9949.198	232.828	-438.537	10182.026	9510.661	13232.433	9949.198
V7	1.9409	1818.225	-179.084	124.870	1639.141	1943.095	2418.240	1818.225
V8	1.5527	1367.909	-135.506	95.289	1232.404	1463.199	1819.320	1367.909
V9	1.1645	947.722	-89.939	63.419	857.783	1011.141	1260.470	947.722
V10	0.7763	515.331	-44.443	32.047	470.889	547.379	685.391	515.331
V11	0.3882	-118.579	6.069	0.237	-112.510	-118.342	-157.710	-118.579

Tabel 4.6 Reaksi Dukungan Yang Terjadi Pada Kuda-Kuda

Joint	Beban	Reaksi (kg)	
		Horizontal	Vertikal
1	Beban mati	-9.09E-11	3923.639
1	Beban hidup	-1.82E-11	904.663
1	Beban angin kanan	429.999	-143.116
1	Beban angin kiri	-429.999	30.616
1	kombinasi 1	-1.38E-10	4828.302
1	kombinasi 2	-515.9988	4685.187
1	kombinasi 3	515.9988	4858.918
2	Beban mati	0	3923.639
2	Beban hidup	0	904.663
2	Beban angin kanan	0	30.616
2	Beban angin kiri	0	-143.116
2	kombinasi 1	0	4828.302
2	kombinasi 2	0	4685.187
2	kombinasi 3	0	4858.918

4.1.7 Perencanaan Profil Kuda – Kuda

1. Batang Tekan

a. Batang Atas (A_1 s/d A_{12})

(tabel 4.5 halaman 65 Gaya Batang Yang Terjadi Pada Rangka Atap)

Gaya batang (tekan) maksimum = -16386,970 kg

Panjang = 1,5078 m = 150,78 cm

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

$$Cc = \frac{6440}{\sqrt{fy}} = \frac{6440}{\sqrt{2531}} = 128{,}009 > \frac{k.L}{r} = 50 \text{ , maka :} \qquad \textbf{(3.1.28)}$$

$$Fs = \frac{5}{3} + \frac{3}{8}\frac{kL/r}{Cc} - \frac{1(kL/r)^3}{8.Cc^3} = \frac{5}{3} + \frac{3}{8}\frac{.50}{128{,}009} - \frac{1}{8}\frac{50^3}{128{,}009^3} = 1{,}8057 \qquad \textbf{(3.1.30)}$$

$$Fa_{perlu} = \frac{fy}{Fs}\left(1 - 0{,}5\left(\frac{kL/r}{Cc}\right)^2\right) = \frac{2531}{1{,}806}\left(1 - 0{,}5\left(\frac{50}{128{,}009}\right)^2\right) = 1294{,}7538 \text{ kg/cm}^2 \textbf{(3.1.31)}$$

$$A_{perlu} = \frac{P}{Fa_{perlu}} = \frac{16386,970}{1294,533} = 12,658 \text{ cm}^2 \quad \textbf{(3.1.33)}$$

Di coba Profil **2L 70 x 70 x 7**

A total = 2 x 9,40 = 18,8 cm^2

r = 2,12 cm

e = 1,97 cm

Ix = Iy = 42,4 cm^4

ix = iy = 2,12 cm

$$X = e + \frac{1}{2} \cdot tp = 1,97 + \frac{1}{2} \cdot 1 = 2,47 \text{ cm}$$

Ix gab = 2 . 42,4 = 84,8 cm^4

Iy gab = Ix gab + 2 . A . x = 84,8 + 2 . 18,8 . 2,47 = 131,236 cm^4

$$\text{ix gab} = \sqrt{\frac{Ixgab}{2A}} = \sqrt{\frac{84,8}{2.9,4}} = 2,124 \text{ cm}$$

$$\text{iy gab} = \sqrt{\frac{Iygab}{2A}} = \sqrt{\frac{131,236}{2.9,40}} = 2,642 \text{ cm}$$

dipakai r = ix gab = 2,120 cm

Kontrol Local Buckling :

$$\frac{bf}{tf} \leq \frac{76}{\sqrt{fy}} \quad \textbf{(3.1.26)}$$

$$\frac{70}{7} \leq \frac{76}{\sqrt{36}}$$

10 $\leq$ 12,667 **(Ok!)**

Kontrol Beban :

$$\frac{kL}{r} = \frac{1.150,78}{2,120} \leq Cc = \frac{6440}{\sqrt{fy}} = \frac{6440}{\sqrt{2531}} \quad \textbf{(3.1.28)}$$

= 71,121 $\leq$ 128,009 (terjadi tekuk elastis)

$$Fs = \frac{5}{3} + \frac{3}{8}\frac{kL/r}{Cc} - \frac{1(kL/r)^3}{8.Cc^3} = \frac{5}{3} + \frac{3}{8}\frac{71{,}121}{128{,}009} - \frac{1}{8}\frac{71{,}121^3}{128{,}009^3} = 1{,}854$$

$$Fa_{ada} = \frac{2531}{1{,}854}\left(1 - 0{,}5\left(\frac{71{,}121}{128{,}009}\right)^2\right) = 1154{,}716 \text{ kg/cm}^2$$

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

$$= 1154{,}716\ .\ 18{,}8$$

$$= 21708{,}658 \text{ kg} \geq 16386{,}970 \text{ kg} \text{ .... (Ok!)}$$

a. Batang Diagonal (D_1 s/d D_{10})

(tabel 4.5 halaman 65 Gaya Batang Yang Terjadi Pada Rangka Atap)

Gaya batang (tekan) maksimum = -1995,290 kg

Panjang = 1,9418 m = 194,18 cm

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

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

$$Fs = \frac{5}{3} + \frac{3}{8}\frac{kL/r}{Cc} - \frac{1(kL/r)^3}{8.Cc^3} = \frac{5}{3} + \frac{3}{8}\frac{.50}{128{,}009} - \frac{1}{8}\frac{50^3}{128{,}009^3} = 1{,}8057$$

$$Fa_{perlu} = \frac{fy}{Fs}\left(1 - 0{,}5\left(\frac{kL/r}{Cc}\right)^2\right) = \frac{2531}{1{,}806}\left(1 - 0{,}5\left(\frac{50}{128{,}009}\right)^2\right) = 1294{,}7538 \text{ kg/cm}^2$$

$$A_{perlu} = \frac{P}{Fa_{perlu}} = \frac{1995{,}290}{1294{,}7538} = 1{,}541 \text{ cm}^2$$

Di coba Profil **2L 40 x 40 x 4**

A total = 2 x 3,08 = 6,160 cm^2

r = 1,210 cm   e = 1,120 cm

Ix = Iy = 4,48 cm^4 ix = iy = 1,210 cm

$x = e + \frac{1}{2} . tp = 1,120 + \frac{1}{2} .1 = 1,620$ cm

Ix gab = 2 . 4,48 = 8,96 cm^4

Iy gab = Ix gab + 2 . A . x = 8,96 + 2 . 6,160 . 8,96 = 18,939 cm^4

$$\text{ix gab} = \sqrt{\frac{Ixgab}{2A}} = \sqrt{\frac{8,96}{2.3,08}} = 1,206 \text{ cm}$$

$$\text{iy gab} = \sqrt{\frac{Iygab}{2A}} = \sqrt{\frac{18,939}{2.3,08}} = 1,753 \text{ cm}$$

dipakai r = ix gab = 1,206 cm

Kontrol Local Buckling :

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

$$\frac{40}{4} \leq \frac{76}{\sqrt{36}}$$

$10 \leq 12,667$ (Ok!)

Kontrol Beban :

$$\frac{kL}{r} = \frac{1.194,18}{1,206} \leq Cc = \frac{6440}{\sqrt{fy}} = \frac{6440}{\sqrt{2531}}$$

$= 161,002 \geq 128,009$

karena kl/r $\geq$ Cc maka :

$$Fa_{ada} = \frac{12}{23} . \frac{\pi^2 .E}{(Kl/r)^2} = \frac{12}{23} . \frac{\pi^2 .2100000}{161,002^2} = 417,505 \text{ kg/cm}^2$$

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

$= 417,505 . 6,160$

$= 2571,829$ kg $\geq 1995,290$ kg .... **(Ok!)**

2. Batang Tarik

a. Batang Bawah (B_1 s/d B_{12})

(tabel 4.5 halaman 65 Gaya Batang Yang Terjadi Pada Rangka Atap)

$P_{tarik\ maks}$ = 14660,236 kg fy = 2531 kg/cm^2

Panjang = 1,330 m Fu = 4077 kg/cm^2

Syarat batang tarik :

$$r_{min} = \frac{L}{240} = \frac{133}{240} = 0{,}554 \text{ cm} \qquad \textbf{(3.1.17)}$$

luas tampang perlu :

$$Ag1 = \frac{T}{0{,}6.Fy} = \frac{14660{,}236}{0{,}6.2531} = 9{,}654 \text{ cm}^2 \qquad \textbf{(3.1.18)}$$

$$Ag2 = \frac{T}{0{,}5.Fu.0{,}75} + \left(\frac{1"}{8} + \theta baut\right).tp.n = \frac{14660{,}236}{0{,}5.4077.0{,}75} + \left(\frac{1"}{8} + \frac{5"}{8}\right).(2{,}54).0{,}6.2$$

$$= 11{,}873 \text{ cm}^2 \qquad \textbf{(3.1.19)}$$

dicoba profil **2L 60 x 60 x 6**

Abruto = 2 x 6,91 cm = 13,820 cm^2

W = 5,42 kg/m

r = 1,82 cm ≥ r min = 0,554 cm

dipakai r = 1,82 cm

$$Alubang = \left(\frac{1"}{8} + \theta baut\right).\ tp\ .n \qquad \textbf{(3.1.20)}$$

$$= \left(\frac{1"}{8} + \frac{5"}{8}\right).(2{,}54).0.6.2$$

$$= 2{,}286 \text{ cm}^2$$

Anetto = Abruto – Alubang = 13,820 – 2,286 = 11,534 cm^2

• Untuk batang ada lubang

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

μ = 0,75 (jumlah baut 2 buah/baris)

$A_{efektif} = Anetto \times \mu$ **(3.1.22)**

$= 11{,}534 \times 0{,}75 = 8{,}651\ cm^2$

Kontrol Tegangan :

• Untuk batang tidak ada lubang

$$fa = \frac{T}{A_{profil}} \leq 0{,}6\,fy \qquad \textbf{(3.1.24)}$$

$$= \frac{14660{,}236}{13{,}820} = 1060{,}798\ kg/cm^2 \leq 0{,}6.2531 = 1518{,}6\ kg/cm^2 \quad \textbf{(Ok!)}$$

• Untuk batang ada lubang

$$fa = \frac{T}{A_{efektif}} \leq 0{,}5Fu \qquad \textbf{(3.1.25)}$$

$$= \frac{14660{,}236}{8{,}651} \leq 0{,}5.4077 = 1694{,}727\ kg/cm^2 \leq 2030{,}5\ kg/cm^2 \quad \textbf{(Ok!)}$$

b. Batang Vertikal (V_1 s/d V_{11})

(tabel 4.5 halaman 65 Gaya Batang Yang Terjadi Pada Rangka Atap)

$P_{tarik\ maks}$ = 9949,198 kg $\qquad$ fy = 2531 kg/cm^2

Panjang = 2,32904 m $\qquad$ Fu = 4077 kg/cm^2

Syarat batang tarik :

$$r_{min} = \frac{L}{240} = \frac{232{,}904}{240} = 0{,}970\ cm$$

luas tampang perlu :

$$Ag1 = \frac{T}{0{,}6.Fy} = \frac{9949{,}198}{0{,}6.2531} = 6{,}552\ cm^2$$

$$Ag2 = \frac{T}{0{,}5.Fu.0{,}75} + \left(\frac{1''}{8} + \theta baut\right).tp.n = \frac{9949{,}198}{0{,}5.4077.0{,}75} + \left(\frac{1''}{8} + \frac{5''}{8}\right).(2{,}54).0.5.2$$

$$= 8{,}411 \text{ cm}^2$$

dicoba profil **2L 50 x 50 x 5**

Abruto = 2 x 4,80 cm = 9,60 cm^2

W = 3,77 kg/m

r = 1,51 cm $\geq$ r min = 0,970 cm

dipakai r = 1,51 cm

$$\text{Alubang} = \left(\frac{1''}{8} + \theta baut\right).\ tp\ .n$$

$$= \left(\frac{1''}{8} + \frac{5''}{8}\right).(2{,}54).0{,}5.2$$

$$= 1{,}905 \text{ cm}^2$$

Anetto = Abruto – Alubang = 9,60 – 1,905 = 7,695 cm^2

• Untuk batang ada lubang

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

μ = 0,75 (jumlah baut 2 buah/baris)

$A_{efektif}$ = Anetto x μ

= 7,695 x 0,75 = 5,771 cm^2

Kontrol Tegangan :

• Untuk batang tidak ada lubang

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

$$= \frac{9949{,}198}{9{,}60} = 1036{,}375 \text{ kg/cm}^2 \leq 0{,}6.2531 = 1518{,}6 \text{ kg/cm}^2 \ \textbf{(Ok!)}$$

- Untuk batang ada lubang

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

$$= \frac{9949{,}198}{5{,}771} \leq 0{,}5.4077 \quad = 1723{,}924 \text{ kg/cm}^2 \leq 2030{,}5 \text{ kg/cm}^2 \quad \textbf{(Ok!)}$$

Kontrol berat kuda – kuda :

- Berat total kuda-kuda (Wtotal) = 618,881 kg
- Berat baut dan plat sambung = 20% x berat total kuda-kuda

= 0,2 x 618,881 = 123,776 kg

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

= 618,881 + 123,776 = 742,657 kg

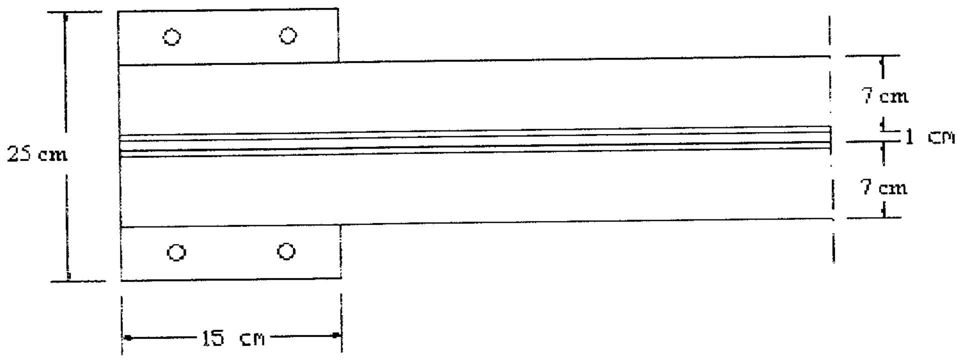
- Panjang bentang kuda-kuda (L) = 15 m

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

$$\frac{742{,}657}{15} \leq 75 \,{}^{kg}\!/_{m'}$$

49,510 kg/m' ≤ 75 kg/m' **(Ok!)**

4.1.7.1 Perencanaan Pelat Kuda – Kuda



Gambar 4.9 Pelat Kuda - Kuda

Beban P dimbil dari tabel 4.6 halaman 67 Reaksi Dukungan Yang Terjadi Pada Kuda - Kuda :

P_{maks} = 4858,918 kg , f'c = 25 Mpa = 250 kg/cm^2

$$A_{perlu} = \frac{P}{0{,}33f'c} = \frac{4858{,}918}{0{,}33.250} = 58{,}896 \text{ cm}^2$$

Dipakai ukuran pelat = 15 cm x 25 cm = 375 cm^2 > A_{perlu} = 58,896 cm^2

$$q = \frac{P}{BxL} = \frac{4858{,}918}{15x25} = 12{,}957 \text{ kg/cm}$$

$$x = \frac{25-(7+1+7)}{2} = 5 \text{ cm}$$

$$M = \tfrac{1}{2}.q.x^2 = \tfrac{1}{2}\,.\,12{,}957.\,5^2 = 161{,}964 \text{ kgcm}$$

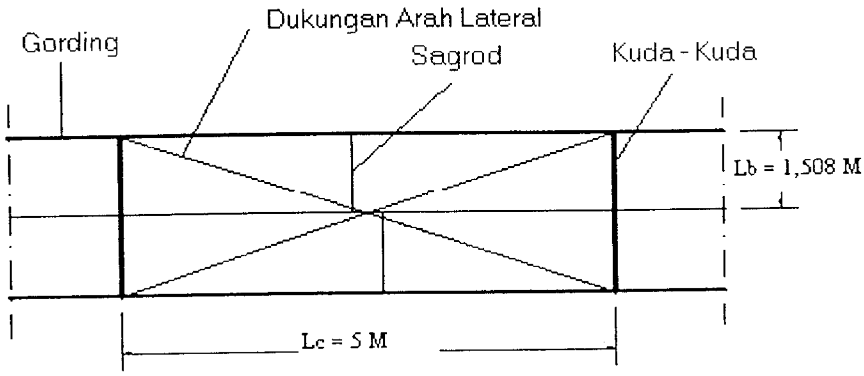
Syarat :

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

$$tp = \sqrt{\frac{10.M}{Fy}} = \sqrt{\frac{10.161{,}964}{2531}} = 0{,}800 \text{ cm} \approx 1 \text{ cm}$$

Pelat kuda – kuda berukuran : 15 x 25 x 1 cm^3

4.1.7.2 Perencanaan Dukungan Lateral



Gambar 4.10 Dukungan Arah Lateral

Diketahui :

Lb = jarak antar gording = 1,508 m

Lc = jarak antar kuda-kuda = 5 m

$L = \sqrt{Lb^2 + Lc^2} = \sqrt{1{,}508^2 + 5^2} = 5{,}2224 \text{ m} = 522{,}24 \text{ cm}$

Syarat :

L/refleks ≤ 300 , sehingga :

$$r_{min} \geq \frac{L}{300} = \frac{522{,}24}{300} = 1{,}741 \text{ cm} = 17{,}41 \text{ mm}$$

Keterangan :

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

Karena L = 5,2224 m, maka dukungan arah lateral dipakai baja tulangan diameter 19 mm > r_{min} = 17,41 mm

4.1.8 Perencanaan Sambungan

- Tebal pelat sambung = 1 cm, d_{baut} = 5/8" = 1,5875 cm
- Mutu Baja Profil :

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

  Kuat tarik (Fu) = 58 Ksi = 4077 kg/cm^2
- Mutu Baut A325X (Non Full Draat) :

  Tegangan tarik (Ft) = 44 Ksi = 3093 kg/cm^2

  Tegangan Geser (Fv) = 30 Ksi = 2109 kg/cm^2

Tinjauan Tegangan Geser 1 Baut :

$P_{geser} = \frac{1}{4} \,.\, \pi \,.\, D^2_{baut}$ x Fv x jumlah bidang geser (n) **(3.1.37)**

$= \frac{1}{4} \,.\, \pi \,.\, 1{,}5875^2 \,.\, 2109 \,.\, 2$

$= 5259{,}0850$ kg

Tinjauan Tegangan Tumpu 1 Baut :

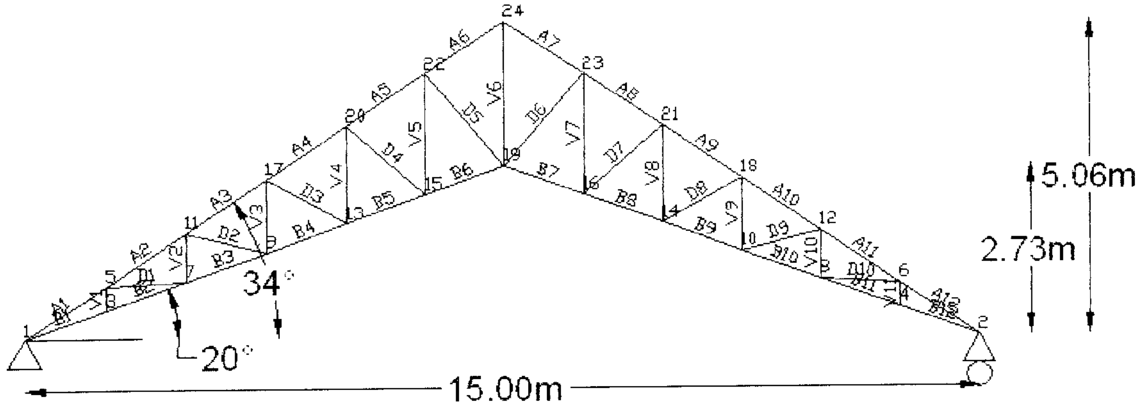
$P_{tumpu} = 1{,}2 \,.\, Fu \,.\, D_{baut} \,.\, t$. jumlah tumpuan (n) **(3.1.35)**

$= 1{,}2 \,.\, 4077 \,.\, 1{,}5875 \,.\, 1 \,.\, 1$

$= 7766{,}685$ kg

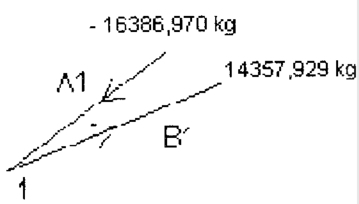
Jadi $P_{1\,baut}$ dipakai P_{tumpu} = 5259,0850 kg

$$\text{Jumlah Baut (N)} = \frac{P_{terjadi}}{P_{1baut}} \quad \textbf{(3.1.39)}$$



Gambar 4.11 Rangka Kuda – Kuda

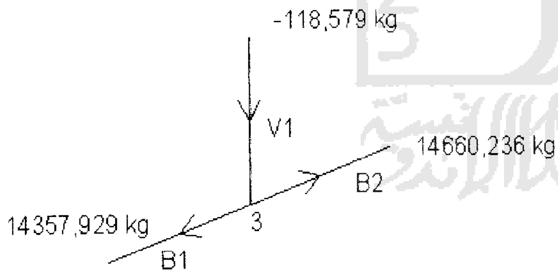
- Joint 1



$$n\ A1 = \frac{16386{,}970}{5259{,}0850} = 3{,}116 \sim 4 \text{ buah}$$

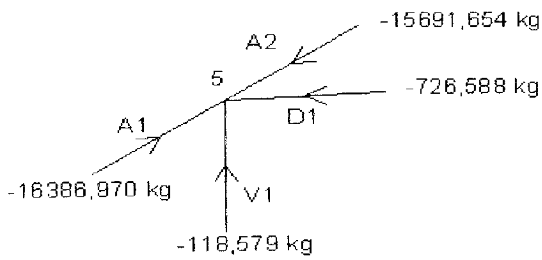
$$n\ B1 = \frac{14357{,}929}{5259{,}0850} = 2{,}73 \approx 4 buah$$

- Joint 3



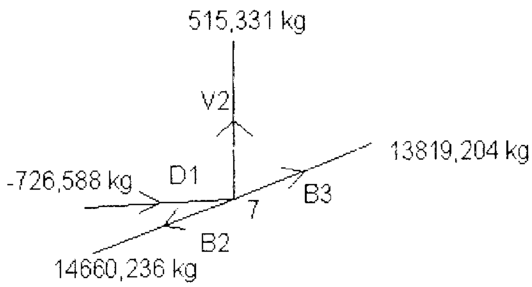
$$n\ V1 = \frac{118{,}579}{5259{,}0850} = 0{,}023 \approx 1 buah$$

- Joint 5



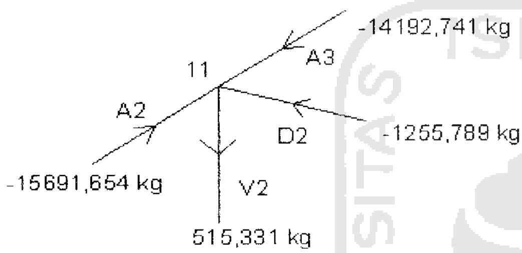
$$n\ D\ 1 = \frac{726{,}588}{5259{,}0850} = 0{,}138 \sim 2 \text{ buah}$$

- Joint 7



$$n\ V\ 2 = \frac{515{,}331}{5259{,}0850} = 0{,}098 \sim 2 \text{ buah}$$

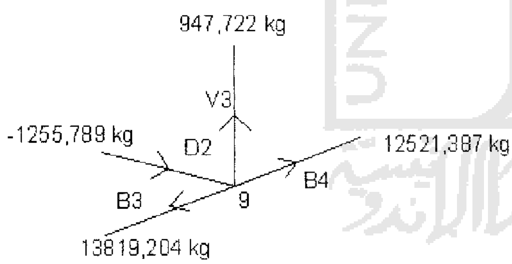
- Joint 11



$$n\ D\ 2 = \frac{1255{,}789}{5259{,}0850} = 0{,}239 \sim 2 \text{ buah}$$

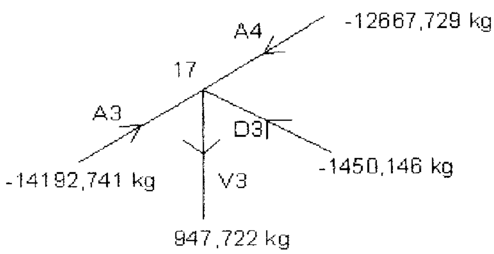
$$n\ V\ 2 = \frac{515{,}331}{5259{,}0850} = 0{,}098 \approx 2 buah$$

- Joint 9



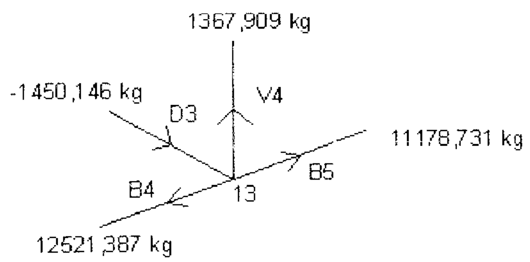
$$n\ V\ 3 = \frac{947{,}722}{5259{,}0850} = 0{,}180 \sim 2 \text{ buah}$$

- Joint 17



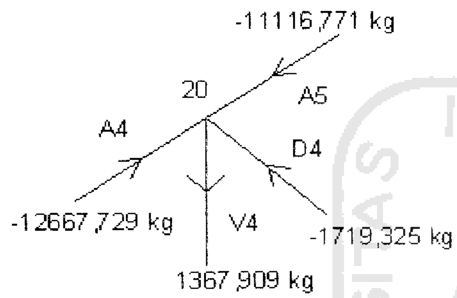
$$n\ D\ 3 = \frac{1450{,}146}{5259{,}0850} = 0{,}276 \approx 2 buah$$

- Joint 13



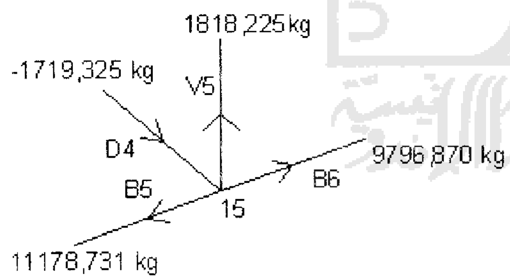
$$n\ V\ 4 = \frac{1367,909}{5259,0850} = 0,260 \approx 2buah$$

- Joint 20



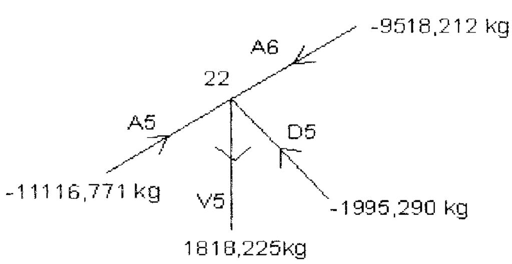
$$n\ D\ 4 = \frac{1719,325}{5259,0850} = 0,327 \approx 2buah$$

- Joint 15



$$n\ V\ 5 = \frac{1818,225}{5259,0850} = 0,346 \approx 2buah$$

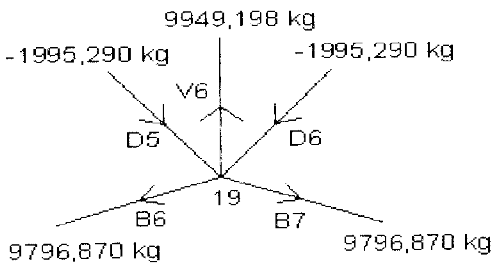
- Joint 22



$$n\ D\ 5 = \frac{1995,290}{5259,0850} = 0,379 \approx 2buah$$

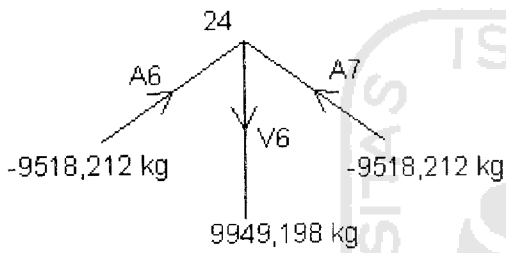
$$n\ V\ 5 = \frac{1818,225}{5259,0850} = 0,346 \approx 2buah$$

- Joint 19



$$n\,V\,6 = \frac{9949{,}198}{5259{,}0850} = 1{,}892 \sim 2 \text{ buah}$$

- Joint 24



$$n\,V\,6 = \frac{9949{,}198}{5259{,}0850} = 1{,}892 \sim 2 \text{ buah}$$

Tabel 4.10 Jumlah baut pada setengah bentang kuda – kuda

Nama Batang	Jumlah Baut (buah)
A1 S/D A6	4
B1 S/D B6	4
V1	1
V2 ; V3 ; V4 ; V5 ; V6	2
D1 ; D2 ; D3 ; D4 ; D5	2

4.2 Perencanaan Pelat

4.2.1 Perencanaan Pelat Lantai

a. Menentukan Tebal dan Tinggi Minimum Pelat Lantai (h)

Diperkirakan balok tepi pelat mempunyai lebar , b = 300 mm

maka : lnx = 5000 – 300 = 4700 mm

lny = 5000 – 300 = 4700 mm

perbandingan bentang bersih sisi panjang dan pendek :

$$\beta = \frac{\ln y}{\ln x} = \frac{4700}{4700} = 1{,}00$$

sehingga tebal pelat tidak boleh kurang dari :

$$h = \frac{\ln(0{,}8 + {}^{fy}/_{1500})}{36 + 9.\beta} = \frac{4700(0{,}8 + {}^{240}/_{1500})}{36 + 9.1{,}00} = 100{,}2667 \text{ mm} \quad \textbf{(3.2.2)}$$

tetapi tidak perlu lebih besar sama dengan dari :

$$h = \frac{\ln(0{,}8 + {}^{fy}/_{1500})}{36} = \frac{4700(0{,}8 + {}^{240}/_{1500})}{36} = 125{,}333 \text{ mm} \quad \textbf{(3.2.3)}$$

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. maka digunakan tebal pelat lantai 12 cm.

b. Pembebanan Pelat Lantai

- Beban mati pelat lantai :

1.	berat sendiri pelat	: 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 :

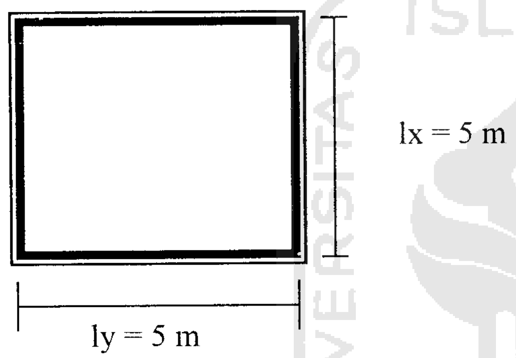
  Gedung ini berfungsi sebagai gedung pertemuan, sehingga beban hidupnya (qL) sebesar 400 kg/m^2 atau 4,0 KN/m^2 (PPIUG, 1983 tabel 3.1, halaman 17)

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

  qU = 1,2.qD + 1,6.qL = 1,2.4,51 + 1,6.4,0 = 11,812 KN/m^2

c. Perencanaan Pelat Lantai

Pelat dianggap terjepit elastis pada keempat sisinya.



$\frac{ly}{lx} = \frac{5}{5} = 1$, dihitung sebagai pelat dua arah.

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

Koef. Momen Pelat (C)	1,0
Mlx = -Mtx	36
Mly = -Mty	36

- Digunakan tulangan pokok Ø 10 mm
- Penutup beton (Pb) digunakan 20 mm

Tinggi manfaat tulangan pelat lantai :

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

  $= 120 - 20 - \frac{1}{2}.10$

  $= 95$ mm

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

$= 120 - 20 - 10 - \frac{1}{2}.10$

$= 85$ mm

Momen – momen yang bekerja pada pelat :

$$Mulx = - Mutx = 0{,}001 \ . \ qU \ . \ lx^2 \ . \ C \qquad \textbf{(3.2.4)}$$

$$= 0{,}001 \ .11{,}812. \ 5^2 \ . \ 36 = 10{,}6308 \text{ KNm}$$

$$Muly = - Muty = 0{,}001 \ . \ qU \ . \ lx^2 \ . \ C \qquad \textbf{(3.2.6)}$$

$$= 0{,}001 \ . \ 11{,}812 \ . \ 5^2 \ . \ 36 = 10{,}6308 \text{ KNm}$$

1) Perencanaan Tulangan lx dan tx

Mulx = - Mutx = 10,6308 KNm

$$Mu/\phi = 10{,}6308/0{,}8 = 13{,}2885 \text{ KNm}$$

Rasio Tulangan (ρ)

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25.0{,}85}{240}\left(\frac{600}{600+240}\right) = 0{,}053746 \qquad \textbf{(3.2.8)}$$

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{240} = 0{,}00583 \qquad \textbf{(3.2.10)}$$

$$\rho_{max} = 0{,}75 \ . \ \rho b = 0{,}75 \ . \ 0{,}053746 = 0{,}04032 \qquad \textbf{(3.2.9)}$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{240}{0{,}85.25} = 11{,}294 \qquad \textbf{(3.2.14)}$$

Koefisien ketahanan (Rn) :

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{13{,}2885.10^6}{1000.95^2} = 1{,}472 \text{ Mpa} \qquad \textbf{(3.2.13)}$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2m.Rn}{fy}}\right) = \frac{1}{11{,}294}\left(1-\sqrt{1-\frac{2.11{,}294.1{,}472}{240}}\right) \quad \textbf{(3.2.15)}$$

$= 0{,}00636 < \rho_{max} = 0{,}04032$

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

maka : $\rho_{pakai} = \rho_{ada} = 0{,}00636$

$As_p = \rho_{pakai} . b . d \geq 0{,}002 . b . h$ **(3.2.16)**

$= 0{,}00636 . 1000 . 95 \geq 0{,}002 . 1000 . 120$

$= 604{,}200 \text{ mm}^2 \geq 240 \text{ mm}^2$

digunakan tulangan pokok Ø 10 mm, sehingga :

$A_1Ø = ¼ . \pi . D^2 = ¼ . \pi . 10^2 = 78{,}54 \text{ mm}^2$

jarak tulangan (s) $= \dfrac{A_1\phi.b}{As_p}$ **(3.2.17)**

$= \dfrac{78{,}54.1000}{604{,}200}$

$= 129{,}990$ mm

dipakai s = 125 mm, maka Tulangan Pokok : **P10 – 125**

$As_{ada} = \dfrac{A_1\phi.b}{s} = \dfrac{78{,}54.1000}{125} = 628{,}320 \text{ mm}^2$ **(3.2.20)**

Kontrol Kapasitas Lentur Pelat (arah x) :

$a = \dfrac{As_{ada}.fy}{0{,}85.f'c.b} = \dfrac{628{,}320.240}{0{,}85.25.1000} = 7{,}096$ mm **(3.2.21)**

$Mn = As_{ada} . fy (d - {}^{a}\!/_{2}) \geq {}^{Mu}\!/_{\phi}$ **(3.2.22)**

$= 628{,}320. 240 (95 - {}^{7{,}096}\!/_{2}) / 10^6$

$= 14{,}326 \text{ KNm} \geq 13{,}2885$ KNm ....**OK !**

2) Perencanaan Tulangan ly dan ty

Muly = - Muty = 10,6308 KNm

$Mu/_{\phi} = 10{,}6308/_{0{,}8} = 13{,}2885$ 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) :

$$Rn = \frac{Mu/_{\phi}}{b.d^2} = \frac{13{,}2885.10^6}{1000.85^2} = 1{,}839 \text{ Mpa}$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2m.Rn}{fy}}\right) = \frac{1}{11{,}294}\left(1-\sqrt{1-\frac{2.11{,}294.1{,}839}{240}}\right)$$

$= 0{,}008026 < \rho_{max} = 0{,}04032$

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

maka : $\rho_{pakai} = \rho_{ada} = 0{,}008026$

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

$= 0{,}008026 \,.\, 1000 \,.\, 85$

$= 682{,}210 \text{ mm}^2 \geq 240 \text{ mm}^2$

digunakan tulangan pokok Ø 10 mm, sehingga :

$A_1Ø = ¼ \,.\, \pi \,.\, D^2 = ¼ \,.\, \pi \,.\, 10^2 = 78{,}54 \text{ mm}^2$

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

$= \dfrac{78{,}54.1000}{628{,}210}$

$= 115{,}126$ mm

dipakai s = 100 mm, maka Tulangan Pokok : **P10 – 110**

$As_{ada} = \dfrac{A_1\phi.b}{s} = \dfrac{78{,}54.1000}{110} = 714 \text{ mm}^2$

Kontrol Kapasitas Lentur Pelat (arah y) :

$a = \dfrac{As_{ada}.fy}{0{,}85.f'c.b} = \dfrac{714.240}{0{,}85.25.1000} = 8{,}064$ mm

$Mn = As_{ada} \,.\, fy\,(d - a/2) \geq Mu/\phi$

$= 785{,}4 \,.\, 240\,(85 - 8{,}064/2\,) / 10^6$

$= 15{,}262 \text{ KNm} \geq 13{,}2885$ KNm ....**OK !**

d. 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) $= \dfrac{A_1\phi.b}{As_{bagi}}$

$= \dfrac{50{,}265.1000}{240} = 209{,}440$ mm

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

4.2.2 Perencanaan Pelat Atap

a. Tebal dan Tinggi Minimum Pelat Atap

Diperkirakan balok tepi pelat mempunyai lebar , b = 200 mm

maka : lnx = 1000 – 200 = 800 mm

lny = 5000 – 200 = 4800 mm

perbandingan bentang bersih sisi panjang dan pendek :

$$\beta = \frac{\ln y}{\ln x} = \frac{4800}{800} = 6$$

sehingga tebal pelat tidak boleh kurang dari :

$$h = \frac{\ln(0{,}8 + {}^{fy}/_{1500})}{36 + 9.\beta} = \frac{4800(0{,}8 + {}^{240}/_{1500})}{36 + 9.6} = 51{,}2 \text{ mm}$$

tetapi tidak perlu lebih besar sama dengan dari :

$$h = \frac{\ln(0{,}8 + {}^{fy}/_{1500})}{36} = \frac{4800(0{,}8 + {}^{240}/_{1500})}{36} = 128{,}0 \text{ mm}$$

digunakan tebal pelat atap 10 cm.

b. Pembebanan Pelat Atap

- Beban mati pelat atap :

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

- Beban hidup pelat atap :

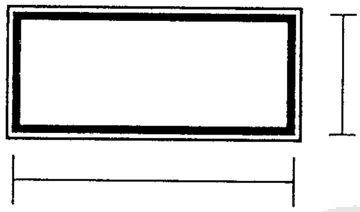
  Pada pelat atap terdapat beban hidup (qL) berupa beban pekerja atau air hujan sebesar 100 kg/m^2 atau 1,0 KN/m^2 (PPIUG, 1983 tabel 3.1 , halaman 17)

- 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

c. Perencanaan Pelat Atap

Pelat dianggap terjepit elastis pada keempat sisinya.



lx = 1,0 m

ly = 5 m

$\frac{ly}{lx} = \frac{5}{1,0} = 5,0$, dihitung sebagai pelat satu arah.

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

Nilai koefisien momen untuk ${}^{ly}/_{lx} = 5,0$ adalah sebagai berikut :

	Koefisien Momen Pelat (C)
Mlx = - Mtx	63
Mly	13
- Mty	38

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

Tinggi manfaat tulangan pelat lantai :

- Arah x : $dx = h - Pb - ½Ø_{tul.x}$

  $= 100 - 20 - ½.8$

  $= 76$ mm

- Arah y : $dy = h - Pb - \varnothing_{tul.x} - ½\varnothing_{tul.y}$

$= 100 - 20 - 8 - ½.8$

$= 68$ mm

Momen – momen yang bekerja pada pelat :

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

$= 0{,}001 \,.\, 5{,}272 \,.\, 1{,}0^2 \,.\, 63 = 0{,}332$ KNm

1) Perencanaan Tulangan lx dan tx

Mulx = - Mutx = 0,332 KNm

$$Mu/_{\phi} = 0{,}332/_{0{,}8} = 0{,}415 \text{ KNm}$$

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) :

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{0{,}415.10^6}{1000.76^2} = 0{,}0718 \text{ Mpa}$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2m.Rn}{fy}}\right) = \frac{1}{11{,}294}\left(1-\sqrt{1-\frac{2.11{,}294.0{,}0718}{240}}\right)$$

$$= 0{,}0002997 < \rho_{max} = 0{,}04032$$

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

$1{,}33\ \rho = 1{,}333\ .\ 0{,}0002997 = 0{,}0003995 < \rho_{\ min} = 0{,}00583$, maka :

$\rho_{\ pakai} = 1{,}33\ \rho = 0{,}0003995$

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

$\quad = 0{,}0003995\ .\ 1000\ .\ 76$

$\quad = 30{,}359\ mm^2 < 200\ mm^2$

dipakai $As_p = 200\ mm^2$

digunakan tulangan pokok Ø 8 mm, sehingga :

$A_1Ø \quad = \frac{1}{4}\ .\ \pi\ .\ D^2 = \frac{1}{4}\ .\ \pi\ .\ 8^2 = 50{,}265\ mm^2$

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

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

$$= 251{,}327\ mm$$

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**

$$As_{\ ada} = \frac{A_1\phi.b}{s} = \frac{50{,}265.1000}{200} = 251{,}327 mm^2$$

Kontrol Kapasitas Lentur Pelat (arah x) :

$$a \quad = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{251{,}327.240}{0{,}85.25.1000} = 2{,}839\ mm$$

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

$$= 251{,}327 \cdot 240 \left(76 - \frac{2{,}839}{2}\right) / 10^6$$

$$= 4{,}4986 \text{ KNm} \geq 1{,}33 \cdot 0{,}415 = 0{,}553 \text{ KNm}$$ ....**OK !**

d. Perencanaan Tulangan Susut Pelat Atap

$$As_{susut} = 0{,}002 \cdot b \cdot h$$

$$= 0{,}002 \cdot 1000 \cdot 100 = 200 \text{ mm}^2$$

digunakan tulangan susut Ø 8 mm, sehingga :

$$A_1Ø = \tfrac{1}{4} \cdot \pi \cdot D^2 = \tfrac{1}{4} \cdot \pi \cdot 8^2 = 50{,}2857 \text{ mm}^2$$

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

$$= \frac{50{,}2857 \cdot 1000}{200} = 251{,}4286 \text{ mm}$$

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

4.3 Perencanaan Struktur Portal Dengan Daktilitas Penuh

Pada perencanaan ulang Gedung Pimpinan Daerah Muhammadiyah Semarang 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 $= 0{,}18\ KN/m^2$ +

  $= 3{,}24\ KN/m^2$

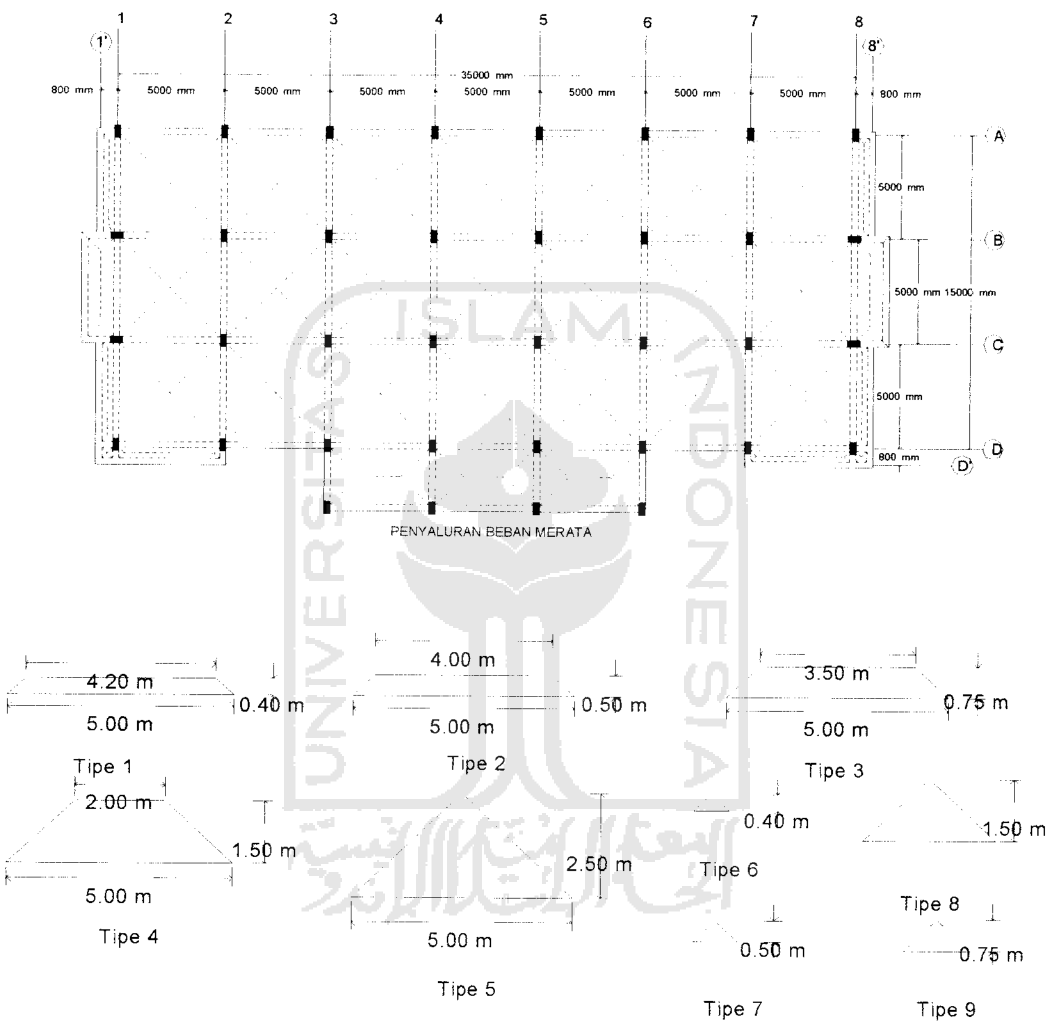
- Beban Dinding ½ bata $= 2{,}5\ KN/m^2$
- Beban tritisan $= 0{,}5\ KN/m^2$ (genteng dengan usuk dan reng)

2. Beban hidup

Beban hidup pelat lantai untuk ruang pertemuan $= 4{,}0\ KN/m^2$

Beban hidup pelat atap $= 1{,}0\ KN/m^2$

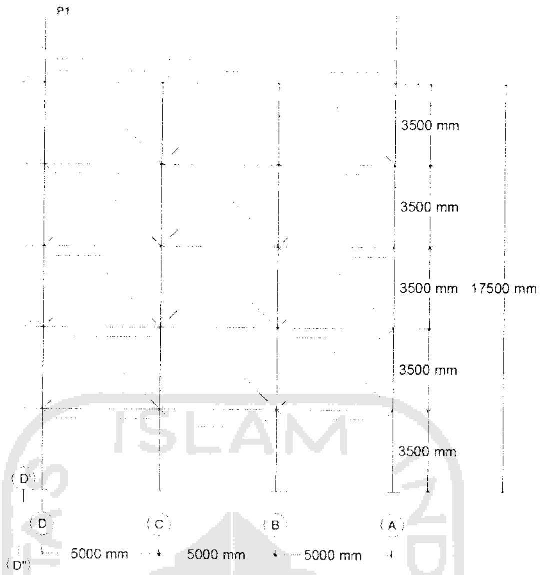
Pemodelan jenis beban pada SAP2000 :



Gambar 4.12 Tipe Pembebanan

4.3.1 Perhitungan Beban Akibat Gravitasi

A. Portal As 1 = As 8



Gambar 4.13 Rencana Beban Gravitasi Portal As 1 = As 8

1) Lantai 2, 3, 4 dan 5

Beban Mati :

- Beban merata
  - Bentang D-C = C-B = B-A

    Pelat lantai tipe 5 = 2,5 . 4,69 = 11,725 KN/m'(beban segitiga)
  - Bentang D-C = B-A

    Pelat lantai tipe 1 = 0,4 . 4,69 = 1,876 KN/m'(beban trapesium)
  - Bentang C-B

    Dinding = 2,5 . 3,5 = 8,75 KN/m'
  - Bentang D'-D

    Pelat lantai tipe 6 = 0,4 . 4,69 . 2 = 3,752 KN/m'(beban segitiga)

Beban Hidup :

- Beban merata
  - Bentang D-C = C-B = B-A

    Pelat lantai tipe 5 = 2,5 . 4,0 = 10 KN/m'(beban segitiga)
  - Bentang D-C = B-A

    Pelat lantai tipe 1 = 0,4 . 4,0 = 1,60 KN/m'(beban trapesium)

- Bentang D'-D

  Pelat lantai tipe 6 = 0,4 . 4,0 . 2 = 3,20 KN/m'(beban segitiga)

2) Lantai 2

Beban Mati :

- Beban merata
  - Bentang C-B

    Pelat lantai tipe 3 = 0,75 . 3,24 = 2,430 KN/m'(beban trapesium)

Beban Hidup :

- Beban merata
  - Bentang C-B

    Pelat lantai tipe 3 = 0,75 . 1 = 0,75 KN/m'(beban trapesium)

3) Pelat Atap

Beban Mati :

- Beban merata
  - Bentang D''- D = A' -A

    Pelat lantai tipe 7 = 0,5 . 3,24 . 2 = 3,24 KN/m'(beban segitiga)
  - Bentang D-C = C-B = B-A

    Pelat lantai tipe 7 = 0,5 . 3,24 = 1,62 KN/m'(beban segitiga)
- P1 beban mati diakibatkan dari reaksi kuda-kuda

  P1 = 3923,639 Kg = 39,23639 KN

Beban Hidup :

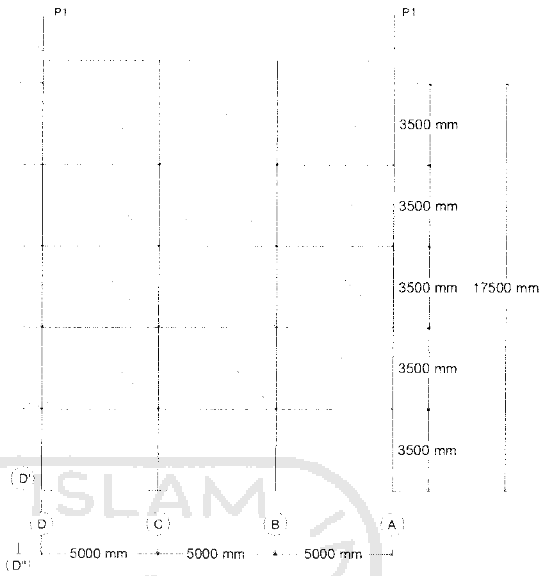
- Beban merata
  - Bentang D''- D = A' -A

    Pelat lantai tipe 7 = 0,5 . 1 . 2 = 1 KN/m'(beban segitiga)
  - Bentang D-C = C-B = B-A

    Pelat lantai tipe 7 = 0,5 . 1 = 0,5 KN/m'(beban segitiga)
- P1 beban hidup diakibatkan dari reaksi kuda-kuda

  P1 = 904,663 Kg = 9,046 KN

B. Portal As 2 = As 7



Gambar 4.14 Rencana Beban Gravitasi Portal As 2 s/d As 7

1) Lantai 2 , 3 , 4 dan 5

Beban Mati :

- Beban merata
  - Bentang D-C = C-B = B-A

    Pelat lantai tipe 5 = 2,5 . 4,69 . 2 = 23,45 KN/m'(beban segitiga)
  - Bentang C-B

    Dinding = 2,5 . 3,5 = 8,75 KN/m'
  - Bentang D'-D

    Pelat lantai tipe 6 = 0,4 . 4,69 = 1,876 KN/m'(beban segitiga)

    Dinding = 2,5 . 3,5 = 8,75 KN/m'

Beban Hidup :

- Beban merata
  - Bentang D-C = C-B = B-A

    Pelat lantai tipe 5 = 2,5 . 4,0 . 2 = 20 KN/m'(beban segitiga)
  - Bentang D'-D

    Pelat lantai tipe 6 = 0,4 . 4,0 = 1,60 KN/m'(beban segitiga)

2) Pelat Atap

Beban Mati :

- Beban merata
  - Bentang D''-D = A'-A

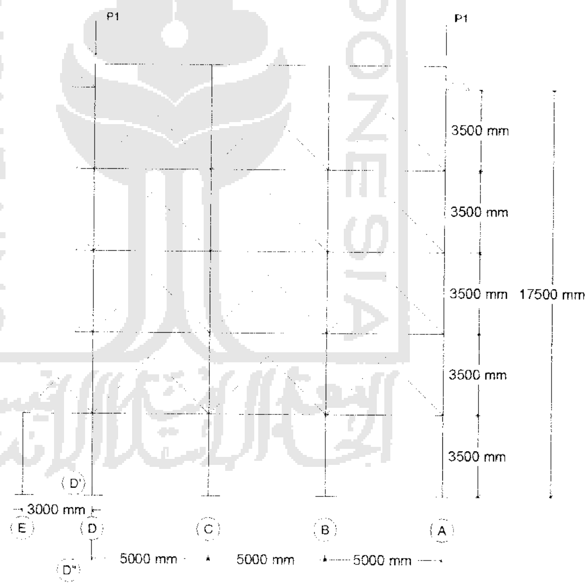
    Pelat lantai tipe 7 = 0,5 . 3,24 . 2 = 3,24 KN/m'(beban segitiga)
- P1 = 3923,639 Kg = 39,23639 KN

Beban Hidup :

- Beban merata
  - Bentang D''-D = A'-A

    Pelat lantai tipe 7 = 0,5 . 1 . 2 = 1 KN/m'(beban segitiga)
- P1 = 904,6632 Kg = 9,046632 KN

C. Portal As 3 = As 6



Gambar 4.15 Rencana Beban Gravitasi Portal As 3 = As 6

1) Lantai 2 , 3 , 4 dan 5

Beban Mati :

- Beban merata
  - Bentang D-C = B-A

    Pelat lantai tipe 5 = 2,5 . 4,69 . 2 = 23,45 KN/m'(beban segitiga)

    Dinding = 2,5 . 3,5 = 8,75 KN/m'

- Bentang C-B

  Pelat lantai tipe 5 = 2,5 . 4,69 . 2 = 23,45 KN/m'(beban segitiga)

Beban Hidup :

- Beban merata
  - Bentang D-C = C-B = B-A

    Pelat lantai tipe 5 = 2,5 . 4,0 . 2 = 20 KN/m'(beban segitiga)

2) Lantai 2

Beban Mati :

- Beban merata
  - Bentang E-D

    Pelat lantai tipe 8 = 1,5 . 3,24 = 4,86 KN/m'(beban segitiga)

Beban Hidup :

- Beban merata
  - Bentang E-D

    Pelat lantai tipe 8 = 1,5 . 1 = 1,5 KN/m'(beban segitiga)

3) Pelat Atap

Beban Mati :

- Beban merata
  - Bentang D''-D = A'-A

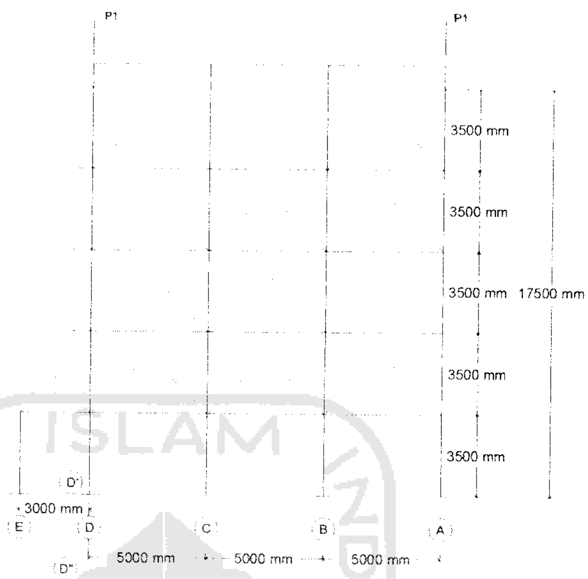
    Pelat lantai tipe 7 = 0,5 . 3,24 . 2 = 3,24 KN/m'(beban segitiga)
- P1 = 3923,639 Kg = 39,23639 KN

Beban Hidup :

- Beban merata
  - Bentang D''-D = A'-A

    Pelat lantai tipe 7 = 0,5 . 1 . 2 = 1 KN/m'(beban segitiga)
- P1 = 904,6632 Kg = 9,046632 KN

D. Portal As 4 = As 5



Gambar 4.16 Rencana Beban Gravitasi Portal As 4 = As 5

1) Lantai 2 , 3 , 4 dan 5

Beban Mati :

- Beban merata
  - Bentang B-A

    Pelat lantai tipe 5 = 2,5 . 4,69 . 2 = 23,45 KN/m'(beban segitiga)

    Dinding = 2,5 . 3,5 = 8,75 KN/m'
  - Bentang D-C = C-B

    Pelat lantai tipe 5 = 2,5 . 4,69 . 2 = 23,45 KN/m'(beban segitiga)

Beban Hidup :

- Beban merata
  - Bentang D-C = C-B = B-A

    Pelat lantai tipe 5 = 2,5 . 4,0 . 2 = 20 KN/m'(beban segitiga)

2) Lantai 2

Beban Mati :

- Beban merata
  - Bentang E-D

    Pelat lantai tipe 8 = 1,5 . 3,24 . 2 = 9,72 KN/m'(beban segitiga)

Beban Hidup :

- Beban merata
  - Bentang E-D

    Pelat lantai tipe 8 = 1,5 . 1 .2 = 3,0 KN/m'(beban segitiga)

3) Pelat Atap

Beban Mati :

- Beban merata
  - Bentang D''-D = A'-A

    Pelat lantai tipe 7 = 0,5 . 3,24 . 2 = 3,24 KN/m'(beban segitiga)
- P1 = 3923,639 Kg = 39,23639 KN

Beban Hidup :

- Beban merata
  - Bentang D''-D = A'-A

    Pelat lantai tipe 7 = 0,5 . 1 . 2 = 1 KN/m'(beban segitiga)
- P1 = 904,6632 Kg = 9,046632 KN

E. **Portal As D**

Gambar 4.17 Rencana Beban Gravitasi Portal As D

1) Lantai 2 , 3 , 4 dan 5

Beban Mati :

- Beban merata
  - Bentang 2-3 = 3-4 = 4-5 = 5-6 = 6-7

    Pelat lantai tipe 5 = 2,5 . 4,69 = 11,725 KN/m'(beban segitiga)

    Dinding = 2,5 . 3,5 = 8,75 KN/m'
  - Bentang 1-2 = 7-8

    Pelat lantai tipe 5 = 2,5 . 4,69 = 11,725 KN/m'(beban segitiga)

    Pelat lantai tipe 1 = 0,4 . 4,69 = 1,876 KN/m'(beban trapesium)
  - Bentang 1'-1 = 8-8'

    Pelat lantai tipe 6 = 0,4 . 4,69 . 2 = 3,752 KN/m'(beban segitiga)

Beban Hidup :

- Beban merata
  - Bentang 2-3 = 3-4 = 4-5 = 5-6 = 6-7

    Pelat lantai tipe 5 = 2,5 . 4,0 = 10 KN/m'(beban segitiga)
  - Bentang 1-2 = 7-8

    Pelat lantai tipe 5 = 2,5 . 4,0 = 10 KN/m'(beban segitiga)

    Pelat lantai tipe 1 = 0,4 . 4,0 = 1,6 KN/m'(beban trapesium)

- Bentang 1'-1 = 8-8'

  Pelat lantai tipe 6 = 0,4 . 1,0 . 2 = 0,8 KN/m'(beban segitiga)

2) Lantai 2

Beban Mati :

- Beban merata
  - Bentang 3-4 = 4-5 = 5-6

    Pelat lantai tipe 4 = 1,5 . 3,24 = 4,86 KN/m'(beban trapesium)

Beban Hidup :

- Beban merata
  - Bentang 3-4 = 4-5 = 5-6

    Pelat lantai tipe 4 = 1,5 . 1 = 1,5 KN/m'(beban trapesium)

3) Pelat Atap

Beban Mati :

- Beban merata
  - Bentang 1-2 = 2-3 = 3-4 = 4-5 = 5-6 = 6-7 = 7-8

    Pelat lantai tipe 2 = 0,5 . 3,24 = 1,62 KN/m'(beban trapesium)
  - Bentang 1''-1 = 8-8''

    Pelat lantai tipe 7 = 0,5 . 3,24 . 2 = 3,24 KN/m'(beban segitiga)
- P1 = 3923,639 Kg = 39,23639 KN

Beban Hidup :

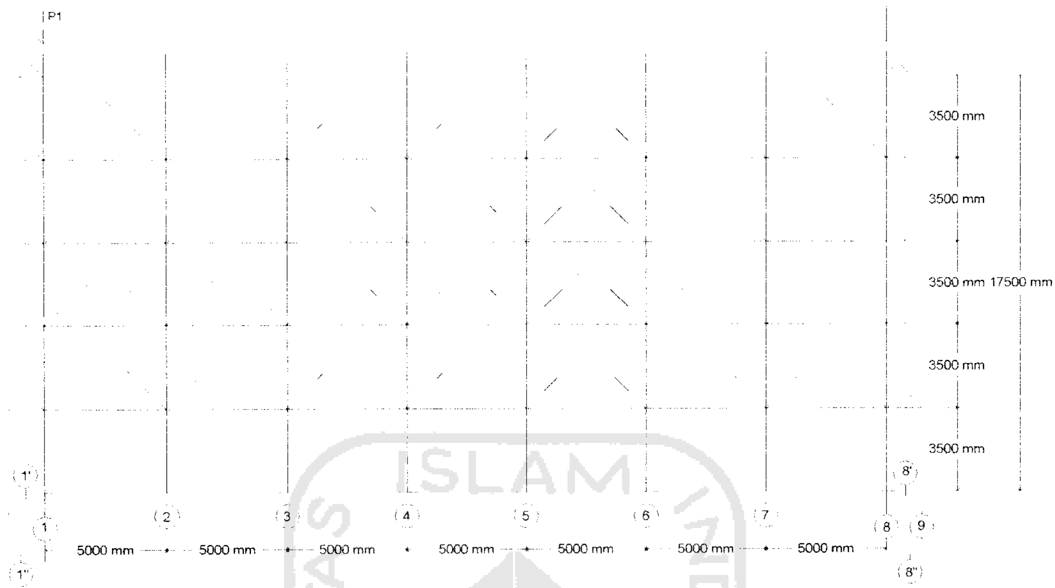
- Beban merata
  - Bentang 1-2 = 2-3 = 3-4 = 4-5 = 5-6 = 6-7 = 7-8

    Pelat lantai tipe 2 = 0,5 . 1 = 0,5 KN/m'(beban trapesium)
  - Bentang 1''-1 = 8-8''

    Pelat lantai tipe 7 = 0,5 . 1 . 2 = 1 KN/m'(beban segitiga)

- P1 = 904,6632 Kg = 9,046632 KN

F. Portal As B = As C



Gambar 4.18 Rencana Beban Grafitasi As B = As C

1) Lantai 2 , 3 , 4 dan 5

Beban Mati :

- Beban merata
  - Bentang 1-2 = 2-3 = 6-7 = 7-8

    Pelat lantai tipe 5 = 2,5 . 4,69 . 2 = 23,45 KN/m'(beban segitiga)

    Dinding = 2,5 . 3,5 = 8,75 KN/m'
  - Bentang 3-4 = 4-5 = 5-6

    Pelat lantai tipe 5 = 2,5 . 4,69 . 2 = 23,45 KN/m'(beban segitiga)
  - Bentang 1'-1 = 8-8'

    Pelat lantai tipe 6 = 0,4 . 4,69 = 1,876 KN/m'(beban segitiga)

    Dinding = 2,5 . 3,5 = 8,75 KN/m'

Beban Hidup :

- Beban merata
  - Bentang 1-2 = 2-3 = 3-4 = 4-5 = 5-6 = 6-7 = 7-8

    Pelat lantai tipe 5 = 2,5 . 4,0 .2 = 20 KN/m'(beban segitiga)
  - Bentang 1'-1 = 8-8'

    Pelat lantai tipe 6 = 0,4 . 4,0 = 1,6 KN/m'(beban segitiga)

2) Lantai 2

Beban Mati :

- Beban merata
  - Bentang 0-1 = 8-9

    Pelat lantai tipe 8 = 1,5 . 3,24 = 4,86 KN/m'(beban segitiga)

Beban Hidup :

- Beban merata
  - Bentang 0-1 = 8-9

    Pelat lantai tipe 8 = 1,5 . 1 = 1,5 KN/m'(beban segitiga)

3) Pelat Atap

Beban Mati :

- Beban merata
  - Bentang 1-1'' = 8-8''

    Pelat lantai tipe 7 = 0,5 . 3,24 . 2 = 3,24 KN/m'(beban segitiga)
- P1 = 3923,639 Kg = 39,23639 KN

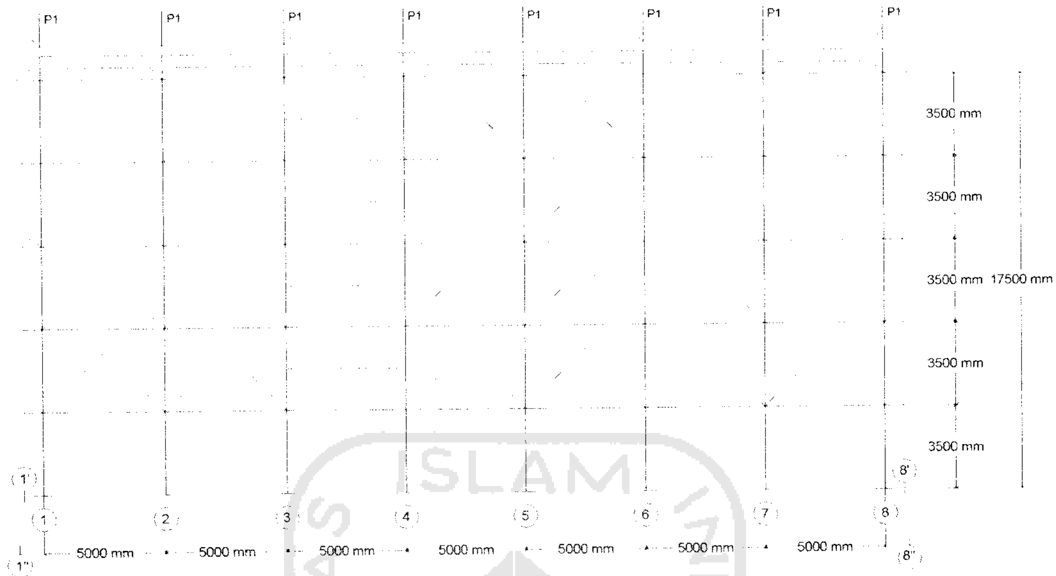
Beban Hidup :

- Beban merata
  - Bentang 1''-1 = 8-8''

    Pelat lantai tipe 7 = 0,5 . 1 . 2 = 1 KN/m'(beban segitiga)

- P1 = 904,6632 Kg = 9,046632 KN

G. Portal As A



Gambar 4.19 Rencana Beban Grafitasi As A

1) Lantai 2 , 3 , 4 dan 5

Beban Mati :

- Beban merata
  - Bentang 1-2 = 2-3 = 4-5 = 5-6 = 6-7 = 7-8

    Pelat lantai tipe 5 = 2,5 . 4,69 = 11,725 KN/m'(beban segitiga)

    Dinding = 2,5 . 3,5 = 8,75 KN/m'
  - Bentang 3-4

    Dinding = 2,5 . 3,5 = 8,75 KN/m'
  - Bentang 1'-1 = 8-8'

    Pelat lantai tipe 6 = 0,4 . 4,69 = 1,876 KN/m'(beban segitiga)

    Dinding = 2,5 . 3,5 = 8,75 KN/m'

Beban Hidup :

- Beban merata
  - Bentang 1-2 = 2-3 = 4-5 = 5-6 = 6-7 = 7-8

    Pelat lantai tipe 5 = 2,5 . 4,0 = 10 KN/m'(beban segitiga)
  - Bentang 1'-1 = 8-8'

    Pelat lantai tipe 6 = 0,4 . 4,0 = 1,6 KN/m'(beban segitiga)

2) Pelat Atap

Beban Mati :

- Beban merata
  - Bentang 1-2 = 2-3 = 3-4 = 4-5 = 5-6 = 6-7 = 7-8

    Pelat lantai tipe 2 = 0,5 . 3,24 = 1,62 KN/m'(beban trapesium)
  - Bentang 1''-1 = 8-8''

    Pelat lantai tipe 7 = 0,5 . 3,24 . 2 = 3,24 KN/m'(beban segitiga)
- P1 = 3923,639 Kg = 39,23639 KN

Beban Hidup :

- Beban merata
  - Bentang 1-2 = 2-3 = 3-4 = 4-5 = 5-6 = 6-7 = 7-8

    Pelat lantai tipe 2 = 0,5 . 1 = 0,5 KN/m'(beban trapesium)
  - Bentang 1''-1 = 8-8''

    Pelat lantai tipe 7 = 0,5 . 1 . 2 = 1 KN/m'(beban segitiga)
- P1 = 904,6632 Kg = 9,046632 KN

4.3.2 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 pertemuan, maka menurut Peraturan Pembebanan Indonesia 1983 (Tabel 3.3) untuk perencanaan beban gempa, beban hidup direduksi sebesar 0,5.

4.3.2.1. Berat total bangunan

1) Lantai 2

- Beban mati :

Pelat lantai	= 4,69 x(35x15+4x4+7,5x2+15x3+4,64x2)	= 2862,213 KN
Balok BA1	= (0,3 x (0,6 – 0,12) x 317,8) x 24	= 1098,317 KN
Kolom KA1	= (0,45 x 0,6) x (3,5x 32) x 24	= 725,760 KN
Dinding	= 3,5 x 195 x 2,5	= 1706,25 KN +
	WD	= 6392,540 KN

- Beban hidup :

  Beban Hidup pelat lantai (WL) = 0,5 x 4,0 x (525+16+15+45) = 1202 KN

  Wt_2 = 6392,540 + 1202 = 7594,540 KN

2) Lantai 3

- Beban mati :

Pelat lantai	= 4,69 x (35x15+4x4+4,64x2)		= 2580,813 KN
Balok BA1	= (0,3 x (0,6 – 0,12) x 317,8) x 24		= 1098,317 KN
Kolom KA1	= (0,45 x 0,6) x (3,5x 32) x 24		= 725,760 KN
Dinding	= 3,5 x 195 x 2,5		= 1706,25 KN +
		WD	= 6111,140 KN

- Beban hidup :

Beban Hidup pelat lantai (WL)	= 0,5 x 4 x 35 x 15		= 1050 KN
	= 0,5 x 4 x 4 x 4		= 32 KN
	= 0,5 x 4 x 4,64 x 2		= 18,56 KN +
		WL	= 1100,56 KN

Wt_3 = 6111,140 + 1100,56 = 7211,700 KN

3) Lantai 4

- Beban mati :

Pelat lantai	= 4,69 x (35x15+4x4+4,64x2)		= 2580,813 KN
Balok BA1	= (0,3 x (0,6 – 0,12) x 317,8) x 24		= 1098,317 KN
Kolom KA1	= (0,45 x 0,6) x (3,5x 32) x 24		= 725,760 KN
Dinding	= 3,5 x 195 x 2,5		= 1706,25 KN +
		WD	= 6111,140 KN

- Beban hidup :

Beban Hidup pelat lantai (WL)	= 0,5 x 4 x 35 x 15		= 1050 KN
	= 0,5 x 4 x 4 x 4		= 32 KN
	= 0,5 x 4 x 4,64 x 2		= 18,56 KN +
		WL	= 1100,56 KN

Wt_4 = 6111,140 + 1100,56 = 7211,700 KN

4) Lantai 5

- Beban mati :

Pelat lantai = 4,69 x (35x15+4x4+4,64x2) = 2580,813 KN

Balok BA1 = (0,3 x (0,6 – 0,12) x 317,8) x 24 = 1098,317 KN

Kolom KA1 = (0,45 x 0,6) x (3,5x 32) x 24 = 725,760 KN

Dinding = 3,5 x 100 x 2,5 = 875 KN +

WD = 5279,890 KN

- Beban hidup :

Beban Hidup pelat lantai (WL) = 0,5 x 4 x 35 x 15 = 1050 KN

= 0,5 x 4 x 4 x 4 = 32 KN

= 0,5 x 4 x 4,64 x 2 = 18,56 KN +

WL = 1100,56 KN

Wt_5 = 5279,890 + 1100,56 = 6380,450 KN

5) Pelat atap

- Beban mati :

Berat atap = (525/cos 32) x 0,5 = 309,534 KN

Berat kuda – kuda = 8 x 6,1482 = 49,1856 KN

Pelat atap = 100 x 3,24 = 324 KN

Balok = (0,3 x (0,45-0,1) x 232) x 24 = 584,64 KN

Kolom = (0,45 x 0,6) x (1 x 32) x 24 = 207,360 KN

Dinding = (1 x 100)x 2,5 = 250 KN+

WD = 1724,720 KN

- Beban hidup :

Beban Hidup pelat atap (WL) = 0,5 x 100 x 1,0 = 50 KN

Beban Hidup atap (WL) = 0,5 x (525/cos 32) x 1,0 = 309,534 KN +

WL = 359,534 KN

Wt_6 = 1724,720 + 359,534 = 2084,254 KN

$W_{total} = Wt_2 + Wt_3 + Wt_4 + Wt_5 + Wt_6$

= 7594,540 + 7211,700 + 7211,700 + 6380,450 + 2084,254

= 30482,644 KN

4.3.2.2. Waktu Getar Bangunan (T)

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

$$T = 0{,}06 \,.\, H^{3/4} = 0{,}06 \,.\, 17{,}5^{3/4} = 0{,}5134 \text{ dt} \qquad \textbf{(3.5.3)}$$

4.3.2.3. Koefisien Gempa Dasar (C)

Pada Redisain ini bangunan berada dalam wilayah gempa 4 pada kondisi tanah lunak. Waktu getar struktur (T) = 0,5134 dt, maka berdasarkan Respon spectrum wilayah 4 didapatkan koefisien gempa dasar (C) = 0,05

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

Berdasarkan fungsi 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.

4.3.2.5.Gaya Geser Horizontal Akibat Gempa (V)

Gaya geser horizontal akibat gempa yang bekerja dapat dihitung dengan :

$$V = C \,.\, I \,.\, K \,.\, Wt = 0{,}05 \,.\, 1{,}0 \,.\, 1{,}0 \,.\, 30482{,}644 = 1524{,}132 \text{ KN} \qquad \textbf{(3.5.1)}$$

4.3.2.6 Distribusi gaya horizontal total akibat gempa ke sepanjang tinggi gedung

1) Arah x

$$H/B = \frac{17{,}5}{35} = 0{,}5 < 3 \text{, maka :}$$

$$F_i x = \frac{Wi.hi}{\sum Wi.hi}.V \qquad \textbf{(3.5.2)}$$

2) Arah y

$$H/B = \frac{17{,}5}{15} = 1{,}167 < 3 \text{, maka :}$$

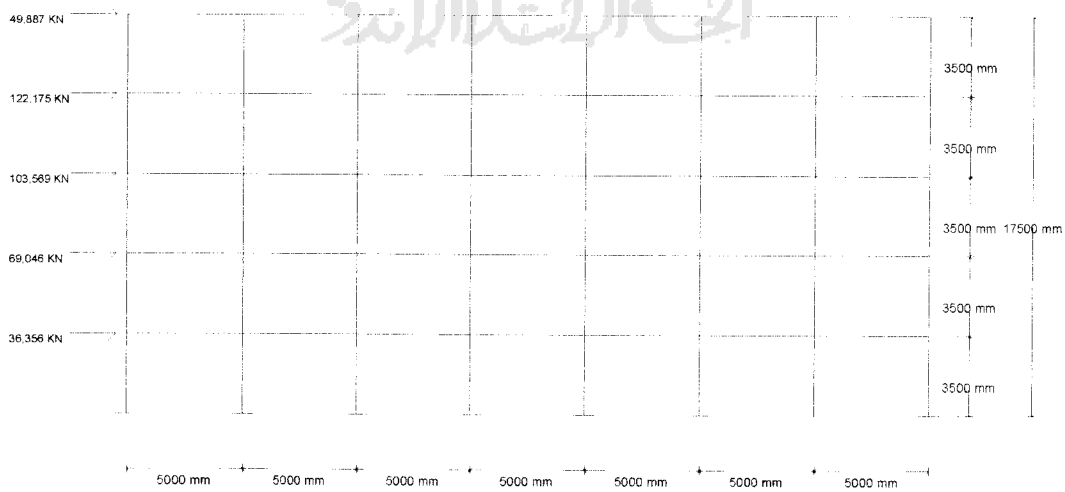
$$F_i y = \frac{Wi.hi}{\sum Wi.hi}.V \qquad \textbf{(3.5.2)}$$

Tingkat	Hi (m)	Wi (KN)	V (KN)	Wi.Hi (KNm)	Fi x,y (KN)
Pelat Atap	17.5	2084	1524.13	36474.445	199.550
Lantai 5	14	6380	1524.13	89326.3	488.700
Lantai 4	10.5	7212	1524.13	75722.85	414.276
Lantai 3	7	7212	1524.13	50481.9	276.184
Lantai 2	3.5	7595	1524.13	26580.89	145.423
Jumlah				278586.39	1524.132

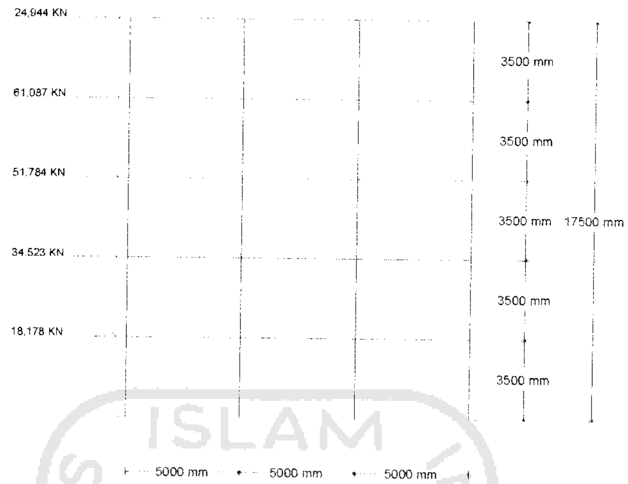
Tabel 4.8 Distribusi Gaya Geser Horizontal total akibat gempa arah x dan arah y

Tingkat	Arah Y 1/8.Fiy (KN)	Arah X 1/4.Fix (KN)
Pelat Atap	24.944	49.887
Lantai 5	61.087	122.175
Lantai 4	51.784	103.569
Lantai 3	34.523	69.046
Lantai 2	18.178	36.356

Tabel 4.9 Distribusi Gaya Geser Horizontal untuk tiap portal arah x dan arah y



Gambar 4.20 Distribusi Gaya Geser Horizontal Arah x



Gambar 4.21 Distribusi Gaya Geser Horizontal Arah y

4.4 Disain Balok

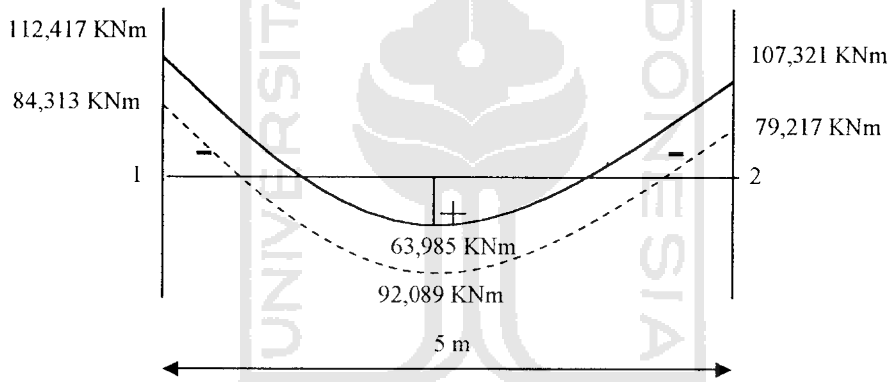
4.4.1 Disain Tulangan Lentur Balok

4.4.1.1 Momen Rencana Balok

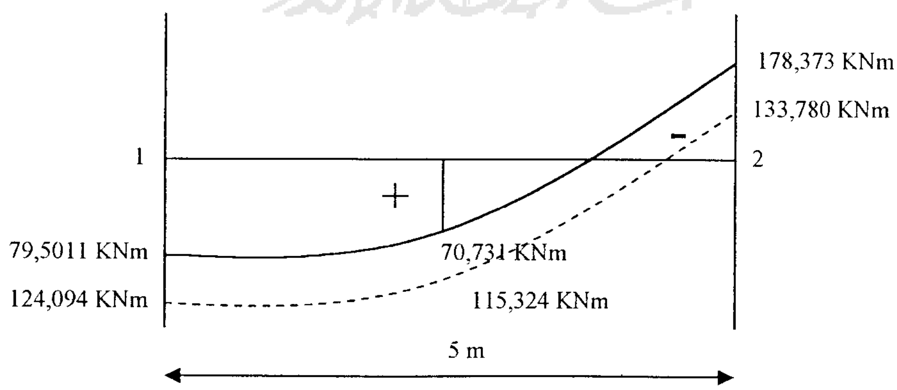
Momen rencana balok diambil yang terbesar dari hasil kombinasi beban sebagai berikut : (tabel 4.17 lampiran II – 17 Momen Rencana Balok arah X)

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

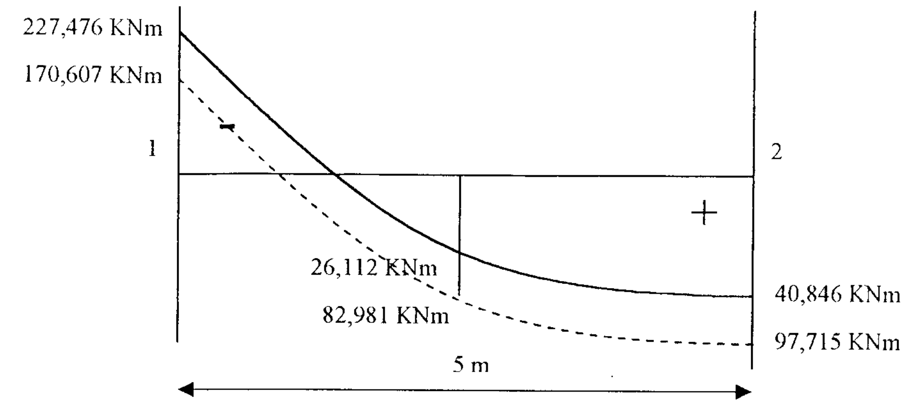
Berikut diberikan contoh perhitungan balok B1 (As B = C , bentang 1 – 2)



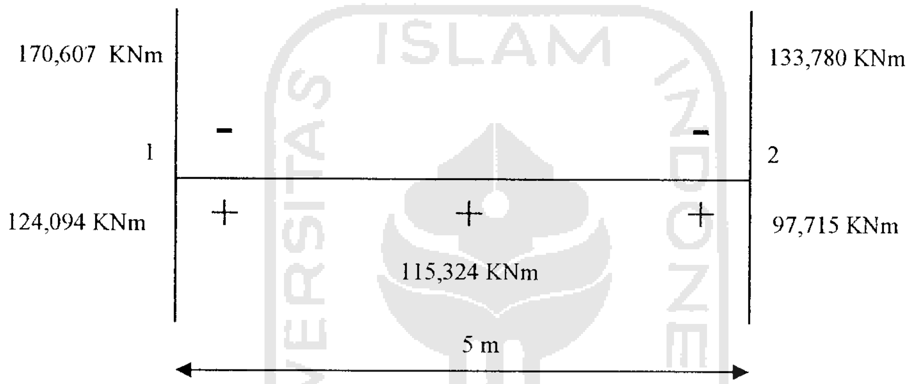
Gambar 4.22 Momen akibat kombinasi beban gravitasi (1,2 MD + 1,6 ML)



Gambar 4.23 Momen akibat kombinasi beban gempa kiri (0,9 MD ± ME dan 1,05 (MD + 0,9 ML ± ME)



Gambar 4.24 Momen akibat kombinasi beban gempa kanan (0,9 MD ± ME dan 1,05 (MD + 0,9 ML ± ME)



Gambar 4.25 Momen rencana terdistribusi

4.4.1.2 Tulangan Tumpuan 1

Dipakai dimensi rencana 300/600

fc' = 25 Mpa fy = 400 Mpa

Mu = 227,4757 KNm

Momen tumpuan didistribusikan ke momen lapangan sebesar = 25%

227,4757 x 25 % = 56,869 KNm

Mu akibat distribusi momen = 227,4757 – 56,869 = 170,607 KNm

$$\frac{Mu}{\phi} = \frac{170{,}607}{0{,}8} = 213{,}258 \text{ KNm}$$

$$\rho_b = \frac{0{,}85.fc'}{fy}\beta_1\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25.0{,}85}{400}\left(\frac{600}{600+400}\right) = 0{,}0271 \qquad \textbf{(3.3.2)}$$

$\rho_{maks} = 0{,}75\ \rho b = 0{,}75.0{,}0271 = 0{,}0203$ **(3.3.3)**

rasio tulangan rencana = 0,5. $\rho_{maks} = 0{,}5.0{,}0203 = 0{,}01015$ **(3.3.4)**

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

$$m = \frac{fy}{0{,}85.fc'} = \frac{400}{0{,}85.25} = 18{,}824 \qquad \textbf{(3.3.5)}$$

$$Rn = \rho.fy.\left(1-\frac{1}{2}.\rho.m\right) = 0{,}01015.400\left(1-\frac{1}{2}.0{,}01015.18{,}824\right) = 3{,}675\ \text{Mpa} \qquad \textbf{(3.3.6)}$$

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{213{,}258.10^6}{3{,}675.300}} = 439{,}808\ \text{mm} \qquad \textbf{(3.3.7)}$$

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

= 600 – 100 = 500 mm > d_{perlu}, maka dipakai tulangan sebelah.

$$Rn_{ada} = \frac{Mu/\phi}{b.d^2} = \frac{213{,}258.10^6}{300.500^2} = 2{,}843\ \text{Mpa} \qquad \textbf{(3.3.8)}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}.\rho = \frac{2{,}843}{3{,}675}.0{,}01015 = 0{,}00785 > \rho_{min} = 0{,}0035 \qquad \textbf{(3.3.9)}$$

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

$As_{perlu} = \rho_{ada}.b.d = 0{,}00785.300.500 = 1177{,}997\ mm^2$ **(3.3.10)**

Dipakai tulangan Ø19 dengan A1Ø = 283,529 mm^2

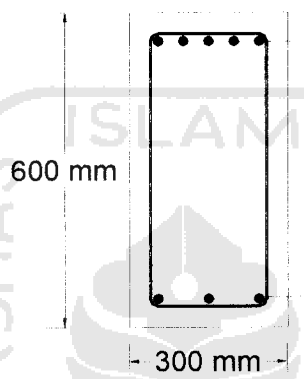
$$\text{jumlah tulangan (n)} = \frac{As_{perlu}}{A1\phi} = \frac{1177{,}997}{283{,}529} = 4{,}155\ \text{batang} \qquad \textbf{(3.3.11)}$$

dipakai 5D19, maka $As_{ada} = 5\ .\ 283{,}529 = 1417{,}645\ mm^2 > As_{perlu}$

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

$$= \frac{300 - 2.40 - 2.10 - 5.19}{(5-1)}$$

= 26,25 mm > 25 mm, maka dipakai tulangan 1 lapis



Gambar 4.26 tulangan pokok balok B1 tumpuan 1

Kontrol kapasitas momen nominal tumpuan negatif :

$$a = \frac{As_{ada.}fy}{0{,}85.fc'.b} = \frac{1417{,}645.400}{0{,}85.25.300} = 88{,}95 \text{ mm} \quad \textbf{(3.3.13)}$$

$$Mn = As_{ada}.fy.(d - a/2) \quad \textbf{(3.3.14)}$$

$$= 1417{,}645.400.(500 - 88{,}95/2)$$

$$= 258{,}309 \text{ KNm} > Mu/\phi = 213{,}258 \text{ KNm} \rightarrow \textbf{OK!}$$

Kontrol kapasitas momen nominal tumpuan positif :

$$a = \frac{As_{ada.}fy}{0{,}85.fc'.b} = \frac{850{,}587.400}{0{,}85.25.300} = 53{,}370 \text{ mm}$$

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

$$= 850{,}587.400.(500 - \frac{53{,}370}{2})$$

$$= 161{,}038 \text{ KNm} > \frac{Mu}{\phi} = 155{,}118 \text{ KNm} \rightarrow \textbf{OK}$$

4.4.1.3 Tulangan Lapangan

Mu = 70,731 KNm

Mu akibat distribusi momen = 115,324 KNm

$$\frac{Mu}{\phi} = \frac{115{,}324}{0{,}8} = 144{,}155 \text{ KNm}$$

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

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

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

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

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

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

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{144{,}155.10^6}{3{,}675.300}} = 361{,}598 \text{ mm}$$

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

= 600 – 100 = 500 mm > d_{perlu}, maka dipakai tulangan sebelah.

$$Rn_{ada} = \frac{Mu/\phi}{b.d^2} = \frac{144{,}155.10^6}{300.500^2} = 1{,}922 \text{ Mpa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}.\rho = \frac{1,922}{3,675}.0,01015 = 0,00531 > \rho_{min} = 0,0035$$

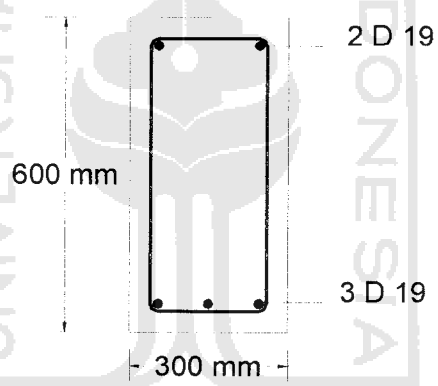
$$< \rho_{maks} = 0,0203$$

$As_{perlu} = \rho_{ada}.b.d = 0,00531.300.500 = 796,285\ mm^2$

Dipakai tulangan Ø19 dengan A1Ø = 283,529 mm^2

$$\text{jumlah tulangan (n)} = \frac{As_{perlu}}{A1\phi} = \frac{796,285}{283,529} = 2,808 \text{ batang}$$

dipakai 3D19, maka $As_{ada} = 3 \,.\, 283,529 = 850,587\ mm^2 > As_{perlu}$



Gambar 4.27 tulangan pokok balok B1 lapangan

Kontrol kapasitas m,omen nominal :

$$a = \frac{As_{ada}.fy}{0,85.fc'.b} = \frac{850,587.400}{0,85.25.300} = 53,370\ mm$$

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

$$= 850,587.400.(500 - 53,370/2)$$

$$= 161,038\ KNm > Mu/\phi = 144,155\ KNm \rightarrow \textbf{OK!}$$

4.4.1.4 Tulangan Tumpuan 2

Mu = 178,373 KNm

Mu akibat distribusi momen = 133,780 KNm

$$\frac{Mu}{\phi} = \frac{133{,}780}{0{,}8} = 167{,}225 \text{ KNm}$$

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

ρ_{maks} = 0,75 ρb = 0,75.0,0271 = 0,0203

rasio tulangan rencana = 0,5. ρ_{maks} = 0,5.0,0203 = 0,01015

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

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

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

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{167{,}225.10^6}{3{,}675.300}} = 389{,}459 \text{ mm}$$

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

= 600 – 100 = 500 mm > d_{perlu}, maka dipakai tulangan sebelah.

$$Rn_{ada} = \frac{Mu/\phi}{b.d^2} = \frac{167{,}225.10^6}{300.500^2} = 2{,}230 \text{ Mpa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}.\rho = \frac{2{,}230}{3{,}675}.0{,}01015 = 0{,}00616 > \rho_{min} = 0{,}0035$$

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

$As_{perlu} = \rho_{ada}.b.d$ = 0,00616.300.500 = 924 mm^2

Dipakai tulangan Ø19 dengan A1Ø = 283,529 mm^2

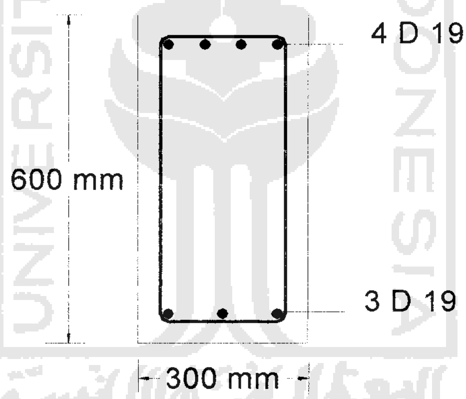
$$\text{jumlah tulangan (n)} = \frac{As_{perlu}}{A1\phi} = \frac{924}{283{,}529} = 3{,}259 \text{ batang}$$

dipakai 4D19, maka As_{ada} = 4. 283,529 = 1134,116 mm^2 > As_{perlu}

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

$$= \frac{300 - 2.40 - 2.10 - 4.19}{(4-1)}$$

= 31,0 mm > 25 mm, maka dipakai tulangan 1 lapis



Gambar 4.28 tulangan pokok balok B1 tumpuan 2

Kontrol kapasitas momen nominal tumpuan negatif :

$$a = \frac{As_{ada}.fy}{0{,}85.fc'.b} = \frac{1134{,}116.400}{0{,}85.25.300} = 71{,}160 \text{ mm}$$

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

$$= 1134{,}116.400.(500 - 71{,}160/2)$$

$$= 210{,}682 \text{ KNm} > Mu/\phi = 167{,}225 \text{ KNm} \rightarrow \textbf{OK!}$$

Kontrol kapasitas momen nominal tumpuan positif :

$$a = \frac{As_{ada}.fy}{0{,}85.fc'.b} = \frac{850{,}587.400}{0{,}85.25.300} = 53{,}370 \text{ mm}$$

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

$$= 850{,}587.400.(500 - 53{,}370/2)$$

$$= 161{,}038 \text{ KNm} > Mu/\phi = 122{,}144 \text{ KNm} \rightarrow \text{OK}$$

4.4.2 Perencanaan Tulangan Geser Balok

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

$V_D = 62{,}550$ KN ; $V_L = 25{,}01$ KN ; $V_E = 45{,}02$ KN

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

$$Vu,b = 0{,}7.1{,}25\left[\frac{258{,}310 + 161{,}038}{4{,}475}\right] + 1{,}05(62{,}550 + 25{,}01) = 173{,}933 \text{ KN}$$

Dengan syarat tidak lebih besar dari :

$$Vu,b = 1{,}07\ (\ 62{,}550 + 25{,}01 + 4/1\ .\ 45{,}02\ ) = 286{,}375 \text{ KN}$$

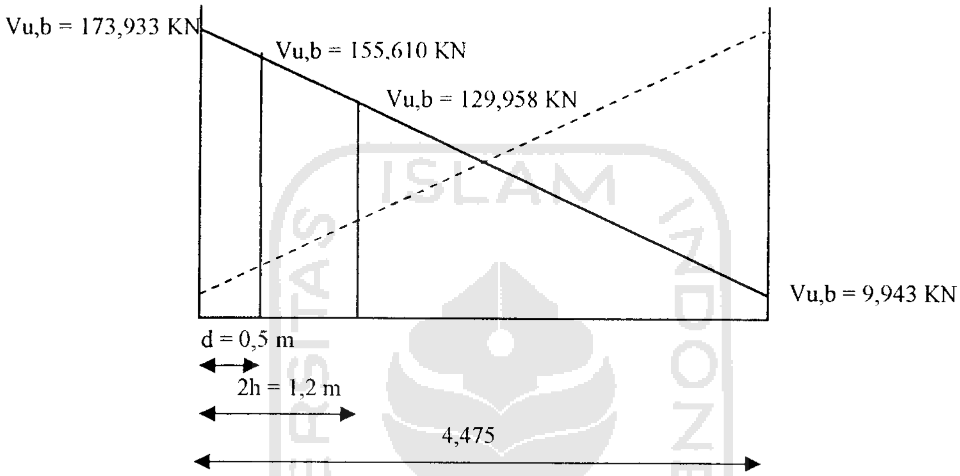
$$Vu,b_{\ pakai} = \left[1{,}05Vg - 0{,}7\phi_0\left(\frac{Mnak,b + Mnak,b'}{Ln}\right)\right] +$$

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

$$= \left[1{,}05.87{,}56 - 0{,}7.1{,}25\left(\frac{258{,}310 + 161{,}038}{4{,}475}\right)\right] +$$

$$\frac{4{,}475 - 0{,}5}{4{,}475}\left[173{,}933 - \left[1{,}05.87{,}56 - 0{,}7.1{,}25\left[\frac{258{,}310 + 161{,}038}{4{,}475}\right]\right]\right]$$

$$= 155{,}610 \text{ KN}$$



Gambar 4.29 Diagram tegangan geser balok B1

1) Dalam daerah sendi plastis

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

Vu,b = 155,610 KN

Vc = 0

$$\frac{Vu,b}{\phi} = \frac{155{,}610}{0{,}6} = 259{,}350 \text{ KN}$$

Digunakan sengkang □ P10 mm, maka : $Av = 2 \cdot \frac{1}{4} \cdot \pi \cdot 10^2 = 157 \text{ mm}^2$

Jarak sengkang :

$$s \leq \frac{Av.fy.d}{Vu,b/\phi - Vc} = \frac{157.240.500}{259{,}350 - 0} \cdot 10^{-3} = 72{,}680 \text{ mm} \qquad \textbf{(3.3.35)}$$

$\leq d/4 = 500/4 = 125$ mm

≤ 300 mm

Jadi dipakai tulangan geser **P10 – 70 mm**

2) Diluar sendi plastis

Diambil jarak sejauh 2h = 2.600 = 1200 mm dengan Vu,b = 129,958 KN

$Vc = 1/6.\sqrt{fc'}.b.d = 1/6.\sqrt{25}.300.500 = 125$ KN

$$Vs = \frac{Vu,b}{\phi} - Vc = \frac{129,958}{0,6} - 125 = 91,597 \text{ KN}$$

Digunakan sengkang □ P10 mm, maka : $Av = 2 . ¼ . \pi . 10^2 = 157 \text{ mm}^2$

Jadi ; s $\leq \dfrac{Av.fy.d}{Vs} = \dfrac{157.240.500}{91,597} . 10^{-3} = 205,788$ mm **(3.3.32)**

$\leq d/2 = 500/2 = 250$ mm

≤ 600 mm

Jadi dipakai tulangan geser **P10 – 200 mm**

4.4.3 Perencanaan Tulangan Torsi

Tu = 3,85 KNm

$\sum x^2.y = 300^2 . 600 = 54.10^6 \text{ mm}^3$

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 :

$$Tu,b = \phi(\frac{1}{9}\sqrt{f'c}.\sum x^2.y) = 0,6.(\frac{1}{9}\sqrt{25}.54.10^6) \qquad \textbf{(3.3.38)}$$

= 18,00 KNm > Tu = 3,85 KNm , tulangan torsi diabaikan.

4.5 Perencanaan Kolom

4.5.1 Perhitungan Momen dan Gaya Aksial Rencana

1. Momen untuk portal arah X

Data Momen K1 (As 3 = 6) lantai 1

(tabel 4.31 Lampiran II – 60 Perhitungan Momen Rencana Kolom Arah X)

$M_{Dy\ atas}$ = -0,27 KNm

$M_{Dy\ bawah}$ = 0,17KNm

$M_{Ly\ atas}$ = -0,04 KNm

$M_{Ly\ bawah}$ = 0,01KNm

$M_{Ey\ atas}$ = -74,52 KNm

$M_{Ey\ bawah}$ = 79,56 KNm

Atas

$1{,}2\ M_{Dy} + 1{,}6\ M_{Ly} = 1{,}2\ .(-0{,}27) + 1{,}6.(-0{,}04) = -0{,}388$ KNm

$1{,}05\ (M_{Dy} + M_{Ly} + M_{Ey}) = 1{,}05\ ((-0{,}27)+(-0{,}04)+(-74{,}52)) = -78{,}572$ KNm

Bawah

$1{,}2\ M_{Dy} + 1{,}6\ M_{Ly} = 1{,}2\ .0{,}17 + 1{,}6.0{,}01 = 0{,}222$ KNm

$1{,}05\ (M_{Dx} + M_{Lx} + M_{Ex}) = 1{,}05(0{,}17 + 0{,}01 + 79{,}56) = 83{,}727$ KNm

2. Momen untuk portal arah Y

Data Momen

(tabel 4.30 Lampiran II – 58 Perhitungan Momen Rencana Kolom Arah Y)

$M_{Dx\ atas}$ = -23,10 KNm

$M_{Dx\ bawah}$ = 10,02 KNm

$M_{Lx\ atas}$ = -10,3 KNm

$M_{Lx\ bawah}$ = 3,56 KNm

$M_{Ex\ atas}$ = -32,90 KNm

$M_{Ex\ bawah}$ = 73,46 KNm

Atas

$1,2\ M_{Dx} + 1,6\ M_{Lx}$ = 1,2 .(-23,10) + 1,6.(-10,30) = - 44,20 KNm

$1,05\ (M_{Dx} + M_{Lx} + M_{Ex})$ =1,05 ((-23,10)+(-10,30) +(- 32,90))= -69,615 KNm

Bawah

$1,2\ M_{Dx} + 1,6\ M_{Lx}$ = 1,2 .10,02 + 1,6.3,56 = 17,72 KNm

$1,05\ (M_{Dx} + M_{Lx} + M_{Ex})$ =1,05(10,02+ 3,56+ 73,46) = 91,392 KNm

3. Gaya Aksial

Data Gaya Aksial

(tabel 4.32 Lampiran II – 60 Perhitungan Gaya Aksial Kolom)

$P_{D\ atas}$ = -812,50 KN

$P_{D\ bawah}$ = -812,50 KN

$P_{L\ atas}$ = -218,14 KN

$P_{L\ bawah}$ = -218,14 KN

$P_{E\ atas}$ = -112,54 KN

$P_{E\ bawah}$ = -112,54 KN

$1,2\ P_D + 1,6\ P_L$ = 1,2 .812,50 + 1,6.218,14 = 1324,02 KN

$1,05\ (P_D + P_L + P_E)$ = 1,05 (812,50 +218,14 +112,54)= 1200,34 KN

Pu pakai : Atas = -1324,02 KN

Bawah = -1324,02 KN

4.5.2 Kriteria Kolom dan Pembesaran Kolom

Menghitung Kekakuan Kolom

1. Arah X

$$Ec = Eg = 4700\sqrt{f'c}$$

$$= 4700\ \sqrt{25}$$

$$= 23500 \text{ Mpa}$$

Profil 450x600

Ic (Inersia kolom) $= \frac{1}{12}.450.600^3 = 8{,}1.10^9 \text{ mm}^4$

$$\beta d = \frac{1{,}2M_D}{1{,}2M_D + 1{,}6M_L} = \frac{1{,}2.0{,}27}{1{,}2.0{,}27 + 1{,}6.0{,}04} = 0{,}835 \qquad \textbf{(3.4.39)}$$

$$EI = \frac{E_c.I_c}{2{,}5(1+\beta.d)} = \frac{23500.x8{,}10.10^9}{2{,}5(1+0{,}835)} = 4{,}15.\ 10^{13} \text{ Nmm}^2 \qquad \textbf{(3.4.38)}$$

Menghitung momen inersia balok di kanan dan kiri kolom, dengan menganggap momen inersia penampang retak balok sebesar setengah dari momen inersia penampang bruto, maka :

1. Momen inersia balok di kanan kiri atas kolom yaitu :

$$I_{cr} = \frac{1}{2}.\left(\frac{1}{12}.b.h^3\right) = \frac{1}{2}.\left(\frac{1}{12}.300.600^3\right) = 2{,}7.10^9 \text{ mm}^4$$

2. Momen inersia balok di kanan kiri balok bawah = 0, karena ujung jepit.

   Lc (panjang bersih kolom) = 3,2 m

   Lg (panjang bersih balok) = 4,55 m

$$\psi_{atas} = \psi_{bawah} = \frac{\sum\left(\frac{EI}{Lc}\right)}{\sum\left(\frac{Ec.Icr}{Lg}\right)} \quad \textbf{(3.4.31)}$$

$$\psi_{atas} = \frac{\left(\frac{4{,}15.10^{13}}{3200} + \frac{4{,}15.10^{13}}{2900}\right)}{\left(\frac{23500.2{,}7.10^{9}}{4550} + \frac{23500.2{,}7.10^{9}}{4550}\right)} = 0{,}978$$

$$\psi_{bawah} = 0 \text{ (ujung jepit)}$$

Dari Nomogram portal tanpa pengaku, didapat k = 1,10

$$\frac{k.lu}{r} = \frac{1{,}10.3200}{0{,}3.450} = 26{,}07 \geq 22 \text{ (termasuk kolom langsing)}$$

Beban tekuk Euler yang terjadi adalah

$$Pc = \frac{\pi.^2 EI}{(k.Lc)^2} = \frac{\pi.^2 4{,}15.10^{13}}{(1{,}10.3200)^2} = 33056929 \text{ N} \quad \textbf{(3.4.36)}$$

Menghitung faktor pembesaran momen δ_{by}

$$\delta_{by} = \frac{Cm}{1-\left(\frac{Pu}{\phi.Pc}\right)} \geq 1 \quad \textbf{(3.4.33)}$$

Cm = 1 (untuk portal tanpa pengaku)

$$\delta_{by} = \frac{1}{1-\left(\frac{1324020}{0{,}65.33056929}\right)} \geq 1$$

$$= 1{,}065 \geq 1$$

Menghitung faktor pembesaran momen δ_{sy}

Σ Pu = (2.1063708 + 2.1231356 + 2.1324024 + 2.1332212)

= 9902600 N

Σ Pc =(2.38710754 + 2.32884116 + 2.33050575 + 2.30705002)

= 270700894 N

$$\delta_{sy} = \frac{1}{1-\sum\left(\frac{Pu}{\phi..Pc}\right)} \geq 1 \qquad \textbf{(3.4.35)}$$

$$= \frac{1}{1-\left(\frac{9902600}{0,65.270700894}\right)} \geq 1$$

= 1,0596 $\geq$ 1

Momen akibat pembesaran momen

Atas , Mby = 1,05.(MDy + 0,9. MLy)

= 1,05. (0,27 + 0,9 . 0,04) = 0,321 KNm

Msy = 1,05. MEy = 1,05. 74,52 = 78,246 KNm

Bawah , Mby = 1,05.(MDy + 0,9. MLy)

= 1,05. (0,17 + 0,9 . 0,01) = 0,188 KNm

Msy = 1,05. MEy = 1,05. 79,56 = 83,538 KNm

Muy, bawah = δ_{by} Mby + δ_{sy} Msy

= (1,065 . 0,188) + (1,0596 . 83,538)

= 88,720 KNm

Muy, atas = δ_{by} Mby + δ_{sy} Msy

= (1,065 . 0,321) + (1,0596 . 78,246)

= 83,255 KNm

2. Arah Y

$$Ec = Eg = 4700\sqrt{f'c}$$

$$= 4700\ \sqrt{25}$$

$$= 23500 \text{ Mpa}$$

Profil 450 x 600

$$Ic \text{ (Inersia kolom)} = \frac{1}{12}.450^3.600 = 4{,}56.10^9 \text{ mm}^4$$

$$\beta d = \frac{1{,}2M_D}{1{,}2M_D + 1{,}6M_L} = \frac{1{,}2.23{,}1}{1{,}2.23{,}1 + 1{,}6.10{,}3} = 0{,}627$$

$$EI = \frac{E_c.I_c}{2{,}5(1+\beta.d)} = \frac{23500.x4{,}56.10^9}{2{,}5(1+0{,}627)} = 2{,}63.10^{14} \text{ Nmm}^2$$

Menghitung momen inersia balok di kanan dan kiri kolom, dengan menganggap momen inersia penampang retak balok sebesar setengah dari momen inersia penampang bruto, maka :

1. Momen inersia balok di kanan kiri atas kolom yaitu :

$$I_{cr} = \frac{1}{2}.\left(\frac{1}{12}.b.h^3\right) = \frac{1}{2}.\left(\frac{1}{12}.300.500^3\right) = 1{,}56.10^9 \text{ mm}^4$$

2. Momen inersia balok di kanan kiri balok bawah = 0, karena ujung jepit.

   Lc (panjang bersih kolom) = 3,25 m

   Lg (panjang bersih balok) = 4,4 m

$$\psi_{atas} = \psi_{bawah} = \frac{\sum\left(\frac{EI}{Lc}\right)}{\sum\left(\frac{Ec.Icr}{Lg}\right)}$$

$$\psi_{atas} = \frac{\left(\dfrac{2{,}63.10^{13}}{3250} + \dfrac{2{,}63.10^{13}}{3000}\right)}{\left(\dfrac{23500.1{,}56.10^{9}}{4400}\right)} = 2{,}02$$

ψ_{bawah} = 0 (ujung jepit)

Dari Nomogram portal tanpa pengaku, didapat k = 1,28

$$\frac{k.lu}{r} = \frac{1{,}28.3250}{0{,}3.600} = 23{,}111 \geq 22 \text{ (termasuk kolom langsing)}$$

Beban tekuk Euler yang terjadi adalah

$$Pc = \frac{\pi.^2 EI}{(k.Lc)^2} = \frac{\pi.^2 2{,}63.10^{13}}{(1{,}28.3250)^2} = 15011396 \text{ N}$$

Menghitung faktor pembesaran momen δ_{bx}

$$\delta_{bx} = \frac{Cm}{1-\left(\dfrac{Pu}{\phi.Pc}\right)} \geq 1$$

Cm = 1 (untuk portal tanpa pengaku)

$$\delta_{bx} = \frac{1}{1-\left(\dfrac{1324024}{0{,}65.15011396}\right)} \geq 1$$

$$= 1{,}16$$

Menghitung faktor pembesaran momen δ_{sx}

Σ Pu = (1324024 + 1823076 + 1731612 + 1344912 + 73272)

= 6296896 N

Σ Pc = (15011396 + 9809577)

= 24820973 N

$$\delta_{sx} = \frac{1}{1-\sum\left(\frac{Pu}{.\phi.Pc}\right)} \geq 1$$

$$= \frac{1}{1-\left(\frac{6296896}{0,65.24820973}\right)} \geq 1$$

$$= 1,64$$

Momen akibat pembesaran momen

Atas , Mbx = 1,05.(MDx + 0,9. MLx)

= 1,05. (23,01 + 0,9 . 10,3) = 33,988 KNm

Msx = 1,05. MEx = 1,05. 32,09 = 34,545 KNm

Bawah , Mbx = 1,05.(MDx + 0,9. MLx)

= 1,05. (10,02 + 0,9 . 3,56) = 13,885 KNm

Msx = 1,05. MEx = 1,05. 73,46 = 77,133 KNm

Mux, bawah = δ_{bx} Mbx + δ_{sx} Msx

= (1,16 . 13,885) + (1,64 . 77,133)

= 142,574 KNm

Mux, atas = δ_{bx} Mbx + δ_{sx} Msx

= (1,16 . 33,988) + (1,64 . 34,545)

= 95,983 KNm

4.5.3 Analisis Gaya Aksial dan Momen akibat balok

Perhitungan kolom K1 (As 3 – 6) lantai 1

h = 3,5 m

hn = 3,2 m

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

ωd = 1,3 kecuali untuk kolom lantai 1 dan lantai paling atas yang kemungkinan terjadi sendi plastis pada kolom, ωd = 1

k = 1

a. Perhitungan Arah X

$Mkap_{(kiri)}$ = 1,25. Mnak = 1,25.210,682 = 263,353 KNm

$Mkap_{(kanan)}$= 1,25. Mnak = 1,25. 161,038 = 201,298 KNm

menghitung gaya aksial rencana :

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

$$= 0,7.\ 1.\ \frac{263,353}{5} + \frac{201,298}{5} + 1,05(812,50 + 218,14)$$

$$= 1147 \text{ KN}$$

tidak perlu melebihi :

$$Pu,k_y = 1,05\ (N_D + N_L + 4.N_E)$$

$$= 1,05\ (812,50 + 218,14 + 4.112,54\ )$$

$$= 1554,84 \text{ KN}$$

menghitung α :

$M_{E,K\ atas}$ = 74,52 KNm

$M_{E,K\ bawah}$ = 79,56 KNm

$$\alpha\ ka = \frac{M_{E,k(lti+1atas)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = \frac{74{,}52}{74{,}52 + 79{,}56} = 0{,}484$$

$$\alpha\ kb = \frac{M_{E,k(ltibawah)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = \frac{79{,}56}{74{,}52 + 79{,}56} = 0{,}516$$

menghitung momen rancang kolom :

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

$$= \frac{3{,}5}{3{,}2}.1{,}0\ .\ 0{,}484.0{,}7.\left(\frac{5}{4{,}55}.263{,}353 + \frac{5}{4{,}55}.201{,}298\right)$$

$$= 189\ KNm$$

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

$$= \frac{3{,}5}{3{,}2}.1{,}0\ .\ 0{,}516\ .0{,}7.\left(\frac{5}{4{,}55}.263{,}353 + \frac{5}{4{,}55}.201{,}298\right)$$

$$= 202\ KNm$$

tidak perlu melebihi :

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

$$= 1{,}05\ (0{,}27 + 0{,}04 + \frac{4}{1}(74{,}52))$$

$$= 313{,}310\ KNm$$

b. .Perhitungan Arah Y

$Mkap_{(kiri)}$ = 1,25. Mnak = 1,25.178,927 = 223,659 KNm

$Mkap_{(kanan)}$ = 1,25. Mnak = 1,25.0 = 0 KNm menghitung gaya aksial rencana :

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

$$= 0{,}7.\ 1.\ \frac{223{,}659 + 0}{5} + 1{,}05(812{,}50 + 218{,}14\ )$$

$$= 1113\ KN$$

tidak perlu melebihi :

$$Pu,k_x \quad = 1{,}05\ (N_D + N_L + 4.N_E)$$

$$= 1{,}05\ (812{,}50 + 218{,}50 + 4.112{,}54\ )$$

$$= 1554{,}84\ KN$$

menghitung α :

$$M_{E,K\ atas} \quad = 32{,}9\ KNm$$

$$M_{E,K\ bawah} \quad = 73{,}46\ KNm$$

$$\alpha\ ka = \frac{M_{E,k(lti+1atas)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = \frac{32{,}9}{32{,}9 + 73{,}46} = 0{,}309$$

$$\alpha\ kb = \frac{M_{E,k(ltibawah)}}{M_{E,k(lti+1atas)} + M_{E,k(ltibawah)}} = \frac{73{,}46}{32{,}9 + 73{,}46} = 0{,}691$$

menghitung momen rancang kolom :

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

$$= \frac{3{,}5}{3{,}25}.1{,}0\ .\ 0{,}309.0{,}7.\left(\frac{5}{4{,}4}.223{,}659 + \frac{5}{4{,}4}.0\right)$$

$$= 59\ KNm$$

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

$$= \frac{3,5}{3,25}.1,0.0,691\ .\ 0,7.\left(\frac{5}{4,4}.223,659 + \frac{5}{4,4}.0\right)$$

$$= 132 \text{ KNm}$$

tidak perlu melebihi :

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

$$= 1,05\ (23,1 + 10,3 + \frac{4}{1}(73,46))$$

$$= 343,602 \text{ KNm}$$

4.5.4 Perencanaan Tulangan Lentur Kolom

Untuk perencanaan penulangan kolom dipakai nilai terbesar dari hasil analisis SAP 2000 dan momen akibat momen kapasitas balok, maka :

Pu,k = 1324 KN

Mu,k_x = 132 KNm

Mu,k_y = 202 KNm

$$\frac{Pu,k}{\phi} = \frac{1324}{0,65} = 2037 \text{ KN}$$

$$\frac{Mu,k_x}{\phi} = \frac{132}{0,65} = 203,077 \text{ KNm}$$

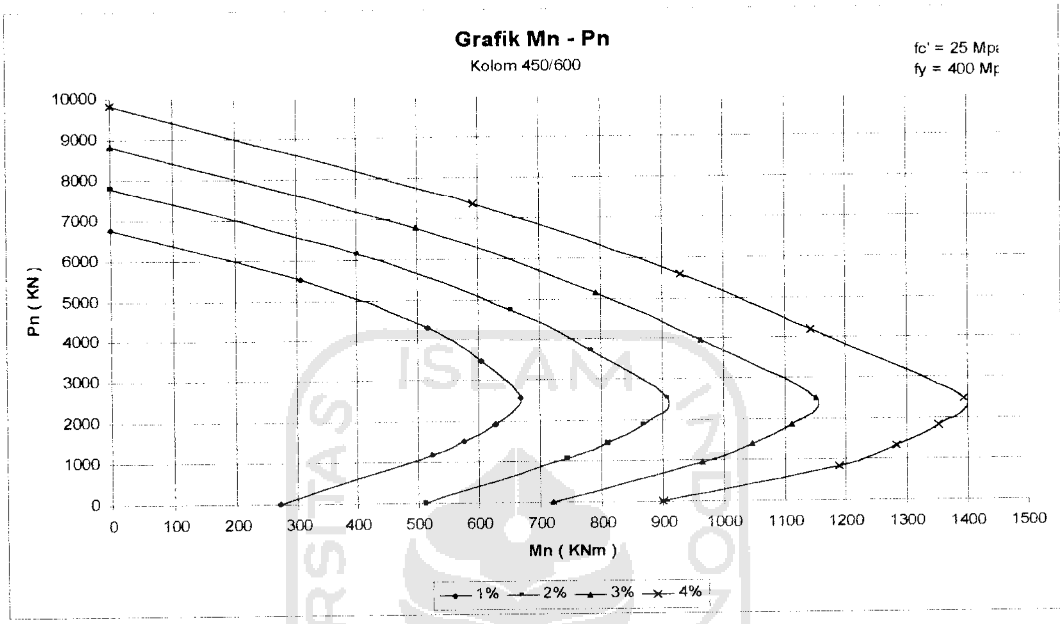
$$\frac{Mu,k_y}{\phi} = \frac{202}{0,65} = 310,377 \text{ KNm}$$

$$e = \frac{M}{P} = \frac{202}{1324} = 0,153 \text{ m} > e \text{ min} = (15 + 0,03.h) \text{ mm}$$

$$= (15 + 0,03.600) = 33 \text{ mm} \quad \textbf{Ok!}$$

$$e = \frac{M}{P} = \frac{132}{1324} = 0{,}099 \text{ m} > e \text{ min} = (15 + 0{,}03.h) \text{ mm}$$

$$= (15 + 0{,}03.600) = 33 \text{ mm } \textbf{Ok!}$$



Gambar 4.30 Grafik Mn - Pn kolom

a. Arah x

$$\frac{Pu,k}{\phi} = \frac{1324}{0{,}65} = 2037 \text{ KN}$$

$$\frac{Mu,k_y}{\phi} = \frac{202}{0{,}65} = 310{,}769 \text{ KNm}$$

Dari grafik Mn vs Pn didapat $\rho_g = 1{,}0$ %

Ast = 0,010.450.600 = 2700 mm^2

As = As' = 0,5.Ast = 1350 mm^2

dipakai 4 D22 dengan $As_{ada} = As'_{ada} = 1521$ mm^2

Cek eksentrisitas balance (e_b)

$$Xb = \frac{600.d}{600 + fy} = \frac{600.530}{600 + 400} = 318 \text{ mm} \quad \textbf{(3.4.12)}$$

$$ab = \beta_1 . Xb = 0{,}85 . 318 = 270{,}3 \text{ mm} \quad \textbf{(3.4.21)}$$

$$f's = 600\frac{(Xb-d')}{Xb} = 600\frac{(318-70)}{318} = 468 \text{ MPa} > fy = 400 \text{ MPa} \qquad \textbf{(3.4.22)}$$

Dengan demikian digunakan f's = fy = 400 MPa

$$Ccb = 0{,}85.f'c.b.ab = 0{,}85.25.450.270{,}3 = 2584743{,}75 \text{ N} \qquad \textbf{(3.4.23)}$$

$$Csb = As'(fs' - 0{,}85.f'c) = 1521.(400 - 0{,}85.25) = 576078{,}75 \text{ N} \qquad \textbf{(3.4.24)}$$

$$Tsb = As.fy = 1521.400 = 608400 \text{ N} \qquad \textbf{(3.4.25)}$$

$$Pnb = Ccb + Csb - Tsb = 2297550 + 576078{,}75 - 608400 \qquad \textbf{(3.4.26)}$$

$$= 2552422{,}50 \text{ N} = 2552 \text{ KN}$$

$$Mnb = Ccb\left[\frac{h}{2}-\frac{ab}{2}\right] + Csb\left(\frac{h}{2}-d'\right) + Tsb\left(d-\frac{h}{2}\right) \qquad \textbf{(3.4.27)}$$

$$= 2584743{,}75\left[\frac{600}{2}-\frac{270{,}3}{2}\right] + 576078{,}75.\left(\frac{600}{2}-70\right) + 608400.\left(530-\frac{600}{2}\right)$$

$$= 698 \text{ KNm}$$

$$eb = \frac{Mnb}{Pnb} = \frac{698}{2552} = 0{,}274 \text{ m} \qquad \textbf{(3.4.28)}$$

$$e = \frac{Mu_k, y/\phi}{Pu_k/\phi} = \frac{310{,}769}{2037} = 0{,}153 \text{ m}$$

karena $e < e_b$ $\longrightarrow$ kolom mengalami patah desak

Kontrol tegangan pada daerah desak :(Rumus Whitney)

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

$$= \frac{1521.400}{\frac{153}{(530-70)}+0{,}5} + \frac{450.600.25}{\frac{3.600.153}{530^2}+1{,}18}$$

$$= 730715{,}405 + 31243392{,}366 = 3859107{,}771 \text{ N}$$

$$Pn = 3860 \text{ KN} > \frac{Pu_k}{\phi} = 2037 \text{ KN} \text{ .... } \textbf{Ok !}$$

$$Mn = Pn \cdot e$$

$$= 3860 \cdot 0{,}153$$

$$= 589 \text{ KNm} > \frac{Mu, k_y}{\phi} = 310{,}769 \text{ KNm}$$

b. Arah y

$$\frac{Pu,k}{\phi} = \frac{1324}{0{,}65} = 2037 \text{ KN}$$

$$\frac{Mu,kx}{\phi} = \frac{132}{0{,}65} = 203{,}077 \text{ KNm}$$

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

$Ast = 0{,}01.450.600 = 2700 \text{ mm}^2$

$As = As' = 0{,}5.Ast = 1350 \text{ mm}^2$

dipakai 4D 22 dengan $As_{ada} = As'_{ada} = 1521 \text{ mm}^2$

Cek eksentrisitas balance (e_b)

$$cb = \frac{600.d}{600 + fy} = \frac{600.380}{600 + 400} = 228 \text{ mm}$$

$ab = \beta_1 \cdot cb = 0{,}85 \cdot 228 = 193{,}8$ mm

$$f'sb = 600 \frac{(cb - d')}{cb} = 600 \frac{(228 - 70)}{228} = 415{,}789 \text{ MPa} > fy = 400 \text{ MPa}$$

digunakan f'sb = fy = 400 MPa

Ccb = 0,85.f'c.b.ab = 0,85.25.600. 193,8 = 2470950 N

Csb = As'(fs'- 0,85.f'c) = 1521.(400 – 0,85.25) = 576078,75 N

Tsb = As.fy = 1521.400 = 608400 N

$$Pnb = Ccb + Csb - Tsb = 2470950 + 576078{,}75 - 608400$$

$$= 2438628{,}75 \text{ N}$$

$$= 2439 \text{ KN}$$

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

$$=2470950.\left[\frac{450}{2}-\frac{193{,}8}{2}\right]+576078{,}75.\left(\frac{450}{2}-70\right)+608400.\left(380-\frac{450}{2}\right)$$

$$= 500 \text{ KNm}$$

$$eb = \frac{Mnb}{Pnb} = \frac{500}{2439} = 0{,}205$$

$$e = \frac{Mu_{,kx}/\phi}{Pu_{,k}/\phi} = \frac{203{,}077}{2037} = 0{,}100 \text{ m}$$

karena $e < e_b$,kolom mengalami patah desak

Kontrol tegangan pada daerah desak :

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

$$= \frac{1521.400}{\frac{100}{(380-70)}+0{,}5} + \frac{450.600.25}{\frac{3.450.100}{380^2}+1{,}18}$$

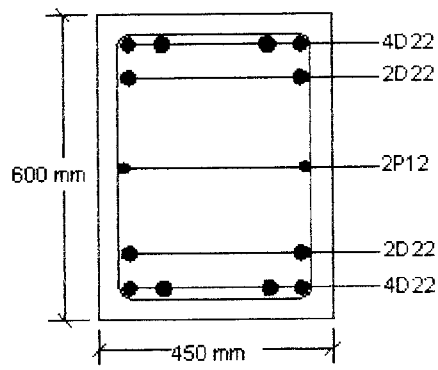
$$= 739623{,}529 + 3191635{,}668 = 3936259{,}197 \text{ N}$$

$$Pn = 3936 \text{ KN} > \frac{Pu_k}{\phi} = 2037 \text{ KN}$$ **................ Ok !**

$$Mn = Pn\ .\ e$$

$$= 3936\ .\ 0{,}100$$

$$= 393{,}6 \text{ KNm} > \frac{Mu,k_y}{\phi} = 203{,}077 \text{ KNm}$$



Gambar 4.31 Penampang Kolom dengan Tulangan

4.5.5 Perencanaan Tulangan Geser Kolom

$Mu_{,k}$ atas = 189 KNm

$Mu_{,k}$ bwh = 202 KNm

$V_{D,}k$ = 9,46 KN

$V_{L,}k$ = 3,96 KN

$V_{E,}k$ = 30,78 KN

h_n = 3,2 m

$$V_{u,k} = \frac{Mu,ky_{atas} + Mu,ky_{bawah}}{h_n} = \frac{189 + 202}{3,2} = 122,188 \text{ KN} \qquad \textbf{(3.6.14)}$$

tetapi tidak perlu lebih besar dari :

$$V_{u,k} = 1,05\,(V_{D,k} + V_{L,k} + \frac{4}{k}(V_{E,k}) \qquad \textbf{(3.6.15)}$$

$$= 1,05\,(\,9,46 + 3,96 + \frac{4}{1}.(30,78))$$

$$= 143,367 \text{ KN}$$

di daerah sejauh *lo*

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

$$Vs = \frac{V_U , k}{\phi} = 203{,}647 \text{ KN}$$

Dipakai tulangan geser □P10 mm, maka :

$Av = 2 \cdot \frac{1}{4} \cdot \pi \cdot 10^2 = 157 \text{ mm}^2$

$$\text{Jarak (s)} < \frac{Av.fy.d}{Vs} = \frac{157.240.530}{203{,}647.10^3} = 98{,}064 \text{ mm} \qquad \textbf{(3.3.30)}$$

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

$< 16.D = 160$ mm

Digunakan sengkang $\mathbf{P_{10\text{-}90}}$ **mm**

di luar daerah *lo*

$$Vc = \left(1 + \frac{P_U . k}{14.A_g}\right) \cdot \frac{1}{6}\sqrt{fc'}.b.d = \left(1 + \frac{1324.10^3}{14.450.600}\right) \cdot \frac{1}{6}\sqrt{25}.450.530$$

$$= 268{,}365 \text{ KN} > \frac{V_U , k}{\phi} = 203{,}647 \text{ KN}$$

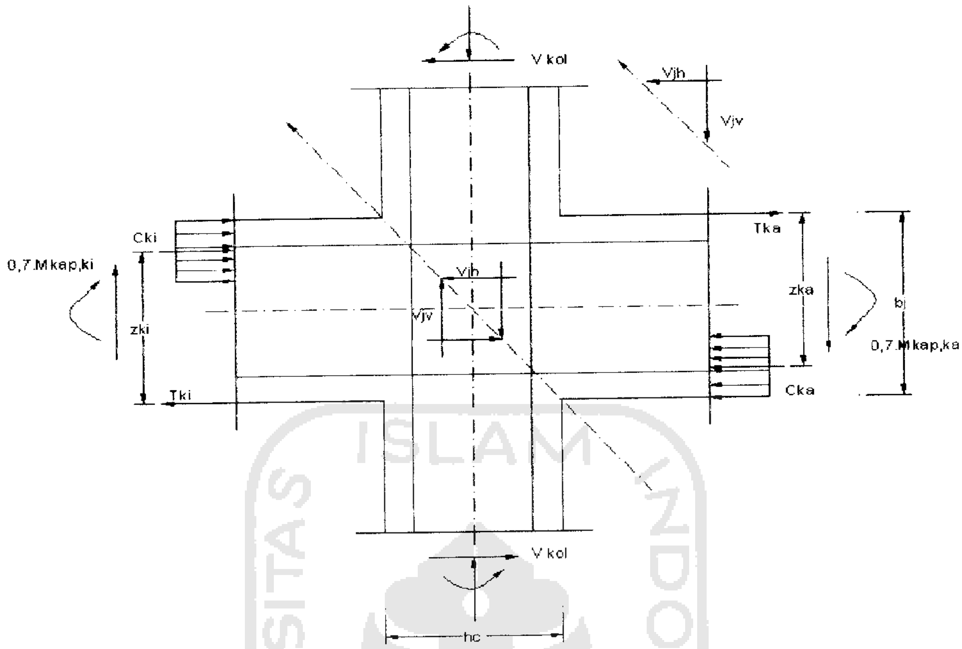
Dipakai tulangan geser □P10 mm, maka :

Jarak (s) $< d/2 = 265$ mm

$< 16.D = 160$ mm

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

4.5.7. Pertemuan Balok Kolom



Gambar 4.32 Joint Balok Kolom dalam

a. Perhitungan gaya-gaya dalam

1). Sumbu X

b_j = bc = 450 mm

= bb + 0,5.hc = 300 + 0,5.600 = 600 mm

b_j pakai = 450 mm

h_c = 600 mm

$$V_{kol,x} = \frac{0{,}7.\phi_o.\left(\sum \frac{l_x}{l_{nx}}.M_{nak,bx} + 0{,}3.\sum \frac{l_y}{l_{ny}}.M_{nak,by}\right)}{\frac{1}{2}.(h_a + h_b)} \quad \textbf{(3.6.20)}$$

$$V_{kol,x} = 0{,}7.1{,}25.\left[\left(\frac{5}{4{,}55}.210{,}682 + \frac{5}{4{,}55}.161{,}038\right)\right.$$

$$\left.+0{,}3.\left(\frac{5}{4{,}4}.178{,}927 + 0\right)\right] \Big/ \frac{1}{2}.(3{,}5 + 3{,}5)$$

$= 117{,}370$ KN

$z_{ki,x} = 0{,}9.d = 0{,}9.530 = 477$ mm $= 0{,}477$ m

$z_{ka,x} = 0{,}9.d = 0{,}9.530 = 477$ mm $= 0{,}477$ m

$C_{ki,x} = T_{ki,x} = 0{,}7.\phi_o.(M_{nak,bx\text{-}ki})/z_{ki,x}$

$= 0{,}7.1{,}25.(210{,}682)/0{,}477 = 386{,}471$ KN **(3.6.18)**

$C_{ka,x} = T_{ka,x} = 0{,}7.\phi_o.(M_{nak,bx\text{-}ka})/z_{ka,x}$

$= 0{,}7.1{,}25.(161{,}038)/0{,}477 = 295{,}405$ KN **(3.6.19)**

$V_{jh,x} = C_{ki,x} + T_{ka,x} - V_{kol,x} = 386{,}471 + 295{,}405 - 117{,}370$

$= 564{,}506$ KN

Kontrol tegangan geser horizontal :

$$v_{jh,x} = \frac{V_{jh,x}}{b_j.h_c} \leq 1{,}5\sqrt{f'c} \qquad \textbf{(3.6.21)}$$

$$v_{jh,x} = \frac{564{,}506}{0{,}45.0{,}6} = 2090{,}763 \ \text{KN/m}^2$$

$= 2{,}1$ N/mm^2 $< 1{,}5.\sqrt{25} = 7{,}5$ N/mm^2 ………….**Ok !**

$$V_{ch,x} = 2/3 . \sqrt{\left\{\left(Pu,k \big/ Ag\right) - 0{,}1.f'c\right\}}.b_j.h_c$$

$$V_{ch,x} = 2/3 . \sqrt{\left\{\left(1324{,}02.10^3 \big/ 600.450\right) - 0{,}1.25\right\}}.450.600 \qquad \textbf{(3.6.23)}$$

$= 279{,}074$ KN

$V_{sh,x} = V_{jh,x} - V_{ch,x}$

$= 564{,}506 - 279{,}074 = 285{,}432$ KN

2). Arah Y

$b_j =$ bc = 600 mm

= bb + 0,5.hc = 300 + 0,5.450 = 525 mm

b_j pakai = 525 mm

$h_c =$ 450 mm

$$V_{kol,y} = \frac{0{,}7.\phi_o.\left(0{,}3.\sum \frac{l_x}{l_{nx}}.M_{nak,bx} + \sum \frac{l_y}{l_{ny}}.M_{nak,by}\right)}{\frac{1}{2}.(h_a + h_b)}$$

$$V_{kol,y} = 0{,}7.1{,}25.\left[0{,}3.\left(\frac{5}{4{,}55}.210{,}682 + \frac{5}{4{,}55}.161{,}038\right)\right.$$

$$\left. + \left(\frac{5}{4{,}4}.178{,}927 + 0\right)\right] \Big/ \frac{1}{2}.(3{,}5 + 3{,}5)$$

$= 81{,}468$ KN

$z_{ki,y} = 0{,}9.d = 0{,}9.430 = 387$ mm $= 0{,}387$ m

$z_{ka,y} = 0{,}9.d = 0{,}9.430 = 387$ mm $= 0{,}387$ m

$C_{ki,y} = T_{ki,y} = 0{,}7.\phi_o.(M_{nak,by\text{-}ki})/z_{ki,y}$

$= 0{,}7.1{,}25.178{,}927/0{,}387 = 404{,}551$ KN

$C_{ka,y} = T_{ka,y} = 0{,}7.\phi_o.(M_{nak,by\text{-}ka})/z_{ka,y}$

$= 0{,}7.1{,}25.\ 0/0{,}477 = 0$ KN

$V_{jh,y} = C_{ki,y} + T_{ka,y} - V_{kol,y}$

$= 404{,}551 + 0 - 81{,}468 = 323{,}083$ KN

Kontrol tegangan geser horizontal :

$$v_{jh,y} = \frac{V_{jh,y}}{b_j.h_c} \leq 1,5\sqrt{f'c}$$

$$v_{jh,y} = \frac{323,083}{0,450.0,525} = 1367,546 \text{ KN/m}^2$$

$$= 1,37 \text{ N/mm}^2 < 1,5.\sqrt{25} = 7,5 \text{ N/mm}^2 \; \ldots\ldots\ldots \textbf{Ok !}$$

$$V_{ch,y} = 2/3 \,.\, \sqrt{\left\{\left({Pu,k}\middle/{Ag}\right) - 0,1.f'c\right\}}\,.b_j.h_c$$

$$V_{ch,y} = 2/3.\, \sqrt{\left\{\left({1324,02.10^3}\middle/{600.450}\right) - 0,1.25\right\}}\,.450.525$$

$$= 244,190 \text{ KN}$$

$$V_{sh,y} = V_{jh,y} - V_{ch,y}$$

$$= 323,083 - 244,190 = 78,893 \text{ KN}$$

b. Penulangan Geser Horisontal

$$V_{sh,mak} = V_{sh,x} = 285,432 \text{ KN}$$

$$A_{jh} = \frac{V_{sh,mak}}{fy} = \frac{285432}{400} = 713,580 \text{ mm}^2$$

Digunakan sengkang □ P10 dengan Av = 157 mm^2

$$\text{Jumlah lapis sengkang} = \frac{713,580}{157} = 4,545 \text{ lapis}$$

digunakan sengkang □ 5P10

c. Penulangan geser vertikal

$$V_{cv} = \frac{A_{sc'}}{A_{sc}} V_{jh,mak} \cdot \left(0{,}6 + {}^{Pu,k}\!\big/_{Ag.f'c}\right) \qquad \textbf{(3.6.30)}$$

$$V_{cv} = 1.564{,}506.10^{3} . \left(0{,}6 + {}^{1324{,}02.10^{3}}\!\big/_{450.600.25}\right)$$

= 449432,079 N = 449,432 KN

$V_{jv} = bj/h_c . V_{jh,mak}$ **(3.6.29)**

= (0,525/0,45). 564,506 = 658,590 KN

$V_{sv} = V_{jv} - V_{cv} = 658{,}590 - 449{,}432 = 209{,}158$ KN

$$A_{jv} = \frac{V_{sv}}{fy} = \frac{209158}{400} = 522{,}896 \text{ mm}^2$$

Digunakan sengkang □ P10 dengan Av = 157 mm^2

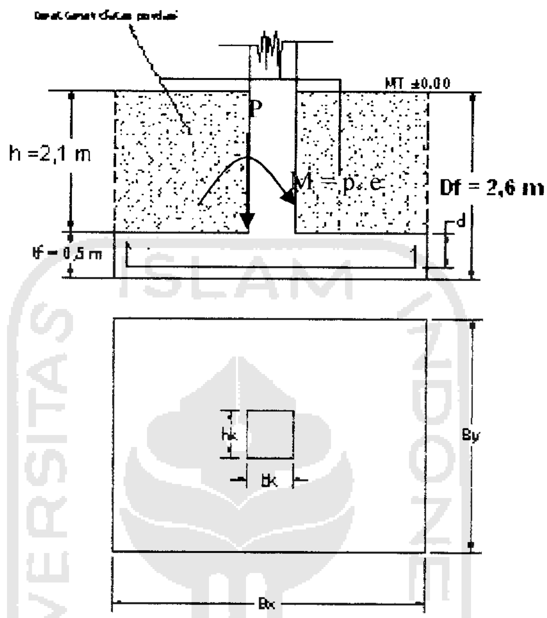
$$\text{Jumlah lapis sengkang} = \frac{522{,}896}{157} = 3{,}331 \text{ lapis}$$

digunakan sengkang □ 4P10

4.6 Perencanaan Pondasi

4.6.1 Perencanaan Pondasi Telapak Setempat (PS)

A. Perencanaan Dimensi Pondasi



Gambar 4.33 Pondasi telapak setempat

σ tanah = 700 KN/m^2

F'c = 25 Mpa

fy = 400 Mpa

P = 1877,391 KN

Mx tetap = 7,223 KNm

My tetap = 1,369 KNm

Mx sementara = 91,636 KNm

My sementara = 125,517 KNm

γbtanah = 16,254 KN/m^3

γbeton = 24 KN/m^3

Asumsi tebal pelat (tf) = 500 mm

Ukuran kolom :

hk = 600 mm

bk = 450 mm

$$\sigma_{netto\ tanah} = \sigma_{tanah} - \Sigma(h.\gamma_{beton}) - \Sigma(h.\gamma'_{tanah}) \qquad \textbf{(3.7.6)}$$

$$= 700 - (0,5.24) - (2,1.16,254)$$

$$= 653,867 \text{ KN/m}^2$$

1. Tinjauan Terhadap Beban Tetap

Digunakan pondasi penampang bujur sangkar, dicoba dengan nilai Bx =By=1,8 m

Luas penampang pelat pondasi :

$$A = Bx\,.\,By = 1,8 \times 1,8 = 3,24 \text{ m}^2$$

Kontrol luas pelat pondasi dan tegangan yang terjadi :

$$\sigma_{terjadi} = \frac{P}{A} \pm \frac{6.My}{Bx^2.By} \pm \frac{6.Mx}{By^2.Bx} \qquad \textbf{(3.7.1)}$$

$$\sigma_{terjadi\ max} = \frac{1877,391}{3,24} + \frac{6.1,369}{1,8^2.1,8} + \frac{6.7,223}{1,8^2.1,8}$$

$$= 588,281 \text{ KN/m}^2 < \sigma_{nettotanah} = 653,867 \text{ KN/ m}^2 \qquad \textbf{..........Ok !}$$

$$\sigma_{terjadi\ min} = \frac{1877,391}{3,24} - \frac{6.1,369}{1,8^2.1,8} - \frac{6.7,223}{1,8^2.1,8}$$

$$= 570,602 \text{ KN/m}^2 > 0 \qquad \textbf{..........Ok!}$$

2. Tinjauan Terhadap Beban Sementara

Eksentrisitas yang terjadi :

$$ex = \frac{My}{P} = \frac{1,369}{1877,391} = 0,00073\text{m}$$

$$ey = \frac{Mx}{P} = \frac{7,223}{1877,391} = 0,00385 \text{ m}$$

$B/6 = 1,8/6 = 0,3 >$ ex dan ey (beban eksentrisitas di dalam teras), maka :

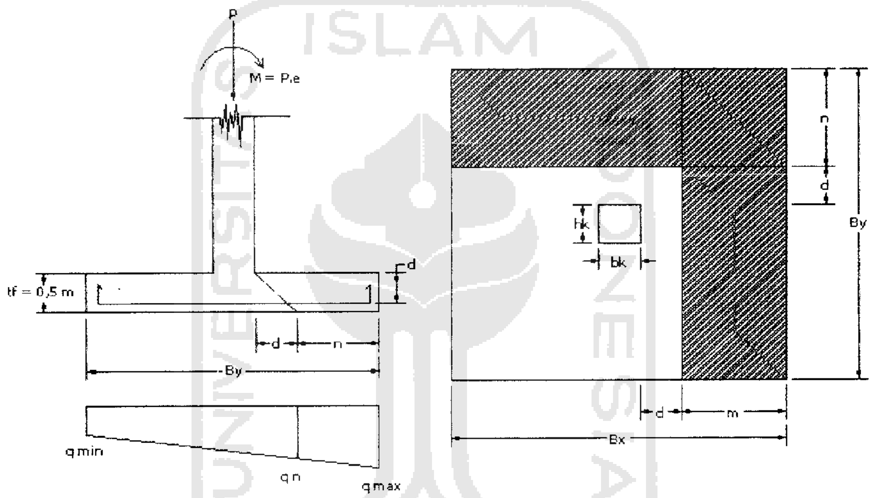
Kontrol tegangan yang terjadi :

$$\sigma_{terjadi} = \frac{P}{A}\left(1+\frac{6.ex}{Bx}+\frac{6.ey}{By}\right) \quad \textbf{(3.7.1)}$$

$$= \frac{1877,391}{3,24}\left(1+\frac{6.0,00073}{1,8}+\frac{6.0,00385}{1,8}\right)$$

$= 588,230$ KN/m^2 $< 1,5.\sigma_{netto} = 1,5.\ 653,867 = 980,799$ KN/m^2 **.....Ok !**

B. Perencananaan Geser Satu Arah



Gambar 4.34 Pondasi dengan geser satu arah

→ Ditinjau pada arah momen terbesar.

P = 1877,391 KN

Mx = 7,223 KNm

My = 1,369 KNm

Jarak pusat tulangan tarik ke serat tekan beton :

$d = tf - Pb - \frac{1}{2}.\varnothing_{tul.\ pokok} = 500 - 70 - \frac{1}{2}.\ 19 = 420,5$ mm $= 0,400$ m

$$m = \frac{Bx - bk - 2.d}{2} = \frac{1,8 - 0,45 - 2.0,40}{2} = 0,275 \text{ m} \quad \textbf{(3.7.18)}$$

$$n = \frac{By - hk - 2.d}{2} = \frac{1{,}8 - 0{,}6 - 2.0{,}40}{2} = 0{,}2 \text{ m} \qquad \textbf{(3.7.20)}$$

berdasarkan momen yang terbesar yaitu Mx = 7,223 KNm, maka geser yang ditinjau adalah arah y.

- Tegangan kontak yang terjadi :

$$q_{\text{terjadi}}\, y = \frac{P}{A} \pm \frac{6.Mx}{By^2.Bx}$$

$$= \frac{1877{,}391}{3{,}24} \pm \frac{6.7{,}223}{1{,}8^2.1{,}8}$$

$q_{tjd}\, y_{mak}$ = 586,873 KN/m^2

$q_{tjd}\, y_{min}$ = 572,011 KN/m^2

$$q_{tjd}\, n = \frac{(q_{tjd}\, y_{mak} - q_{tjd}\, y_{min}).(By - n)}{By} + q_{tjd}\, y_{min}$$

$$= \frac{(586{,}873 - 572{,}011).(1{,}8 - 0{,}2)}{1{,}8} + 572{,}011$$

= 585,222 KN/m^2

$q_{tjd}\, n$ = ½. ($q_{tjd}\, x_{mak}$ + $q_{tjd}\, x_{min}$) = ½. (586,873 + 572,011)

= 579,442 KN/m^2

Jadi $q_{tjd}\, n$ = 585,222 KN/m^2

- Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

Vu = $q_{tjd}\, n$. n . Bx = 585,222. 0,2 . 1,8 = 210,680 KN

$$Vu/\phi = 210{,}680/0{,}6 = 351{,}133 \text{ KN}$$

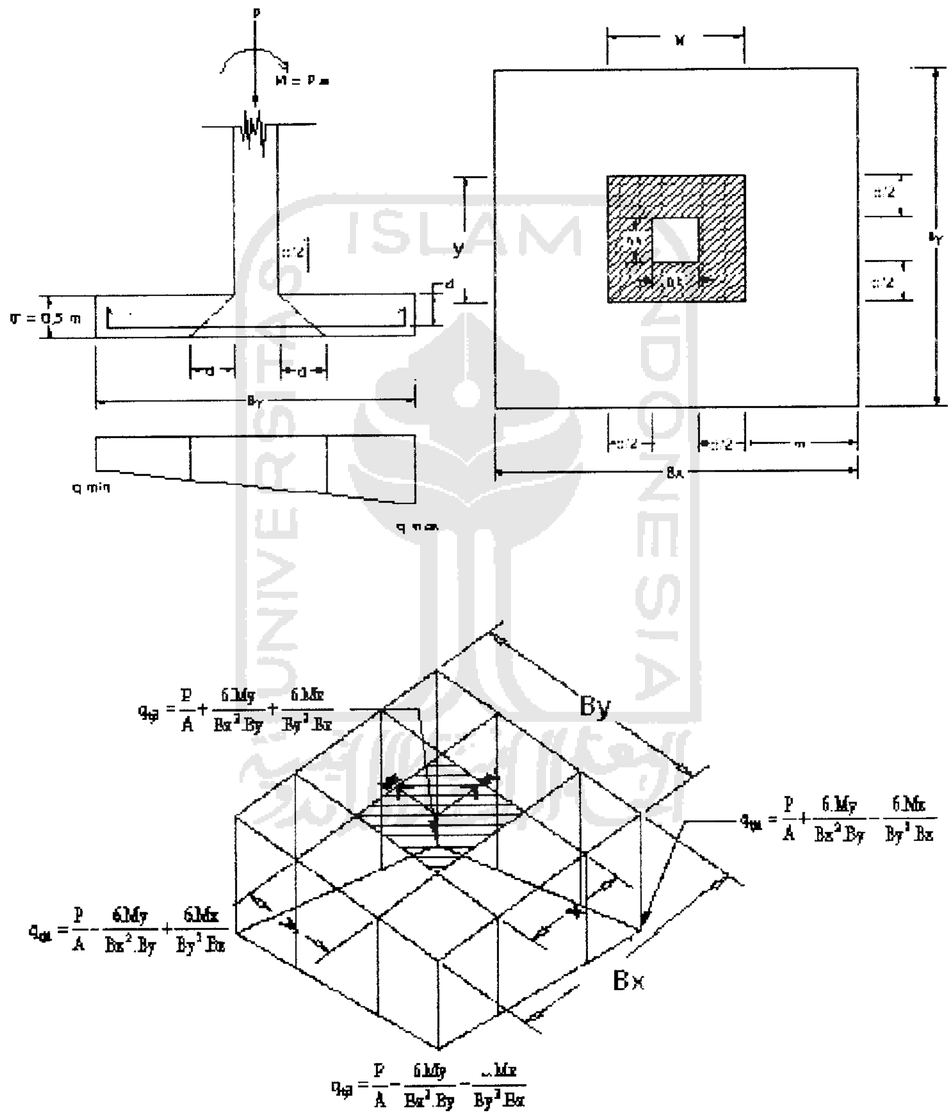
- Kekuatan beton menahan geser:

$$Vc = 1/6.\ \sqrt{f'c}\ .\ Bx.\ d = 1/6.\ \sqrt{25}\ 1{,}8.\ 0{,}400\ .\ 10^3 = 600 \text{ KN} \qquad \textbf{(3.7.21)}$$

• Kontrol gaya geser :

$$V_c = 600 \text{ KN} \geq {V_u}/{\phi} = 351{,}133 \text{ KN} \textbf{ ....Ok !}$$

C. Perencananaan Geser Dua Arah



Gambar 4.35 Pondasi dengan geser dua arah

$x = bk + d$ **(3.7.24)**

$= 450 + 400$

$= 850 \text{ mm} = 0{,}850 \text{ m}$

$$y \ = hk + d \tag{3.7.25}$$

$$= 600 + 400$$

$$= 1000 \text{ mm} = 1{,}0 \text{ m}$$

- Tegangan kontak yang terjadi :

$$q_{tjd} = \frac{P}{A} \pm \frac{6.My}{Bx^2.By} \pm \frac{6.Mx}{By^2.Bx}$$

$$= \frac{1877{,}391}{3{,}24} \pm \frac{6.1{,}369}{1{,}8^2.1{,}8} \pm \frac{6.7{,}223}{1{,}8^2.1{,}8}$$

$q_{tjd\,max} = 588{,}281 \text{ KN/m}^2$

$q_{tjd\,min} = 570{,}602 \text{ KN/m}^2$

$q_{tjd\,pakai} = ½ (q_{tjd\,max} + q_{tjd\,min}) = ½ (588{,}281 + 570{,}602) = 579{,}442 \text{ KN/m}^2$

- Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

$$Vu \ = q_{tjd\,pakai}.\ ((Bx.By) - (x.y)) \tag{3.7.23}$$

$$=579{,}442.\ ((1{,}8\ .\ 1{,}8) - (0{,}850.1{,}0)) = 1384{,}866 \text{ KN}$$

$$Vu/\phi = 1384{,}866/0{,}6 = 2308{,}111 \text{ KN}$$

- Kekuatan beton menahan geser :

$$\beta c \ = \frac{sisipanjang}{sisipendek} = \frac{y}{x} = \frac{1}{0{,}85} = 1{,}176$$

$$bo \ =2.(x+y) = 2\ .\ (850 + 1000) = 3600 \text{ mm} \tag{3.7.28}$$

$$Vc_1 \ = (1 + 2/\beta c).(2.\sqrt{f'c}).\ bo.\ d \tag{3.7.27}$$

$$= (1 + 2/1{,}176).\ (2.\sqrt{25}).\ 3600.\ 0{,}40.\ 10^3 = 38890 \text{ KN}$$

$$Vc_2 \ = 4.\sqrt{f'c}.bo.d= 4.\ \sqrt{25}\ .\ 3600.\ 0{,}400.10^3 = 28800 \text{ KN} \tag{3.7.26}$$

$$Vc = 38890 \text{ KN} \geq {}^{Vu}\!/_{\phi} = 2308{,}111 \text{ KN}$$ ....**Ok !**

D. Kuat Tumpuan Pondasi

• Kuat tumpuan Pondasi :

$$\phi.Pn = \phi.\ (0{,}85.\ f'c.\ A_1.\ \sqrt{A_2/A_1}\ )$$

Luas penampang kolom (A_1) = bk . hk = 0,45. 0,60 = 0,27 m^2

Luas pelat pondasi (A_2) = Bx . By = 1,8 . 1,8 = 3,24 m^2

$$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{3{,}24}{0{,}27}} = 3{,}464 > 2$$ (jika lebih besar dari 2, dipakai nilai 2)

$$\phi.Pn = \phi.\ (0{,}85.\ f'c.\ A_1.2)$$

$$= 0{,}7.\ (0{,}85\ .\ 25\ .\ 0{,}27\ .\ 2).10^3 = 8032{,}5 \text{ KN}$$

• Kuat tumpuan kolom :

$$\phi.Pn = \phi.\ (0{,}85.\ f'c.\ A_1)$$

$$= 0{,}7.\ (0{,}85.\ 25.\ 0{,}27).10^3 = 4016{,}25 \text{ KN}$$

• Kontrol kuat tumpuan :

$\phi.Pn_{pondasi}$ = 8032,5 KN > $\phi.Pn_{\ kolom}$ = 4016,25 KN ....**Ok !**

E. Perencanaan Tulangan Lentur Pondasi

Karena penampang pondasi berbentuk bujur sangkar, sehingga arah x dan arah y sama panjang, maka perencanaan tulangan lenturnya dianggap sama.

$$L = \frac{Bx - bk}{2} = \frac{1{,}8 - 0{,}45}{2} = 0{,}675 \text{ m} \qquad \textbf{(3.7.29)}$$

$q_{terjadi}$ = 588,281 KN/m^2

$$Mu = 0{,}5.\ q_{terjadi}.\ L^2 = 0{,}5\ .\ 588{,}281\ .\ 0{,}675^2 = 134{,}018 \text{ KNm} \qquad \textbf{(3.7.30)}$$

$$Mu/\phi = \frac{134{,}018}{0{,}8} = 167{,}523 \text{ KNm}$$

- Digunakan tulangan pokok $\varnothing_{19}$ mm, maka : $A_{1\varnothing}$ = 283,529 mm^2
- Tebal pelat pondasi tf = 500 mm, selimut beton (Pb) = 70 mm

d = tf – Pb – 0,5. $\varnothing_{\text{tul. pokok}}$ = 500 – 70 – 0,5. 19 = 421 mm

$$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.25} = 18{,}82 \qquad \textbf{(3.7.33)}$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm :

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{167{,}523.10^6}{1000.421^2} = 0{,}945 \text{ MPa} \qquad \textbf{(3.7.34)}$$

Rasio Tulangan :

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

$$\rho b = \frac{0{,}85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25.0{,}85}{400}.\left(\frac{600}{600+400}\right) = 0{,}0271$$

$$\rho_{mak} = 0{,}75.\ \rho b = 0{,}75.\ 0{,}0271 = 0{,}0203$$

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

$$= \frac{1}{18{,}82}.\left(1-\sqrt{1-\frac{2.18{,}82.0{,}945}{400}}\right) = 0{,}00242 < \rho_{mak} = 0{,}0203$$

$$< \rho_{min} = 0{,}0035$$

$1{,}33.\rho_{ada} = 0{,}00322 < \rho_{min}$,maka : $\rho_{perlu} = 0{,}00322$

$As_{perlu} = \rho_{perlu}.\ b\ .\ d = 0{,}00322\ .\ 1000\ .\ 421 = 1355{,}62$ mm^2

$0{,}002.b.h = 0{,}002.1000.500 = 1000\text{mm}^2 < As_{perlu}$, maka, $As_{perlu} = 1355{,}62$ mm^2

Jarak antar tulangan :

$$s \leq \frac{A_{01}.b}{As_{perlu}} = \frac{283{,}529.1000}{1355{,}62} = 209{,}151 \text{ mm}$$

$$s \leq 2.h = 2.500 = 1000 \text{ mm}$$

$$s \leq 250 \text{ mm}$$

→ **Dipakai Tulangan Pokok : D_{19} – 200 mm**

$$As_{ada} = \frac{A_{10}.1000}{s} = \frac{283{,}529.1000}{200} = 1417{,}645 \text{ mm}^2$$

- Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{1417{,}645.400}{0{,}85.25.1000} = 26{,}685 \text{ mm}$$

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

$$= 1417{,}645 \,.\, 400\,(421 - 26{,}685/2)$$

$$= 231{,}165 \text{ KNm} \geq Mu/\phi = 167{,}523 \text{ KNm}$$ **.........Ok !**

<u>Perencanaan Tulangan Bagi Pondasi</u>

$$As_{bagi} = 0{,}002.\,b.\,h = 0{,}002.\,1000.\,500 = 1000 \text{ mm}^2$$

- Digunakan tulangan bagi ∅12 mm, maka: $A_{1\varnothing} = 113{,}097 \text{ mm}^2$

Jarak antar tulangan susut :

$$s \leq \frac{A_{\theta 1}.b}{As_{susut}} = \frac{113{,}097.1000}{1000} = 113{,}097 \text{ mm} \approx 110 \text{ mm}$$

→ **Dipakai Tulangan Susut : P_{12} – 110 mm**

4.6.2 Perencanaan Pondasi Sumuran

σ tanah = 400 KN/m^2

F'c = 25 Mpa

fy = 400 Mpa

P = 1877,391 KN

Mx tetap = 7,223 KNm

My tetap = 1,369 KNm

γbtanah = 16,254 KN/m^3

γbeton = 24 KN/m^3

Asumsi diameter sumuran (Ø) = 3 m

bj beton siklop = 23 KN/ m^3

h sumuran = 3,6 m

A. Perencanaan Pondasi Sumuran

P pondasi sumuran $= 0,25 \cdot \pi \cdot Ø^2 \cdot h \cdot \text{bj btn siklop}$ **(3.7.43)**

$= 0,25 \cdot \pi \cdot 3^2 \cdot 3,6 \cdot 23$

$= 585,279$ KN

P total $= P + P$ pondasi sumuran **(3.7.44)**

$= 1877,391 + 585,279$

$= 2462,6697$ KN

σ ijin di bawah sumuran = 400 KN/m^2

σ netto tanah $= \sigma_{tanah} - \Sigma(h \cdot \gamma_{beton}) - \Sigma(h \cdot \gamma'_{tanah})$

$= 400 - 3,6 \cdot 23 - 2,1 \cdot 16,254 - 0,5 \cdot 24$

$= 361,867$ KN/m^2

$$\text{Aperlu} = \frac{Ptotal}{\sigma tnh} = \frac{2462,6697}{361,867} = 9,08 \text{ m}^2$$ **(3.7.47)**

$$\text{Diameter sumuran perlu} = \sqrt{\frac{Aperlu}{0,25.\pi}} = \sqrt{\frac{9,01}{0,25.\pi}} = 3,41 \text{ m} \approx 3,5 \text{ m}$$ **(3.7.48)**

4.5 Perencanaan Tangga

4.5.1 Spesifikasi Struktur

a. Tinggi antar lantai (h) = 3,5 m = 350 cm

b. Lebar bordes (Lb) = 2,0 m = 200 cm

c. Tinggi optrede rencana diambil 18 cm

jumlah optrede = 350/18 = 19,4 dipakai 20 buah **(3.8.1)**

tinggi optrede pakai (h'_o) = 350/20 = 17,5 cm **(3.8.2)**

jumlah antrede = 20 – 2 = 18 buah **(3.8.3)**

dipakai lebar antrede (La) = 30 cm

d. dimensi tangga :

tangga dibagi menjadi dua (2) bagian, sehingga panjang bentang tangga (Pt) :

Pt = (La x jumlah antrede/2) + Lb **(3.8.4)**

= (30 x 18/2) + 200

= 470 cm = 4,7 m

Lebar bersih tangga (Lt) = 0,5 (4,7 – 3.0,15) = 2,125 m = 212,5 cm **(3.8.8)**

e. Beban sandaran tangga :

tinggi sandaran = 1 m

tebal sandaran = 0,12 m

berat sandaran tangga = (0,12 . 1 . 24 . 2)/2,125 = 2,711 KN/m^2

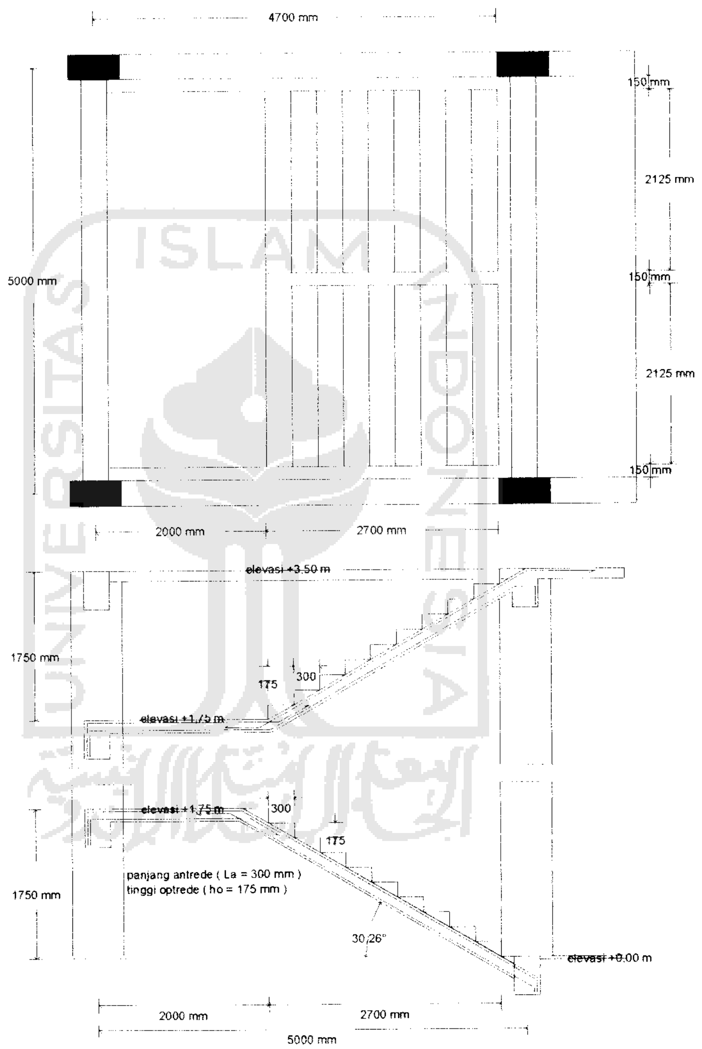
berat sandaran bordes = (0,12 . 1 . 24)/2,125 = 1,355 KN/m^2

f. Sudut kemiringan tangga (α) :

$$\alpha = \text{arc tgn}\frac{h'_o}{La} = arc\,tgn\frac{17{,}5}{30} = 30{,}26^\circ$$ **(3.8.6)**

g. Tebal pelat tangga dan bordes diambil = 15 cm

tebal pelat sisi miring (h') = $\dfrac{15}{\cos 30{,}26°}$ = 17,366 cm **(3.8.5)**



Gambar 4.36 Dimensi Tangga

4.5.2 Pembebanan

a. Pembebanan bordes

Beban mati :

- Berat sendiri pelat = 0,15 x 24 = 3,60 KN/m^2
- Berat spesi = 0,03 x 21 = 0,63 KN/m^2
- Berat keramik = 0,01 x 20 = 0,20 KN/m^2
- Berat sandaran = 1,355 KN/m^2 +

Q_D = 5,785 KN/m^2

untuk lebar 2,125 m, maka qD = 2,125 x 5,785 = 12,294 KN/m'

Beban hidup :Q_L = 300 kg/cm^2 = 3 KN/m^2

qL = 2,125 x 3 = 6,375 KN/m'

q_U = 1,2qD + 1,6qL = 1,2.12,294 + 1,6.6,375 = 24,953 KN/m'

b. Pembebanan tangga

Beban mati :

- Berat sendiri = (0,17366 + 0,175/2) x 24 = 6,268 KN/m^2
- Berat spesi = 0,03 x 21 = 0,63 KN/m^2
- Berat keramik = 0,01 x 20 = 0,20 KN/m^2
- Berat sandaran = 2,711 KN/m^2 +

Q_D = 9,809 KN/m^2

qD = 2,125 x 9,809 = 20,844 KN/m'

Beban hidup :Q_L = 300kg/cm^2 = 3 KN/m^2

qL = 2,125 x 3 = 6,375 KN/m'

qu = 1,2qD + 1,6qL = 1,2.20,844 + 1,6.6,375 = 35,213 KN/m

4.5.3 Penulangan Tangga

Perhitungan pelat bordes

Mu maks = 23,46 KNm

$$Mu/_{\phi} = \frac{23{,}46}{0{,}8} = 29{,}325 \text{ KNm}$$

Digunakan tulangan Ø13 mm, sehingga luas tampang 1 tulangan pokok :

$A_{1\emptyset} = ¼ . \pi . D^2 = ¼ . \pi . 13^2 = 132{,}732 \text{ mm}^2$

tebal pelat bordes (h) = 150 mm, selimut beton (Pb) = 20 mm, maka :

$d = h - Pb - 0{,}5.\emptyset_{tul\ pokok} = 150 - 20 - 0{,}5.13 = 123{,}5$ mm **(3.2.11)**

Rasio tulangan :

$$\rho b = \frac{0{,}85.f'c}{fy}\beta 1.\left(\frac{600}{600+fy}\right) = \frac{0{,}85.25}{400}0{,}85.\left(\frac{600}{600+400}\right) = 0{,}0271 \quad \textbf{(3.2.8)}$$

$\rho maks = 0{,}75.\rho b = 0{,}75 . 0{,}0271 = 0{,}0203$ **(3.2.9)**

$$\rho \min = \frac{1{,}4}{fy} = \frac{1{,}4}{400} = 0{,}0035 \quad \textbf{(3.2.10)}$$

Koefisien ketahanan (Rn) :

$$Rn = \frac{Mu/_{\phi}}{b.d^2} = \frac{29{,}325.10^6}{2125.123{,}5^2} = 0{,}905 \text{ Mpa} \quad \textbf{(3.2.13)}$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{400}{0{,}85.25} = 18{,}824 \quad \textbf{(3.2.14)}$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2.m.Rn}{fy}}\right) = \frac{1}{18{,}824}\left(1-\sqrt{1-\frac{2.18{,}824.0{,}905}{400}}\right) \quad \textbf{(3.2.15)}$$

$= 0{,}00231 < \rho \text{ maks} = 0{,}02302$

$< \rho\min = 0{,}0035$

$1{,}33 \,.\, \rho = 1{,}33 \,.\, 0{,}00231 = 0{,}00308 < \rho_{min} = 0{,}0035$

sehingga $\rho_{pakai} = \rho_{min} = 0{,}00308$

$$As = \rho_{pakai} \,.\, b \,.\, d \geq 0{,}002 \,.\, b \,.\, h \qquad \textbf{(3.2.16)}$$

$$= 0{,}00308 \,.\, 2125 \,.\, 123{,}5 \geq 0{,}002 \,.\, 2125 \,.\, 150$$

$$= 807{,}084 \text{ mm}^2 > 637{,}5 \text{ mm}^2 \quad \text{(Ok!)}$$

$$\text{jarak tulangan (s)} = \frac{A_1 \phi . b}{As} = \frac{132{,}732.2125}{807{,}084} = 349{,}476 \text{ mm} \qquad \textbf{(3.2.17)}$$

$$\leq 2h \quad = 2 \,.\, 150 \quad = 300 \text{ mm}$$

$$\leq 250$$

diambil terkecil s = 250 mm

Dipakai tulangan pokok : D13 – 250 mm

Kontrol kapasitas lentur pelat tangga yang terjadi :

$$As_{ada} = \frac{A_1 \phi . b}{s} = \frac{132{,}732.2125}{340} = 829{,}577 \text{ mm}^2 \qquad \textbf{(3.2.20)}$$

$$a = \frac{As_{ada} . fy}{0{,}85 . f'c . b} = \frac{829{,}577.400}{0{,}85.25.2125} = 7{,}349 \text{ mm} \qquad \textbf{(3.2.21)}$$

$$Mn = As_{ada} \,.\, fy \left(d - \frac{a}{2}\right) \geq \frac{Mu}{\phi} \qquad \textbf{(3.2.22)}$$

$$= 829{,}577 \,.\, 400 \left(123{,}5 - \frac{7{,}349}{2}\right)$$

$$= 39{,}762 \text{ KNm} > 29{,}325 \text{ KNm} \; \ldots\ldots\ldots\ldots\; \text{(Ok!)}$$

Tulangan Bagi Pelat Bordes

$As_{bagi} = 0{,}002 \;.\; b \;.\; h = 0{,}002 \;.\; 2125 \;.\; 150 = 637{,}5\ mm^2$

Dipakai tulangan bagi Ø8, maka $A_1Ø = 50{,}265\ mm^2$

$$\text{jarak tulangan bagi (s)} = \frac{A_1\phi.b}{As_{bagi}} = \frac{50{,}265.2125}{637{,}5} = 167{,}55\ mm$$

$$\leq 2h \quad = 2 \;.\; 150 \quad = 300\ mm$$

$$\leq 250$$

diambil terkecil s = 160 mm

Dipakai tulangan bagi : P8 – 160 mm

4.7.4 Perencanaan Balok Bordes

Dimensi rencana balok

Tinggi (h) = 400 mm

Lebar (b) = 300 mm

Tinggi efektif (d) = 400 – 70 (dianggap tul. sebelah 1 lapis)

= 330 mm

Pembebanan :

- beban akibat tangga (33,33/2,125)= 15,685 KN/m
- berat sendiri = 1,2.0,3.0,4.24 = 3,456 KN/m +

qu = 19,141 KN/m

Momen tumpuan :

$$Mu = -\frac{1}{16} qu \cdot L^2 = -\frac{1}{16} 19{,}141 \cdot 5^2 = -29{,}907 \text{ KNm}$$

Momen lapangan :

$$Mu = \frac{1}{11} qu \cdot L^2 = \frac{1}{11} 19{,}141 \cdot 5^2 = 43{,}502 \text{ KNm}$$

a.Perencanaan tulangan lentur balok bordes

Tulangan lapangan

$$\frac{Mu}{\phi} = \frac{43{,}502}{0{,}8} = 54{,}377 \text{ KNm}$$

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

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

rasio tulangan rencana = 0,5. ρ_{maks} = 0,5.0,0203 = 0,01015

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

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

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

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

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{54,377.10^6}{3,675.300}} = 222,071 \text{ mm} < d = 330 \text{ mm}$$, maka dipakai

tulangan sebelah

$$Rn_{ada} = \frac{Mu/\phi}{b.d^2} = \frac{54,377.10^6}{300.330^2} = 1,664 \text{ Mpa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}.\rho = \frac{1,664}{3,675}.0,01015 = 0,0046 > \rho_{min} = 0,0035$$

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

$As_{perlu} = \rho_{ada}.b.d = 0,0046.300.330 = 455,505 \text{ mm}^2$

Dipakai tulangan Ø16 dengan A1Ø = 201,062 mm^2

$$\text{jumlah tulangan (n)} = \frac{As_{perlu}}{A1\phi} = \frac{455,505}{201,062} = 2,265 \text{ batang}$$

dipakai 3D16, maka $As_{ada} = 3 \,.\, 201,062 = 603,186 \text{ mm}^2 > As_{perlu}$

Kontrol kapasitas lentur yang terjadi :

$$a = \frac{As_{ada}.fy}{0,85.fc'.b} = \frac{603,186.400}{0,85.25.300} = 37,847 \text{ mm}$$

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

$$= 603{,}186.400.(330 - 37{,}847/2)$$

$$= 75{,}055 \text{ KNm} > Mu/\phi = 57{,}162 \text{ KNm} \rightarrow \text{ OK!}$$

b.perencanaan tulangan geser balok bordes

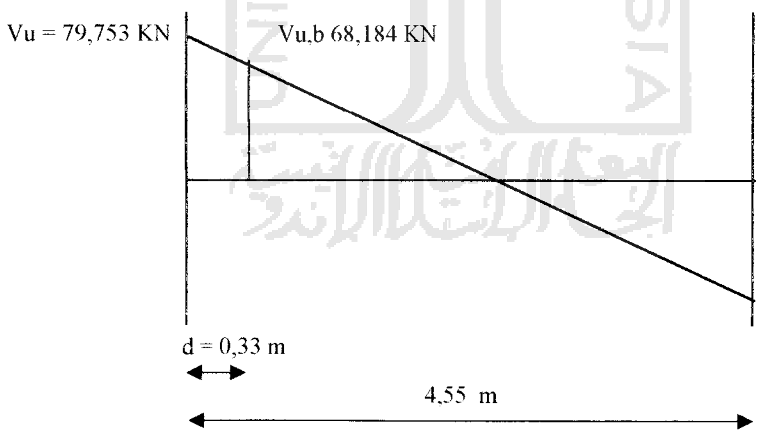
Gaya geser dukungan

$$\text{Vu dukungan} = \frac{1}{2}\,qu\,.\,L$$

$$= \frac{1}{2}\,19{,}141.\,5$$

$$= 47{,}852 \text{ KN}$$

maka $\dfrac{Vu}{\Phi} = \dfrac{47{,}852}{0{,}6} = 79{,}753$ KN



$$\text{Vu pakai} = \left(\frac{2{,}275 - 0{,}33}{2{,}275}\right) 79{,}753 = 68{,}184 \text{ KN}$$

Tegangan geser beton (Vc) :

$$Vc = 1/6.\sqrt{fc'}.b.d = 1/6.\sqrt{25}.300.330 = 82{,}5 \text{ KN}$$

Vu pakai = 68,184 KN > Ø .Vc = 49,5 KN, maka

$Ø(3.Vc) = 0{,}6(3.82{,}5) = 148{,}5$ KN

$Ø.Vc < Vu \leq Ø(3.Vc) = 49{,}5 < 68{,}184 \leq 148{,}5$

$$Vs = \frac{Vu}{\phi} - Vc = 68{,}184 - 82{,}5 = -14{,}314 \text{ KN}$$

$$Vs\ min = \frac{1}{3} . b . d = \frac{1}{3} . 300 . 330 = 33000 \text{ N} = 33 \text{ KN}$$

Maka diambil Vs min = 33 KN

Digunakan sengkang □ P10 mm, maka : $Av = 2 . ¼ . \pi . 10^2 = 157 \text{ mm}^2$

$$\text{Jadi ; } s \leq \frac{Av.fy.d}{Vs} = \frac{157.240.330}{33} . 10^{-3} = 376{,}8 \text{ mm}$$

$$\leq d/2 = 330/2 = 165 \text{ mm}$$

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

Jadi dipakai tulangan geser **P10 – 165 mm**