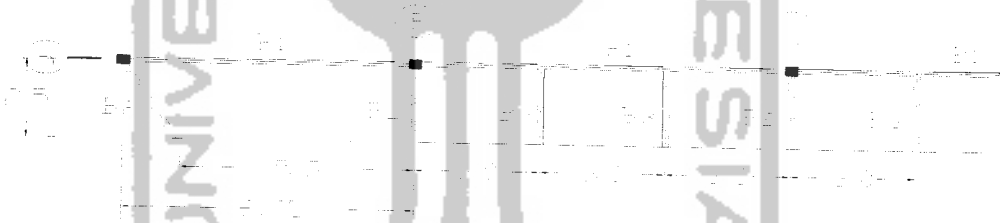
BAB V

ANALISIS DAN DESAIN STRUKTUR

5.1 Perencanaan Awal (Preliminari Design)

Perencanaan awal (preliminary design) merupakan perkiraan penampang balok dan kolom yang ditentukan oleh panjang bentang antara dua dukungan. Gambar denah pola balok lantai (elevasi lantai +2.500) ditunjukkan gambar 5.1 dibawah ini.



Gambar 5.1 Denah Pola Balok (Elevasi Lantai +2.500)

- Perkiraan penampang balok B_1 (Cantilever)

$$d = \frac{1}{12} \times L = \frac{1}{12} \times 9 = 0{,}75 \text{ m} = 750 \text{ mm}$$

Pakai W30 × 99 (d = 753,11 mm ; bf = 265,43 mm)

Berat = 1,444 KN/m

- Perkiraan penampang balok B_2 (Cantilever)

Pakai W21 × 57 (d = 534,924 mm ; bf = 166,497 mm)

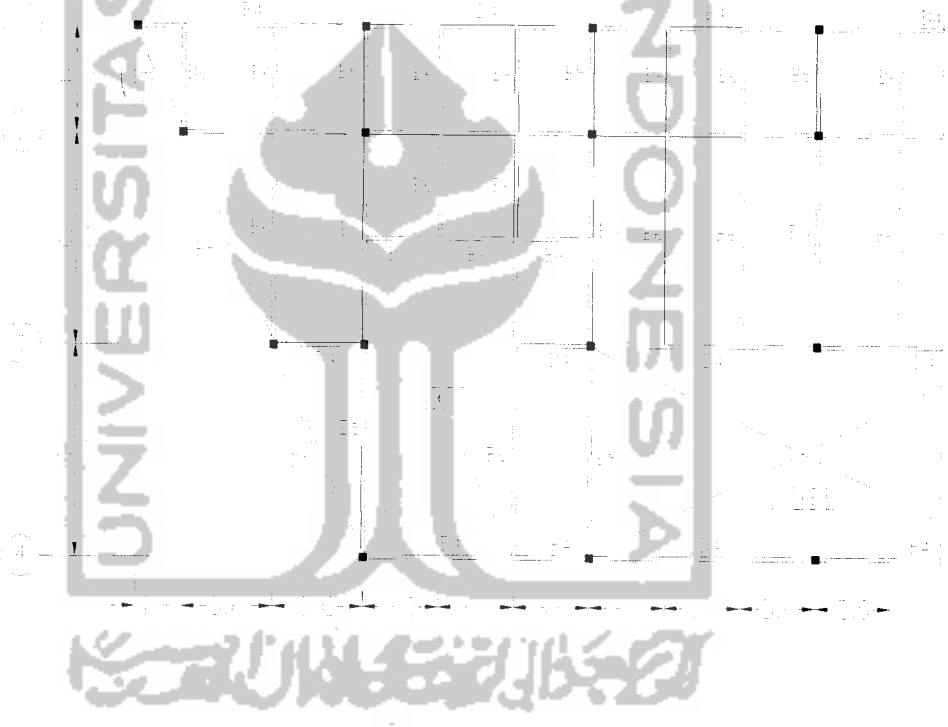
Berat = 0,832 KN/m

- Perkiraan penampang balok B_3 (Cantilever)

  W 18 × 55 (d = 459,99 mm ; bf = 191,262 mm)

  Berat = 0,803 KN/m

Gambar denah pola balok lantai (elevasi lantai +5.000) ditunjukkan gambar 5.2 dibawah ini.



Gambar 5.2 Denah Pola Balok (Elevasi Lantai +5.000)

- Perkiraan penampang balok B_4

$$d = \frac{1}{12} \times L = \frac{1}{12} \times 9 = 0{,}75 \text{ m} = 750 \text{ mm}$$

Pakai W30 × 99 (d = 753,11 mm ; bf = 265,43 mm)

Berat = 1,444 KN/m

- Perkiraan penampang balok B_5

$$d = \frac{1}{12} \times L = \frac{1}{12} \times 8 = 0{,}667 \text{ m} = 667 \text{ mm}$$

Pakai W27 × 84 (d = 678,434 mm ; bf = 252,984 mm)

Berat = 1,226 KN/m

- Perkiraan penampang balok B_6

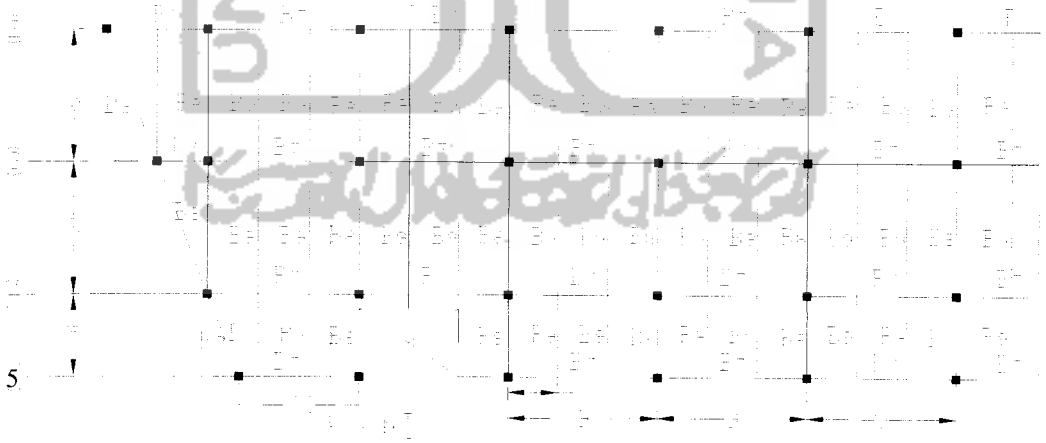
$$d = \frac{1}{15} \times L = \frac{1}{15} \times 8 = 0{,}533 \text{ m} = 533 \text{ mm}$$

Pakai W21 × 57 (d = 534,924 mm ; bf = 166,497 mm)

Berat = 0,832 KN/m

Perkiraan dimensi balok miring

Gambar pola balok miring ditunjukkan gambar 5.3 dibawah ini.



Gambar 5.3 Denah Pola Balok miring

- Perkiraan penampang balok B_7

$$d = \frac{1}{12} \times L = \frac{1}{12} \times 9 = 0{,}75 \text{ m} = 750 \text{ mm}$$

Pakai W30 × 99 (d = 753,11 mm ; bf = 265,43 mm)

Berat = 1,444 KN/m

- Perkiraan penampang balok B_8

Panjang balok miring (L) = $\frac{8}{Cos.27^0} = 8{,}98$ m

$$d = \frac{1}{12} \times L = \frac{1}{12} \times 8{,}98 = 0{,}748 \text{ m} = 748 \text{ mm}$$

Pakai W30 × 99 (d = 753,11 mm ; bf = 265,43 mm)

Berat = 1,444 KN/m

- Perkiraan penampang balok B_9

$$d = \frac{1}{15} \times L = \frac{1}{15} \times 8{,}98 = 0{,}599 \text{ m} = 599 \text{ mm}$$

Pakai W24 × 55 (d = 598,678 mm ; bf = 177,927 mm)

Berat = 0,803 KN/m

Perkiraan dimensi kolom W 18 × 76 (d = 462 mm ; bf = 280,289 mm)

Berat = 1,131 KN/m

5.2 Pembebanan

Pembebanan merupakan langkah paling awal dalam mengerjakan hitungan struktur. Beban-beban yang dihitung, kemudian ditransfer ke balok portal sehingga mendapatkan gaya-gaya dalam yang dipergunakan untuk

menghitung dimensi profil struktur. Beban-beban yang bekerja dibagi menjadi tiga kelompok, yaitu beban mati, beban hidup dan beban gempa. Untuk selanjutnya beban mati dan beban hidup disebut dengan beban gravitasi.

5.2.1 Beban Gravitasi

Beban gravitasi merupakan beban yang terdiri dari beban mati dan beban hidup. Perhitungan berat pelat lantai dan pelat tribun pada struktur stadion sleman ini adalah sebagai berikut :

Berat pelat lantai

Pelat = $0{,}12 \times 24$ = 2,88 KN/m^2

Pasir = $0{,}05 \times 16$ = 0,8 KN/m^2

Spesi = $0{,}03 \times 24$ = 0,72 KN/m^2

Keramik = $0{,}01 \times 24$ = 0,24 KN/m^2

W_D= 4,64 KN/m^2

Beban hidup pelat lantai **(W_L) = 5 KN/m^2**

Berat pelat tribun

Pelat = $\left[\left(\frac{0{,}12}{\cos 27°}\right)+\left(\frac{0{,}4}{2}\right)\right]\times 24$ = 8,36 KN/m^2

Kedap air = $0{,}03 \times 24$ = 0,72 KN/m^2

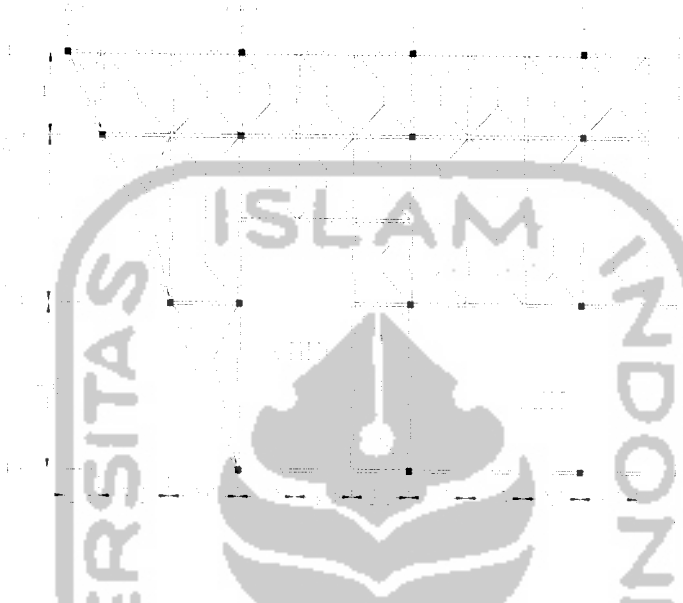
Spesi = $0{,}03 \times 24$ = 0,72 KN/m^2

W_D = 9,8 KN/m^2

Beban hidup pelat tribun **(W_L) = 5 KN/m^2**

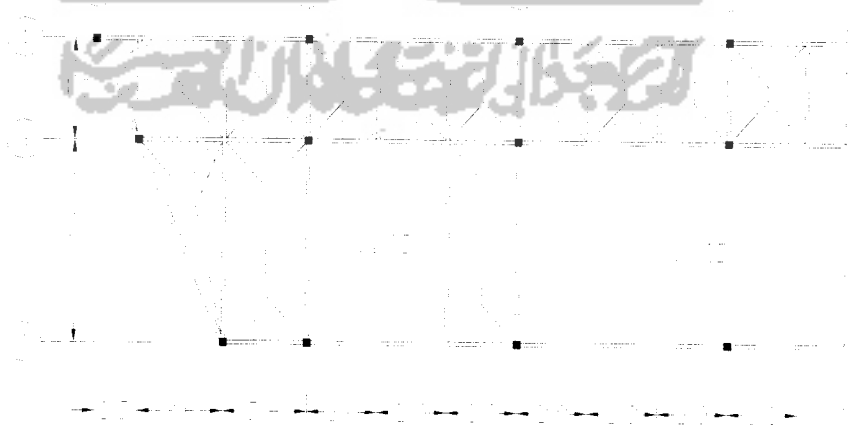
Pembebanan Balok Portal

Distribusi beban gravitasi dari pelat ke balok portal pada elevasi lantai +5.000 ditunjukkan gambar 5.4 dibawah ini.



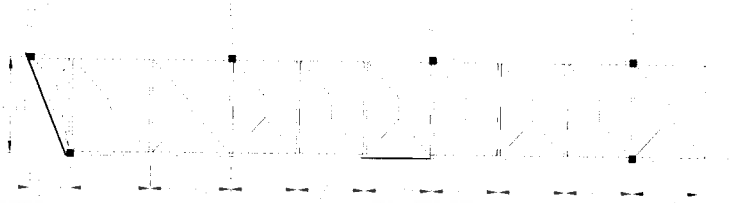
Gambar 5.4 Distribusi Beban (elevasi lantai +5.000)

Distribusi beban gravitasi dari pelat ke balok portal pada elevasi lantai +9.000 ditunjukkan gambar 5.5 dibawah ini.



Gambar 5.5 Distribusi Beban (elevasi lantai +9.000)

Distribusi beban gravitasi dari pelat ke balok portal pada elevasi lantai +13.000 ditunjukkan gambar 5.6 dibawah ini.

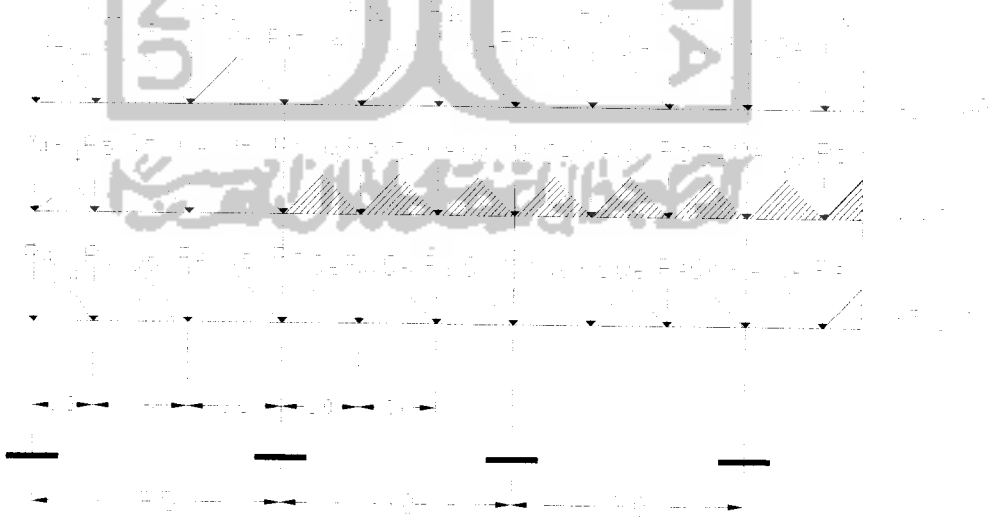


Gambar 5.6 Distribusi Beban (elevasi lantai +13.000)

Pembebanan Balok Portal

Pembebanan gravitasi dari pelat ke balok portal 1 ditunjukkan gambar 5.6 dibawah ini.

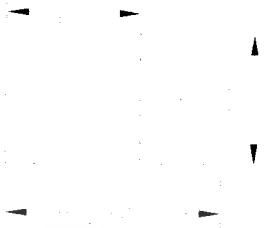
- **Portal 1**



Gambar 5.6 Pembebanan Balok Portal 1

Beban merata

- Tipe 4



Q_{D4} = 4,64 × 1,33 = 6,17 KN/m

Q_{L4} = 5 × 1,33 = 6,65 KN/m

- Tipe 5

Q_{D5} = 4,64 × 1,8 = 8,352 KN/m

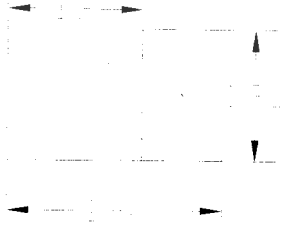
Q_{L5} = 5 × 1,8 = 9 KN/m

- Tipe 6

Q_{D6}= 4,64 × 1,5 = 6,96 KN/m

Q_{L6}= 5 × 1,5 = 7,5 KN/m

- Tipe 7



Q_{D7}= 4,64 × 1,33 = 5,746 KN/m

Q_{L7}= 5 × 1,33 = 6,65 KN/m

- Tipe 8

Q_{D8}= 4,64 × 1,8 = 7,776 KN/m

Q_{L8}= 5 × 1,8 = 9 KN/m

- Tipe 9

Q_{D9}= 4,64 × 1,5 = 6,48 KN/m

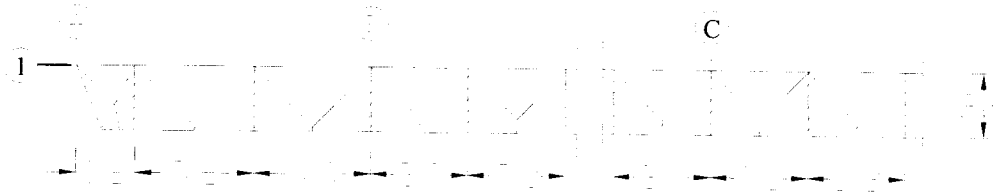
Q_{L9}= 5 × 1,5 = 7,5 KN/m

- Tipe 10

Q_{D9}= 4,64 × 1,56 = 6,48 KN/m

Q_{L9}= 5 × 1,56 = 7,5 KN/m

Beban titik



Gambar 5.7 Distribusi Beban Titik Portal 1

- Tipe 4

$A_1 = 0{,}5 \times 1{,}1 \times 0{,}7 = 0{,}385\ m^2$

$A_2 = \left[\frac{0{,}4 + 0{,}7}{2}\right].1{,}4 = 1{,}015\ m^2$

A total = 0,385 + 1,015 = 1,4 m^2

$P_{D4} = (4{,}64 \times 1{,}4) + (1{,}226 \times 2{,}2) = 9{,}19$ KN

$P_{L4} = (5 \times 1{,}4) = 7$ KN

- Tipe 5

$A_1 = 0{,}5 \times 0{,}6 \times 0{,}8 = 0{,}399\ m^2$

$A_2 = \left(\frac{0{,}2 + 2}{2}\right).1{,}8 = 1{,}98\ m^2$

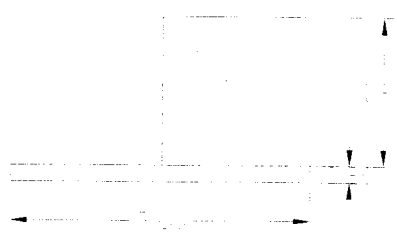
$A_3 = \left(\frac{0{,}5 + 0{,}6}{2}\right).1{,}4 = 0{,}77\ m^2$

A total = 0,399 + 1,98 + 0,77 = 3,15 m^2

$P_{D5} = (4{,}64 \times 3{,}15) + (0{,}832 \times 2) = 16{,}336$ KN

$P_{L5} = 5 \times 3{,}15 = 15{,}75$ KN

- Tipe 6



$A_1 = 0{,}5 \times 3{,}6 \times 1{,}8 = 3{,}24\ m^2$

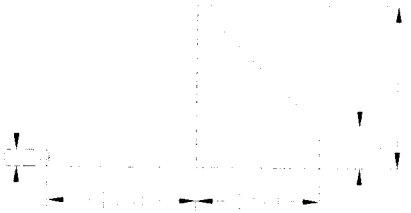
$A_2 = 3{,}6 \times 0{,}2 = 0{,}72\ m^2$

A total = 3,24 + 0,72 = 3,96 m^2

$P_{D6} = (4{,}64 \times 3{,}96) + (0{,}832 \times 2) = 20{,}04$ KN

$P_{L6} = 5 \times 3{,}96 = 19{,}8$ KN

- Tipe 7



$$A_1 = \left(\frac{0{,}5+2}{2}\right).1{,}5 = 1{,}875 \text{ m}^2$$

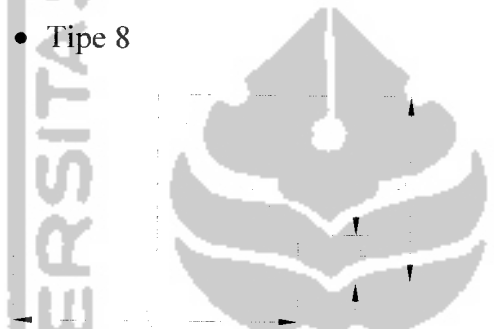
$$A_2 = \left(\frac{0{,}2+2}{2}\right).1{,}8 = 1{,}98 \text{ m}^2$$

A total = 1,875 + 1,98 = 3,855 m^2

$P_{D7} = (4{,}64 \times 3{,}855) + (1{,}226 \times 2) = 20{,}34$ KN

$P_{L7} = 5 \times 3{,}855 = 19{,}275$ KN

- Tipe 8



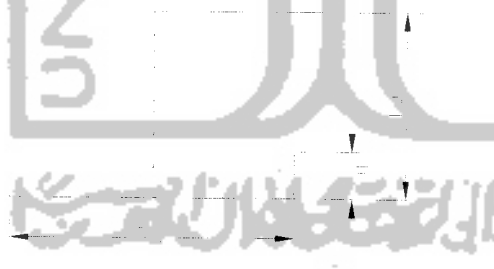
$$A = \left(\frac{0{,}5+2}{2}\right).1{,}5 \times 2 = 3{,}75 \text{ m}^2$$

$P_{D8} = (4{,}64 \times 3{,}75) + (0{,}832 \times 2)$

$= 20{,}34$ KN

$P_{L8} = (5 \times 3{,}75) = 18{,}75$ KN

- Tipe 9



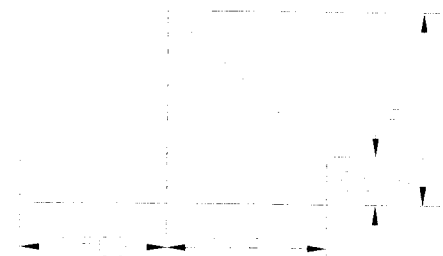
$$A = \left(\frac{0{,}5+2}{2}\right).1{,}5 \times 2 = 3{,}75 \text{ m}^2$$

$P_{D9} = (4{,}64 \times 3{,}75) + (1{,}226 \times 2)$

$= 19{,}852$ KN

$P_{L9} = (5 \times 3{,}75) = 18{,}75$ KN

- Tipe 10



$$A1 = \left(\frac{0{,}5+2}{2}\right).1{,}5 = 1{,}875 \text{ m}^2$$

$$A2 = \left(\frac{0{,}5+2}{2}\right).1{,}667 = 2{,}08 \text{ m}^2$$

A total = 1,875 + 2,08 = 3,96 m^2

$P_{D10} = (4{,}64 \times 3{,}96) + (1{,}226 \times 2) = 20{,}83$ KN

$P_{L10} = (5 \times 3{,}96) = 19{,}8$ KN

- Tipe 11

$$A = \left(\frac{0{,}5+2}{2}\right).1{,}667 \times 2$$

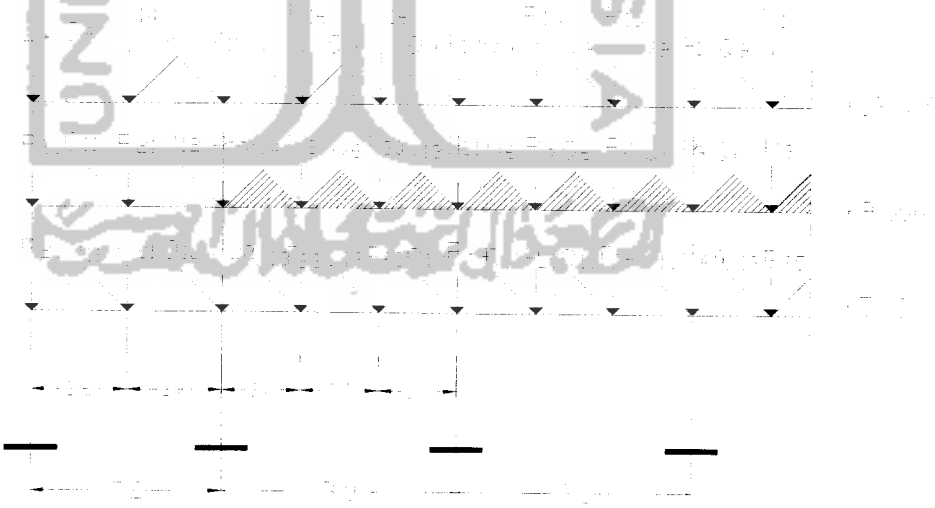
$$= 4{,}16 \text{ m}^2$$

$P_{D11} = (4{,}64 \times 4{,}16) + (0{,}832 \times 2) = 20{,}966$ KN

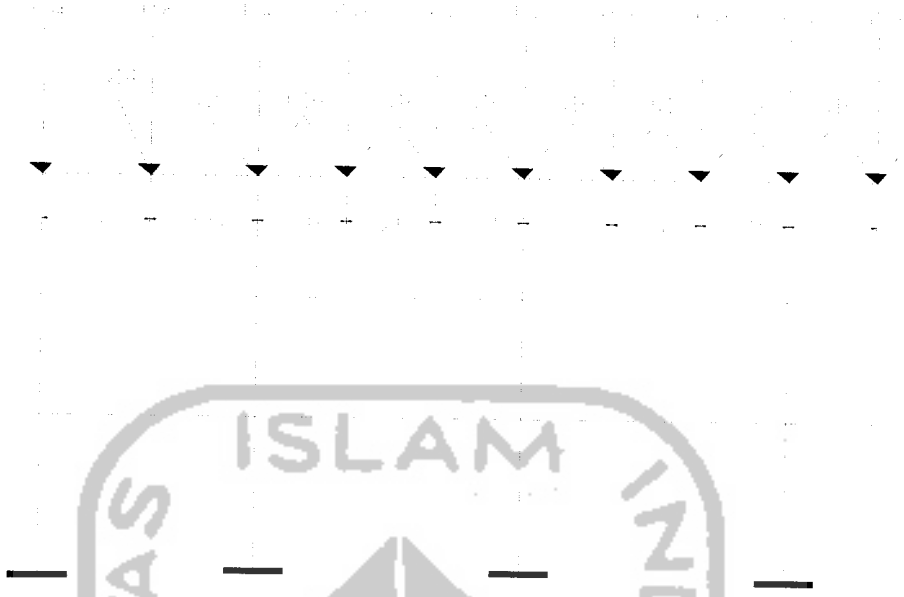
$P_{L11} = (5 \times 4{,}16) = 20{,}8$ KN

o **Portal 2**

Pembebanan gravitasi dari pelat ke balok portal 2 ditunjukkan gambar 5.9a dan 5.9b dibawah ini.



Gambar 5.9a Pembebanan Balok Portal 2 Akibat Pelat Lantai



Gambar 5.9b Pembebanan Balok Portal 2 Akibat Pelat Tribun

Beban merata

- Tipe 11

Q_{D11a} = 4,64 × 1,8 = 8,352 KN/m

Q_{D11b} = 4,64 × 2,67 = 12,39 KN/m

Q_{L11a} = 5 × 1,8 = 9 KN/m

Q_{L11b} = 5 × 2,67 = 13,35 KN/m

- Tipe 12

$Q_{D12} = 2 \times 4{,}64 \times 1{,}8 = 16{,}704$ KN/m

$Q_{L12} = 2 \times 5 \times 1{,}8 = 18$ KN/m

- Tipe 13

$Q_{D13} = 2 \times 1{,}5 \times 4{,}64 = 13{,}92$ KN/m

$Q_{L13} = 2 \times 1{,}5 \times 5 = 15$ KN/m

- Tipe 14

$Q_{D14} = 2{,}7 \times 9{,}8 = 26{,}46$ KN/m

$Q_{L14} = 2{,}7 \times 5 = 13{,}5$ KN/m

- Tipe 15

$Q_{D15} = 2 \times 9{,}8 = 19{,}6$ KN/m

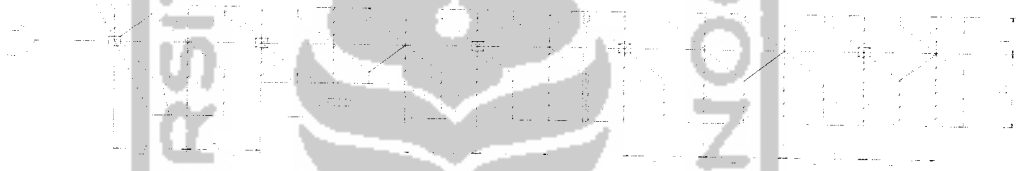
$Q_{L15} = 2 \times 5 = 10$ KN/m

- Tipe 16

$Q_{D16} = 2 \times 9{,}8 \quad = 19{,}6$ KN/m

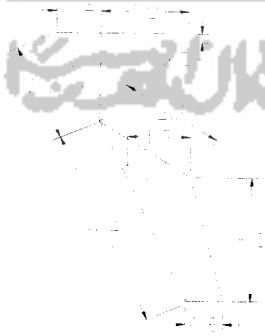
$Q_{L16} = 2 \times 5 \quad = 10$ KN/m

Beban titik



Gambar 5.10a Distribusi Beban Titik Portal 2 Elevasi +5.000

- Tipe 20



$$A_1 = \left(\frac{0{,}2 + 2}{2}\right).1{,}8 \quad = 1{,}98 \text{ m}^2$$

$$A_2 = 0{,}5 \times 0{,}9 \times 2 \quad = 0{,}9 \text{ m}^2$$

$$A_3 = 0{,}5 \times 1{,}3 \times 2{,}2 \quad = 1{,}43 \text{ m}^2$$

$$A_4 = \left(\frac{1{,}3 + 0{,}8}{2}\right).2{,}8 \quad = 2{,}94 \text{ m}^2$$

A total = 1,98 + 1,43 + 0,9 + 2,94 = 7,25 m^2

$P_{D20} = (4{,}64 \times 7{,}25) + (0{,}832 \times 2) + (1{,}226 \times 6{,}6) = 43{,}4$ KN

$P_{L20} = (5 \times 7{,}25) = 36{,}25$ KN

- Tipe 21

$$A_1 = \left(\frac{0{,}2+2}{2}\right).1{,}8 \times 2 = 3{,}96 \text{ m}^2$$

$$A_2 = \left(\frac{2{,}2+4}{2}\right).1{,}8 = 5{,}58 \text{ m}^2$$

$$A_3 = 0{,}5 \times 1 \times 1{,}8 = 1{,}602 \text{ m}^2$$

$$A_4 = \left(\frac{1{,}1+1{,}8}{2}\right).3 = 4{,}35 \text{ m}^2$$

A total = 3,96 + 5,58 + 1,602 + 4,35 = 15,539 m^2

$P_{D21} = (4{,}64 \times 15{,}539) + (0{,}832 \times 6) = 63{,}173$ KN

$P_{L21} = (5 \times 12{,}539) = 62{,}695$ KN

- Tipe 22

$$A_1 = \left(\frac{0{,}2+2}{2}\right).1{,}8 = 1{,}98 \text{ m}^2$$

$$A_2 = \left(\frac{2{,}2+4}{2}\right).1{,}8 = 5{,}58 \text{ m}^2$$

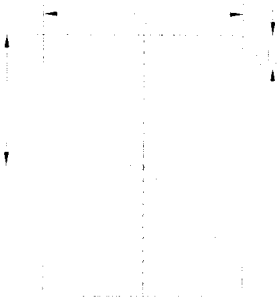
$$A_3 = \left(\frac{0{,}5+2}{2}\right).1{,}5 \times 2 = 3{,}75 \text{ m}^2$$

A total = 1,98 + 5,58 + 3,75 = 11,31 m^2

$P_{D22} = (4{,}64 \times 11{,}31) + (1{,}226 \times 6) = 59{,}834$ KN

$P_{L22} = (5 \times 11{,}31) = 56{,}55$ KN

- Tipe 23



$$A = \left(\frac{0,5+2}{2}\right).1,5 \times 4 \quad = 7,5\ m^2$$

$$P_{D23} = (4,64 \times 7,5) + (0,832 \times 4) \ = 38,128\ KN$$

$$P_{L23} = (5 \times 7,5) = 37,5\ KN$$

- Tipe 24

$$A_1 = \left(\frac{2,5+4}{2}\right).1,5 \quad = 4,875\ m^2$$

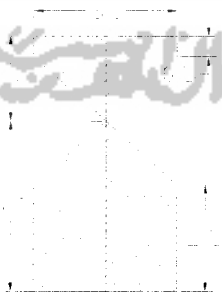
$$A_2 = \left(\frac{0,5+2}{2}\right).1,5 \times 3 \quad = 5,625\ m^2$$

$$A\ total = 4,875 + 5,625 = 10,5\ m^2$$

$$P_{D24} = (4,64 \times 10,5) + (1,226 \times 6) \ = 56,076\ KN$$

$$P_{L24} = (5 \times 10,5) = 52,5\ KN$$

- Tipe 25



$$A_1 = \left(\frac{2,5+4}{2}\right).1,5 \times 2 \quad = 9,75\ m^2$$

$$A_2 = \left(\frac{0,5+2}{2}\right).1,5 \times 2 \quad = 3,75\ m^2$$

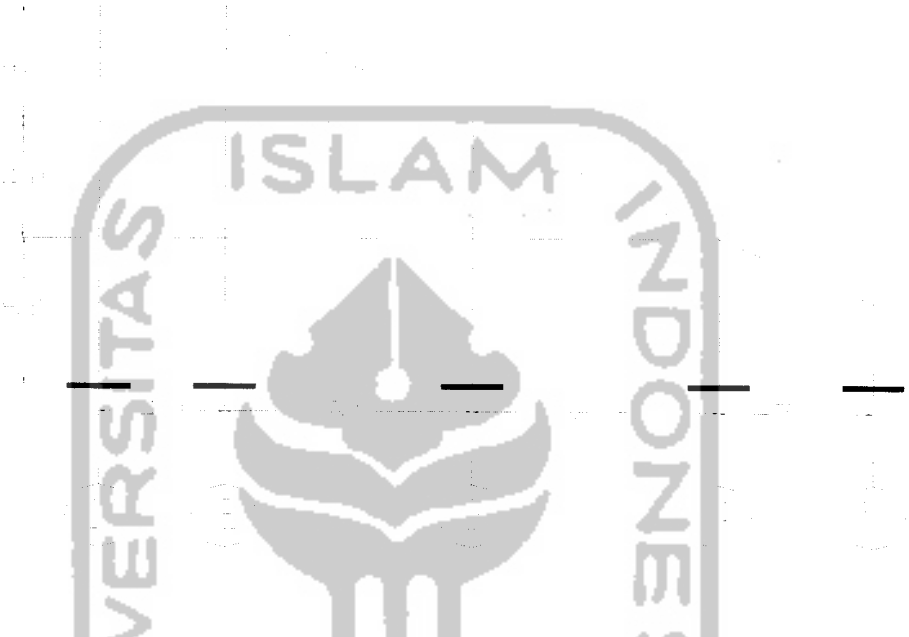
$$A\ total = 9,75 + 3,75 = 13,5\ m^2$$

$$P_{D25} = (4,64 \times 13,5) + (0,832 \times 6) \ = 67,632\ KN$$

$$P_{L25} = (5 \times 13,5) = 67,5\ KN$$

o **Portal C**

Pembebanan gravitasi dari pelat ke balok portal C ditunjukkan gambar 5.9a dibawah ini.



Gambar 5.9 Portal C

Bebab merata lantai 2.5

$Q_{D10} = 4.64 \times 1{,}5 \times 2 = 12{,}96$ KN/m

$Q_{L10} = 5 \times 1{,}5 \times 2 = 15$ KN/m

beban merata lantai 5.15 & 9.15

$Q_{D7} = 4{,}64 \times 1{,}5 \times 2 = 13{,}92$ KN/m

$Q_{L7} = 5 \times 1{,}5 \times 2 = 15$ KN/m

$Q_{D1} = 4{,}64 \times 1{,}5 = 6{,}96$ KN/m

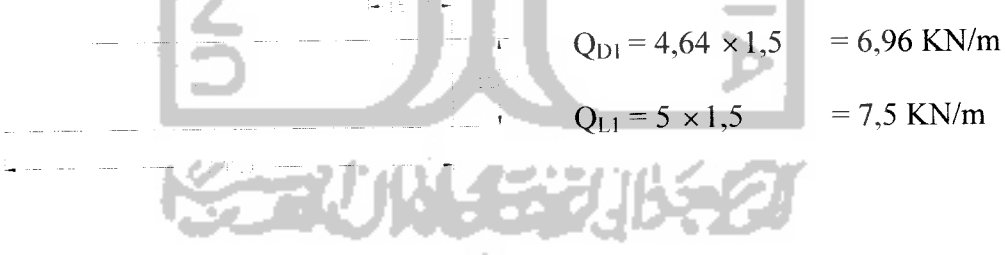
$Q_{L1} = 5 \times 1{,}5 = 7{,}5$ KN/m

$Q_{D7a} = 4{,}64 \times 1{,}5 = 6{,}96$ KN/m

$Q_{L7a} = 5 \times 1{,}5 = 7{,}5$ KN/m

$Q_{D7b} = 4{,}64 \times 1{,}5 = 6{,}96$ KN/m

$Q_{L7b} = 5 \times 1{,}5 = 7{,}5$ KN/m



$Q_{D1} = 4{,}64 \times 1{,}5 = 6{,}96$ KN/m

$Q_{L1} = 5 \times 1{,}5 = 7{,}5$ KN/m

Beban merata lantai 13.15

$Q_{D2} = 4{,}64 \times 1{,}5 \times 2 = 12{,}96$ KN/m

$Q_{L2} = 5 \times 1{,}5 \times 2 = 15$ KN/m

$Q_{D2} = 9{,}8 \times 1{,}5 \times 2 \quad = 12{,}96$ KN/m

$Q_{L2} = 5 \times 1{,}5 \times 2 \quad = 15$ KN/m

$Q_{D2} = 9{,}8 \times 1{,}5 \times 2 = 12{,}96$ KN/m

$Q_{L2} = 5 \times 1{,}5 \times 2 \quad = 15$ KN/m

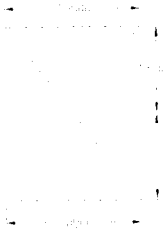
$Q_{D2} = 9{,}8 \times 1{,}5 \times 2 = 12{,}96$ KN/m

$Q_{L2} = 5 \times 1{,}5 \times 2 \quad = 15$ KN/m

- Beban titik lantai 5.150

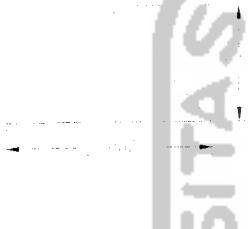
$$P_D = \left(\frac{1}{2} \times 1{,}5 \times 1{,}5 \times 2 \times 4{,}64\right) + (1{,}444 \times 3) = 14{,}768 \text{ KN}$$

$$P_L = \left(\frac{1}{2} \times 1{,}5 \times 1{,}5 \times 2 \times 5\right) = 11{,}25 \text{ KN}$$



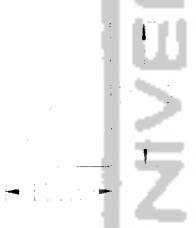
$$P_D = \left(\frac{1}{2} \times 3 \times 1{,}5 \times 2 \times 4{,}64\right) + (1{,}444 \times 3) = 25{,}212 \text{ KN}$$

$$P_L = \left(\frac{1}{2} \times 3 \times 1{,}5 \times 2 \times 5\right) = 22{,}5 \text{ KN}$$



$$P_D = \left(\frac{1}{2} \times 3 \times 1{,}5 \times 4{,}64\right) + (0{,}832 \times 1.5) = 11{,}688 \text{ KN}$$

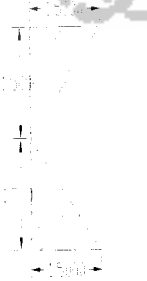
$$P_L = \left(\frac{1}{2} \times 3 \times 1{,}5 \times 5\right) = 11{,}25 \text{ KN}$$



$$P_D = \left(\frac{1}{2} \times 1{,}5 \times 1{,}5 \times 4{,}64\right) + (1{,}444 \times 1.5) = 6{,}664 \text{ KN}$$

$$P_L = \left(\frac{1}{2} \times 1{,}5 \times 1{,}5 \times 5\right) = 5{,}625 \text{ KN}$$

- Beban titik lantai 9,150



$$P_D = \left(\frac{1}{2} \times 1{,}5 \times 1{,}5 \times 2 \times 4{,}64\right) + (1{,}444 \times 3) = 14{,}76 \text{ KN}$$

$$P_L = \left(\frac{1}{2} \times 1{,}5 \times 1{,}5 \times 2 \times 5\right) \quad = 11{,}25 \text{ KN}$$

$$P_D = \left(\frac{1}{2} \times 3 \times 1{,}5 \times 4{,}64\right) + (0{,}832 \times 1{,}5) = 11{,}272 \text{ KN}$$

$$P_L = \left(\frac{1}{2} \times 3 \times 1{,}5 \times 5\right) = 11{,}25 \text{ KN}$$

$$P_D = \left(\frac{1}{2} \times 1{,}5 \times 1{,}5 \times 4{,}64\right) + (1{,}444 \times 1{,}5) = 6{,}664 \text{ KN}$$

$$P_L = \left(\frac{1}{2} \times 1{,}5 \times 1{,}5 \times 5\right) = 5{,}625 \text{ KN}$$

$$P_D = \left(\frac{1}{2} \times 1{,}5 \times 1{,}5 \times 4{,}64\right) + (1{,}444 \times 1{,}5) = 6{,}664 \text{ KN}$$

$$P_L = \left(\frac{1}{2} \times 1{,}5 \times 1{,}5 \times 5\right) = 5{,}625 \text{ KN}$$

- Beban titik lantai 13.150

$$P_D = \left(\frac{1}{2} \times 1{,}5 \times 1{,}5 \times 2 \times 4{,}64\right) + (1{,}444 \times 3) = 14{,}768 \text{ KN}$$

$$P_L = \left(\frac{1}{2} \times 1{,}5 \times 1{,}5 \times 2 \times 5\right) = 11{,}25 \text{ KN}$$

Beban titik pada balok miring

$$P_D = \left(\frac{1}{2} \times 3 \times 1{,}5 \times 2 \times 9{,}8\right) + (1{,}444 \times 3) = 25{,}212 \text{ KN}$$

$$P_L = \left(\frac{1}{2} \times 3 \times 1{,}5 \times 2 \times 5\right) = 22{,}5 \text{ KN}$$

$$P_D = \left(\frac{1}{2} \times 3 \times 1{,}5 \times 2 \times 9{,}8\right) + (1{,}444 \times 3) = 25{,}212 \text{ KN}$$

$$P_L = \left(\frac{1}{2} \times 3 \times 1{,}5 \times 2 \times 5\right) = 22{,}5 \text{ KN}$$

$$_{D} = \left(\frac{1}{2} \times 1{,}5 \times 1{,}5 \times 2 \times 9{,}8\right) + (1{,}444 \times 3) = 14{,}768 \text{ KN}$$

$$_{L} = \left(\frac{1}{2} \times 1{,}5 \times 1{,}5 \times 2 \times 5\right) = 11{,}25 \text{ KN}$$

5.2.2 Beban Gempa

Perhitungan gaya geser dasar horizontal akibat gempa diawali dengan menentukan waktu getar bangunan (T), koefisien gempa dasar (C), factor keutamaan (I), dan factor jenis struktur (K). Selain itu juga dilakukan perhitungan terhadap berat total bangunan (Wt).

1. Berat bangunan

Portal C

a. Lantai 3

Beban mati

Pelat lantai	= 4,64 × 4 × 9	= 155,52	KN
Pelat tribun	= 9,8 × 4,49 × 9	= 193,968	KN
Balok induk	= 1,444 × [4 + (2 .9)]	= 31,768	KN

Balok anak	= 0,832 × 4 × 2	= 6,656	KN
B.miring induk	= 1,444 × 4,49	= 6,484	KN
B.miring anak	= 0,803 × 2 × 4,49	= 7,211	KN
Kolom	= 1,131 × 2 × 2	= 4,524	KN
Dinding	= 2,5 × 2 × 9	= 45	KN
		W_{D3} = 451,131	KN

Beban hidup

$Q_L = 4$ KN/m^2

Koefisien reduksi = 0,5

$W_L = 0,5 \times 5 \times [(4 \times 9) + (4,49 \times 9)] = 191,025$ KN

Beban total $W_3 = W_D + W_L = 431,734 + 191,025 = 642,156$ KN

b. Lantai 2

Beban mati

Pelat lantai	= 4,64 × [(4 × 9) + (8 × 9)]	= 278,4	KN
Pelat tribun	= 9,8 × 8,98 × 9	= 387,936	KN
Balok induk	= 1,444 × [12 + (3 .9)]	= 56,316	KN
Balok anak	= 0,832 × 4 × 4	= 13,312	KN
B.miring induk	= 1,444 × 8,98	= 12,967	KN
B.miring anak	= 0,803 × 2 × 8,98	= 14,422	KN
Kolom	= 1,131 × 3 × 4	= 13,572	KN
Dinding	= 2,5 × 4 × 9	= 90	KN
		W_{D2} = 866,925	KN

Beban hidup

$Q_L = 4$ KN/m^2

Koefisien reduksi = 0,5

$W_L = 0{,}5 \times 5 \times [(4 \times 9) + (8 \times 9) + (8{,}98 \times 9)] = 352{,}05$ KN

Beban total $W_2 = W_D + W_L = 828{,}131 + 352{,}05 = 1218{,}975$ KN

a. Lantai 1

Beban mati

Pelat lantai	$= 4{,}64 \times [(4 \times 9) + (2 \times 8 \times 9)]$	= 584,64	KN
Pelat tribun	$= 9{,}8 \times 10{,}1 \times 9$	= 436,32	KN
Pelat kantilever	$= 4{,}64 \times 9 \times 2.5$	= 97.2	KN
Balok induk	$= 1{,}444 \times [20 + 15 + (2 .9)]$	= 65,702	KN
Balok anak	$= 0{,}832 \times [20 + 12 + 4{,}5]$	= 30,368	KN
B.miring induk	$= 1{,}444 \times 10{,}1$	= 14,584	KN
B.miring anak	$= 0{,}803 \times 2 \times 10{,}1$	= 16,221	KN
Kolom	$= 1{,}131 \times 4{,}575 \times 4$	= 20,697	KN
Dinding	$= 2{,}5 \times 4{,}575 \times 9$	= 102,938	KN
		W_{D1} = 1368,67	KN

Beban hidup

$Q_L = 4$ KN/m^2

Koefisien reduksi = 0,5

$W_L = 0{,}5 \times 5 \times [(4 \times 9) + (10{,}1 \times 9) + (2 \times 8 \times 9) + (9 \times 2{,}5)]$

$= 598{,}5$ KN

Beban total $W_1 = W_D + W_L = 1325{,}038 + 598{,}5 = 1967{,}17$ KN

Maka berat total bangunan :

$Wt = W_1 + W_2 + W_3$

$= 1967{,}17 + 1218{,}975 + 642{,}156$

$= 3828{,}301$ KN

2. Beban Gempa

a) Waktu getar bangunan (T)

Dengan rumus empiris :

$Tx = Ty = 0{,}085 . H^{3/4} \rightarrow H = 13{,}15$ m

$Tx = Ty = 0{,}085 . 13{,}15^{3/4} = 0{,}587$ detik

b) Koefisien Gempa Dasar (C)

Koefisien gempa dasar (C) diperoleh dari gambar 3.1 sesuai dengan pedoman perencanaan ketahanan gempa untuk rumah dan gedung 1987.

Dengan : T = 0,587 detik, Daerah gempa 3

Jenis tanah adalah tanah lunak

Maka dari Respon Spektra diperoleh nilai C = 0,07

c) Faktor Keutamaan (I) dan Faktor Jenis Struktur (K)

Berdasarkan PPKGURDG, 1987 diperoleh I = 1,5 dan K = 1

d) Gaya Geser Dasar (V)

V = C.I.K.Wt ..... (3.1)

= 0,07. 1,5. 1. 3828,301 = 401,972 KN

e) Distribusi Gaya Horizontal Tingkat

H = 13,15 m, B = 20 m

$$\frac{H}{B} = \frac{13,15}{20} = 0,6575 < 3$$

Maka seluruh beban didistribusikan sebagai gaya horizontal dengan menggunakan persamaan (3.3) :

$$Fi = \frac{Wi.hi}{\sum Wi.hi} \times V \quad \text{..... (3.3)}$$

Tabel 5.1 Hitungan Gaya Horizontal Portal C

No	*Bagian*	*Wi*	*Hi*	*Wi Hi*	*Fx,y*
1	tingkat 3	642.156	13.15	8444.351	114.178
2	tingkat 2	1218.975	9.15	11153.621	150.811
3	tingkat 1	1967.17	5.15	10130.926	136.983
			Wi Hi	29728.898	

Tabel hitungan gaya horizontal akibat gempa dapat dilihat pada lampiran 1.

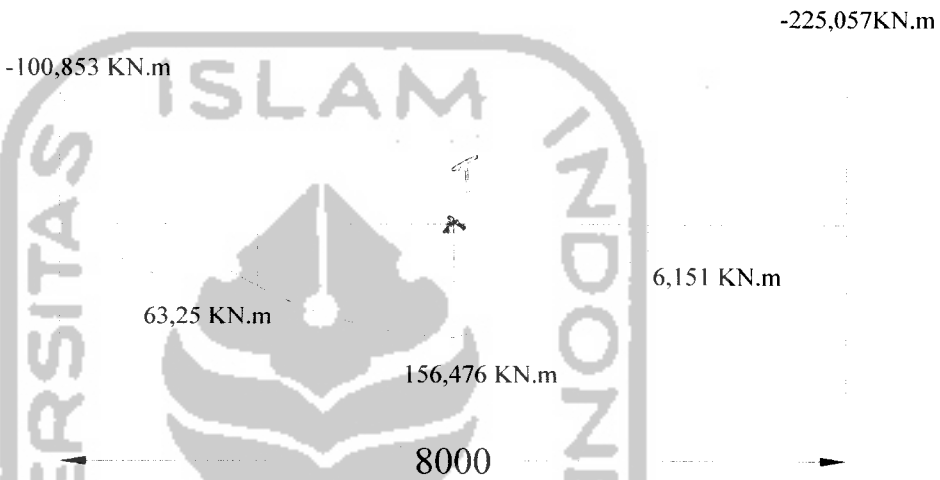
5.3 Perencanaan Balok

5.3.1 Perencanaan Balok Portal As B

a. Analisis dua dimensi (2D)

Kombinasi beban gempa dan gravitasi

Balok B2 (Batang 2)



Gambar 5.12 Bending Momen

Bentang yang terjadi lengkung ganda (*double curvature*)

Mu = 225,057 KN.m (Lampiran A4)

$$Cb = \frac{12{,}5.M_{maks}}{2{,}5.M_{maks} + 3M_A + 4M_B + 3M_C} \leq 2{,}3 \quad \text{..... (3.16)}$$

$$= \frac{12{,}5.225{,}057}{2{,}5.225{,}057 + 3.63{,}253 + 4.156{,}476 + 3.6{,}151}$$

$$= 2{,}01 < 2{,}3$$

Dipakai profil W16X36 (ry = 38,61 mm; Zx = 1048772,1 mm^3;

X_1 = 11721,5 MPa; $X_2 = 438.10^{-6}$ $(1/MPa)^2$; Sx = 925869,12 mm^3)

Cek penampang kompak :

Rasio lebar sayap terhadap tebal sayap

$$\frac{b}{2.tf} \leq \frac{171}{\sqrt{Fy}} \quad \text{..... (3.11)}$$

$$\frac{177,419}{2.10,922} = 8,122 \leq \frac{171}{\sqrt{250}} = 10,815$$

Rasio tinggi badan terhadap tebal badan

$$\frac{d}{tw} \leq \frac{1680}{\sqrt{Fy}} \quad \text{..... (3.12)}$$

$$\frac{402,844}{7,493} = 53,763 \leq \frac{1680}{\sqrt{250}} = 106$$

$$\text{Lp} = 1,76.ry.\sqrt{\frac{E}{fy}} = 1,76.38,61\sqrt{\frac{2.10^5}{250}} = 1921,92 \text{ mm}$$

$$\text{Lr} = \frac{ry.X_1}{(fy-fr)}.\sqrt{1+\sqrt{1+X_2.(fy-fr)^2}} \quad \text{..... (3.15)}$$

$$= \frac{38,61.11721,5}{(250-70)}.\sqrt{1+\sqrt{1+438.10^{-6}.(250-70)^2}}$$

$$= 5562,76 \text{ mm}$$

$$\text{L} = 4 \text{ m} = 4000 \text{ mm}$$

Lp = 19121,92 mm < Lb = 4000 mm < Lr = 5562,76 mm

Maka kuat nominal komponen struktur terhadap momen lentur adalah

$$\text{Mn} = \text{Cb}.\left[Mr + (Mp - Mr).\frac{(Lr-L)}{(Lr-Lp)} \right] \leq Mp \quad \text{..... (3.14)}$$

dengan:

Mp= Zx.Fy = 1048772,1 $\times$250

$= 262{,}19.10^6$ N.mm = 262,19 KN.m

Mr = (fy – fr).Sx = (250 – 70). 925869,12

$= 166{,}66.10^6$ N.mm = 166,66 KN.m

Maka :

$$Mn = 2{,}01 \cdot \left[166{,}66 + (262{,}19 - 166{,}66).\frac{(5{,}563 - 4)}{(5{,}563 - 1{,}922)}\right]$$

= 418,26 KN.m > Mp = 262,19 KN.m

Mn pakai = 262,19 KN.m

0,9 . Mn = 0,9 . 262,19 = 235,971 KN.m > Mu = 215,057 KN.m

AMAN

$\frac{235{,}971}{225{,}057} = 1{,}1 < 1{,}25$ (sesuai dengan batasan yang kami tentukan)

Kontrol Defleksi

Dari Program SAP 2000 didapatkan defleksi profil balok W16x36 ditengah bentang adalah 1,16 cm

Maksimum defleksi diizinkan adalah = $\frac{8}{360}$ = 0,022 m = 2,2 cm

Profil balok W16x36 dapat digunakan

Kuat Tarik yang tersedia

P aksial balok = 77,774 KN (Lampiran A12)

Profil balok W16x36 (Ag = 6838,696 mm^2)

Kuat tarik yang tersedia = 0,9 . fy . Ag ..... (3.19b)

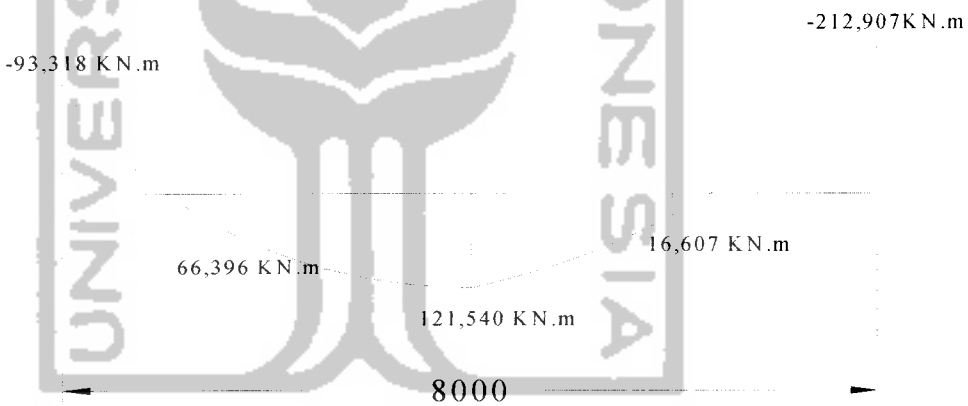
= 0,9 . 250 . 6838,696 = 1538706,6 N

= 1538,707 KN > P = 77,774 KN

b. Analisis tiga dimensi (3D)

Kombinasi beban gempa dan gravitasi

Balok B2



Gambar 5.12 Bending Momen

Bentang yang terjadi lengkung ganda (*double curvature*)

Mu = 212,907 KN.m (Lampiran A8)

$$Cb = \frac{12{,}5.M_{maks}}{2{,}5.M_{maks} + 3M_A + 4M_B + 3M_C} \leq 2{,}3 \quad \text{..... (3.16)}$$

$$= \frac{12{,}5.212{,}907}{2{,}5.212{,}907 + 3.66{,}396 + 4.121{,}54 + 3.16{,}607}$$

$= 2{,}1 < 2{,}3$

Dipakai profil W16X36 (ry = 38,61 mm; Zx = 1048772,1 mm^3;

X_1 = 11721,5 MPa; $X_2 = 438.10^{-6}$ $(1/MPa)^2$; Sx = 925869,12 mm^3)

Cek penampang kompak

Rasio lebar sayap terhadap tebal sayap

$$\frac{b}{2.tf} \leq \frac{171}{\sqrt{Fy}} \quad \text{..... (3.11)}$$

Rasio tinggi badan terhadap tebal badan

$$\frac{177{,}419}{2.10{,}922} = 8{,}122 \leq \frac{171}{\sqrt{250}} = 10{,}815$$

$$\frac{d}{tw} \leq \frac{640}{\sqrt{Fy}} \quad \text{..... (3.12)}$$

$$\frac{402{,}844}{7{,}493} = 53{,}763 \leq \frac{1680}{\sqrt{250}} = 106$$

$$Lp = 1{,}76.ry.\sqrt{\frac{E}{fy}} = 1{,}76.38{,}61\sqrt{\frac{2.10^5}{250}} = 1921{,}92 \text{ mm}$$

$$Lr = \frac{ry.X_1}{(fy - fr)}.\sqrt{1 + \sqrt{1 + X_2.(fy - fr)^2}} \quad \text{..... (3.15)}$$

$$= \frac{38{,}61.11721{,}5}{(250 - 70)}.\sqrt{1 + \sqrt{1 + 438.10^{-6}.(250 - 70)^2}}$$

$= 5562{,}76$ mm

L = 4 m = 4000 mm

Lp = 19121,92 mm < Lb = 4000 mm < Lr = 5562,76 mm

Maka kuat nominal komponen struktur terhadap momen lentur adalah

$$Mn = Cb.\left[Mr + (Mp - Mr).\frac{(Lr - L)}{(Lr - Lp)} \right] \leq Mp \quad \text{..... (3.14)}$$

dengan:

Mp = Zx.Fy = 1048772,1 × 250

= 262,19.10^6 N.mm = 262,19 KN.m

Mr = (fy – fr).Sx = (250 – 70). 925869,12

= 166,66.10^6 N.mm = 166,66 KN.m

Maka :

$$Mn = 2{,}1 \cdot \left[166{,}66 + (262{,}19 - 166{,}66).\frac{(5{,}563 - 4)}{(5{,}563 - 1{,}922)} \right]$$

= 436,05 KN.m > Mp = 262,19 KN.m

Mn pakai = 262,19 KN.m

0,9 . Mn = 0,9 . 262,19 = 235,971 KN.m > Mu = 212,907 KN.m

AMAN

$\frac{235{,}971}{212{,}907} = 1{,}12 < 1{,}25$ (sesuai dengan batasan yang kami tentukan)

Kontrol Defleksi

Dari Program SAP 2000 didapatkan defleksi profil balok W16x36 ditengah bentang adalah 0,61 cm

Maksimum defleksi diizinkan adalah = $\frac{8}{360}$ = 0,022 m = 2,2 cm

Profil balok W16x36 dapat digunakan

Kuat Tarik yang tersedia

P aksial balok = 79,437 KN (Lampiran A16)

Profil balok W16x36 (Ag = 6838,696 mm^2)

Kuat tarik yang tersedia = 0,9 . fy . Ag ..... (3.19b)

= 0,9 . 250 . 6838,696 = 1538706,6 N

= 1538,707 KN > P = 79,437 KN

Perhitungan perencanaan balok selanjutnya dapat dilihat pada lampiran D.

5.3.2 Gaya Geser Rencana Balok

a. Analisis dua dimensi (2D)

Gaya geser pada balok B_2 portal B adalah (Lihat lampiran A17)

V_D = 53,186 KN

V_L = 61,277 KN

V_E = 12,758 KN

Gaya geser balok diambil dari nilai minimum dua persamaan berikut :

$$Vu = 1{,}2.V_D + 0{,}5.V_L + \frac{2.Mpb}{L'} \quad \text{..... (3.20a)}$$

$$= 1{,}2.\ 53{,}186 + 0{,}5.\ 61{,}277 + \frac{2.262{,}19}{7{,}54} = 170{,}964 \text{ KN}$$

$$Vu = 1{,}2.V_D + 0{,}5.V_L + \mu.V_E \quad \text{..... (3.20b)}$$

$$= 1{,}2.\ 53{,}186 + 0{,}5.\ 61{,}277 + 4.\ 12{,}758 = 145{,}494 \text{ KN}$$

Gaya gsesr rencana balok adalah 145,494 KN

b. Analisis tiga dimensi (3D)

Gaya geser pada balok B_2 portal B adalah (Lihat lampiran A18)

V_D = 57,151 KN

V_L = 43,656 KN

V_E = 11,552 KN

Gaya geser balok diambil dari nilai minimum dua persamaan berikut :

$$Vu = 1{,}2.V_D + 0{,}5.V_L + \frac{2.Mpb}{L'} \quad \text{..... (3.20a)}$$

$$= 1{,}2.\ 57{,}151 + 0{,}5.\ 43{,}656 + \frac{2.262{,}19}{7{,}54} = 166{,}912 \text{ KN}$$

$$Vu = 1{,}2.V_D + 0{,}5.V_L + \mu.V_E \quad \text{..... (3.20b)}$$

$$= 1{,}2.\ 57{,}151 + 0{,}5.\ 43{,}656 + 4.\ 11{,}552 = 136{,}617 \text{ KN}$$

Gaya gsesr rencana balok adalah 136,617 KN

Perhitungan gaya geser rencana selanjutnya dapat dilihat pada lampiran A17 dan A18.

5.3.3 Kuat Geser Nominal Balok Portal

a. Analisis dua dimensi (2D)

Diketahui gaya geser yang bekerja pada balok B2 Portal B adalah

Vu = 145,494 KN (Lampiran A17)

Kontrol perbandingan tinggi terhadap tebal panel (h/tw) pendukung geser

h = 0,95. d = 0,95. 402,844 = 362,560 mm

$$\frac{h}{tw} < 1{,}1.\sqrt{\frac{k_n E}{fy}} \quad \text{..... (3.22a)}$$

$$\frac{h}{tw} = \frac{362{,}560}{7{,}493} = 48{,}387 < 1{,}1.\sqrt{\frac{k_n E}{fy}} = 1{,}1.\sqrt{\frac{5{,}01.200000}{250}} = 69{,}67$$

Aw = d.tw = 402,844. 7,493 = 3018,51 mm^2

ϕ Vn = 0,9. 0,6.fy.Aw ..... (3.22b)

= 0,9. 0,6. 250. 3018,51

= 407498,85 N = 407,499 KN

Rasio tegangan yang terjadi

$$\frac{Vu,k}{\phi Vn} = \frac{145{,}494}{407{,}499} = 0{,}357 < 1{,}0 \rightarrow \text{AMAN}$$

b. Analisis tiga dimensi (3D)

Diketahui gaya geser yang bekerja pada balok B2 adalah

Vu = 136,617 KN (Lampiran A18)

Kontrol perbandingan tinggi terhadap tebal panel (h/tw) pendukung geser

h = 0,95. d = 0,95. 402,844 = 362,560 mm

$$\frac{h}{tw} < 1{,}1.\sqrt{\frac{k_n E}{fy}} \quad \text{..... (3.22a)}$$

$$\frac{h}{tw} = \frac{362{,}560}{7{,}493} = 48{,}387 < 1{,}1.\sqrt{\frac{k_n E}{fy}} = 1{,}1.\sqrt{\frac{5{,}01.200000}{250}} = 69{,}67$$

Aw = d.tw = 402,844. 7,493 = 3018,51 mm^2

ϕ Vn = 0,9. 0,6.fy.Aw ..... (3.22b)

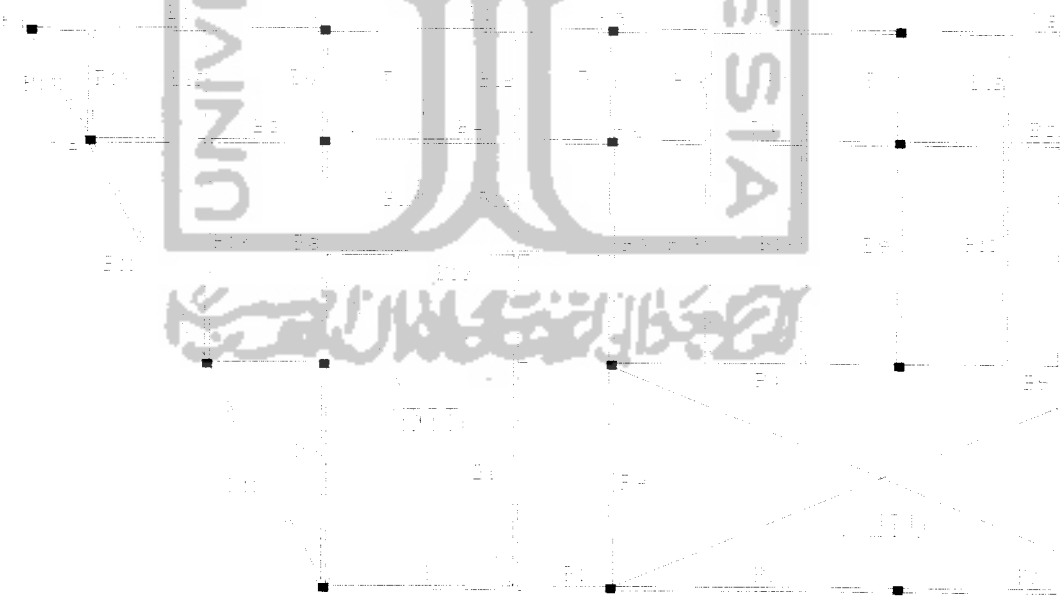
= 0,9. 0,6. 250. 3018,51

= 407498,85 N = 407,499 KN

Rasio tegangan yang terjadi

$$\frac{Vu,k}{\phi Vn} = \frac{136{,}617}{407{,}499} = 0{,}340 < 1{,}0$$

Perhitungan kuat geser balok dapat dilihat pada lampiran D9 dan D10



Gambar 5.13 Pola Balok Lantai 1

Tabel 5.2 Rekapitulasi Profil Balok Lantai 1

Analisis 2D		Analisis 3D	
Balok	**Profil**	**Balok**	**Profil**
B1	W18X60	B1	W18X55
B2	W18X60	B2	W18X55
B3	W18X55	B3	W18X55
B4	W18X76	B4	W18X71
B5	W18X86	B5	W18X76
B6	W16X40	B6	W16X31
B7	W16X26	B7	W16X26
B8	W16X40	B8	W16X31
B9	W16X36	B9	W16X36
B10	W14X22	B10	W14X22
B11	W14X26	B11	W14X26
B12	W14X22	B12	W14X22
B13	W14X226	B13	W14X226

Tabel 5.3 Rekapitulasi Profil Balok Lantai 2

Analisis 2D		Analisis 3D	
Balok	**Profil**	**Balok**	**Profil**
B1	W18X55	B1	W18X46
B2	W18X55	B2	W18X46
B3	W18X46	B3	W18X46
B4	W18X55	B4	W18X55
B5	W18X60	B5	W18X55
B6	W16X26	B6	W16X26
B7	W16X26	B7	W16X26
B8	W16X36	B8	W16X26
B9	W16X31	B9	W16X31
B10	W14X22	B10	W14X22
B11	W14X26	B11	W14X26
B12	W14X22	B12	W14X22
B13	W14X26	B13	W14X26

Tabel 5.4 Rekapitulasi Profil Balok Lantai 3

Analisis 2D		Analisis 3D	
Balok	**Profil**	**Balok**	**Profil**
B1	W16X36	B1	W18X46
B2	W16X36	B2	W18X46
B3	W16X26	B3	W18X46
B4	W16X31	B4	W18X55
B5	W16X36	B5	W18X55
B6	W14X22	B6	W16X26
B7	W14X26	B7	W16X26
B10	W14X22	B10	W14X22
B11	W14X26	B11	W14X26
B12	W14X22	B12	W14X22
B13	W14X26	B13	W14X26

5.4 Perencanaan kolom

Perencanaan kolom didasarkan pada prinsip desain *strong column and weak beam*, dimana respon struktur pada kolom (Muk, Nuk, dan Vu,k) terlebih dulu dicari berdasarkan kapasitas maksimum profil balok yang digunakan.

5.4.1 Momen Rencana Kolom (Mu,k)

Analisis tiga dimensi (3D)

Momen Rencana kolom

Profil balok lantai 1 (B1) adalah W16x26 (d = 398,526 mm, Zx = 724308,229 mm^3).

Mp = Zx. Fy = 724308,229. 250 =181,077.10^6 N.mm = 181,077 KN.m

Angka kekakuan (α)

$$\alpha a = \frac{I_a / L_a}{[(I_b / L_b) + (I_a / L_a)]} \quad \text{..... (3.24a)}$$

$$= \frac{107912}{107912 + 138397} = 0{,}438$$

$$\alpha b = \frac{I_b / L_b}{[(I_b / L_b) + (I_a / L_a)]} \quad \text{..... (3.24b)}$$

$$= \frac{138397}{107912 + 138397} = 0{,}562$$

Perhitungan selanjutnya dapat dilihat pada tabel 5.11 dibawah ini.

Tabel 5.11 Angka Kekakuan

Ting kat	Elemen	Arah X				Arah Y			
		Ib/Lb (mm3)	Ia/La (mm3)	αbx	αax	Ib/Lb (mm3)	Ia/La (mm3)	αby	αay
1	Eksterior, ki	107912	138397	0,438	0,562	12332,75	15816,75	0,438	0,562
	Interior, ki	107912	138397	0,438	0,562	12332,75	15816,75	0,438	0,562
	Eksterior,ka	221435	0,000	1,000	0,000	25306,8	0,000	1,000	0,000
	Interior,ka	107912	0,000	1,000	0,000	12332,75	0,000	1,000	0,000
2	Eksterior,ki	138397	138397	0,500	0,500	15816,75	15816,75	0,500	0,500
	Interior	138397	138397	0,500	0,500	15816,75	15816,75	0,500	0,500
	Eksterior,ka	138397	0,000	1,000	0,000	138397	0,000	1,000	0,000
3	Eksterior	138397	0,000	1,000	0,000	138397	0,000	1,000	0,000
	Interior	138397	0,000	1,000	0,000	138397	0,000	1,000	0,000

L balok = 4m = 4000 mm

$$L^1 = 4000 - \frac{462{,}534}{2} - \frac{462{,}534}{2} = 3537{,}466 \text{ mm}$$

H kolom = 5,13 m = 5130 mm

$$hn = 5130 - \frac{398{,}526}{2} = 4930{,}737 \text{ mm}$$

$$Mu,k = \frac{hn}{h}.\alpha.1{,}1.Ry.\left(\frac{L_{bi}}{L^1_{bi}}.M_{bi} + \frac{L_{ba}}{L^1_{ba}}.M_{ba}\right) \quad \text{..... (3.23a)}$$

Maka :

$$Mu_{,kb} = \frac{4,93}{5,13}.0,438.1,1.1,5.\left(\frac{4}{3,54}.199,185\right) = 156,315 \text{ KN.m}$$

$$Mu_{,kb} = 1,2.\ 9,102 + 0,5.\ 7,841 + 4.\ 129,245 = 531,821 \text{ KN.m}$$

Dengan demikian, momen rencana kolom ($Mu_{,k}$ x) bawah pada join lantai 1 (K3) adalah sebesar 156,315 KN.m

Perhitungan momen rencana kolom dapat dilihat pada lampiran B1.

5.4.2 Gaya aksial rencana kolom

Hasil analisis struktur pada kolom K5 lantai 3 adalah (lihat lampiran B2)

P_D,k = 76,861 KN

P_L,k = 54,56 KN

P_E,k = 3,235 KN

Kolom diapit 3 balok yang saling tegak lurus, yaitu satu balok arah x dengan profil W14x26 dan dua balok arah y dengan profil W16x26.

Kapasitas lentur sendi plastis pada kedua ujung balok.

$$Pu,k = \left(\frac{Mpb,ki}{L'_{ki}} + \frac{Mpb,ka}{L'_{ka}}\right) + 1,05.Pg,k \quad \text{..... (3.26)}$$

$$= \left(\frac{2.181,159}{3,54} + \left[2.\left(\frac{2.199,185}{8,54}\right)\right]\right) + 1,05.(76,861 + 54,56)$$

$$= 235,814 \text{ KN}$$

Tetapi tidak lebih besar dari :

$$Pu,k = 1,2.P_D + 0,5.P_L + \mu.P_E = 1,2.76,861 + 0,5.54,56 + 4.3,235$$

$$= 106,573 \text{ KN}$$

Jadi gaya aksial rencana kolom (P_U,k) terpakai adalah sebesar 106,573 KN

Perhitungan gaya aksial rencana kolom dapat dilihat pada lampiran B2.

5.4.3 Gaya Geser Kolom

Hasil analisis struktur pada kolom lantai 2 (lihat lampiran B3)

V_D,k x = 3,269 KN

V_L,k x = 3,799 KN

V_E,k x = 29,536 KN

Mu,k desain = 86,142 KN.m (dari hitungan momen rencana kolom)

$$Vu,k = \frac{Mu,k_{bawah} + Mu,k_{atas}}{h'_k} \quad \text{..... (3.29)}$$

$$= \frac{2.86,142}{3,62}$$

$$= 47,592 \text{ KN}$$

Tetapi tidak lebih besar dari :

$$Vu,k = 1,2.V_D,k + 0,5.V_L,k + \mu .V_E,k \quad \text{..... (3.30)}$$

$$= 1,2.\ 3,269 + 0,5.\ 3,799 + 4.\ 29,536$$

$$= 123,966 \text{ KN}$$

Jadi gaya geser rencana kolom (V_u,k) terpakai adalah sebesar 47,592 KN

Perhitungan gaya geser rencana kolom dapat dilihat pada lampiran B3.

5.4.4 Perencanaan Kolom Terhadap Momen Lentur dan Gaya Aksial Rencana

Analisis tiga dimensi (3D)

Kolom Eksterior lantai 2 (K3)

Mu,k x = 92.103 KN.m

Mu,k y = 248.014 KN.m

Pu,k = 484.327 KN

Kekakuan kolom arah x

$$G_A = \frac{\left(\frac{553587796}{5150}\right)+\left(\frac{553587796}{4000}\right)}{\left(\frac{125285659,1}{4000}\right)} = 6,86 \quad \text{..... (3.20)}$$

$$G_B = \frac{\left(\frac{553587796}{5150}\right)+\left(\frac{553587796}{4000}\right)}{\left(\frac{125285659,1}{4000}\right)} = 6,86$$

Dari grafik Johnson dan moreland didapatkan nilai kx = 2,25

Kekakuan kolom arah y

$$Ga = \frac{\left(\frac{553587796}{5150}\right)+\left(\frac{553587796}{4000}\right)}{\left(\frac{2.254733632,5}{9000}\right)} = 1,688 \quad \text{..... (3.20)}$$

$$G_B = \frac{\left(\frac{553587796}{5150}\right)+\left(\frac{553587796}{4000}\right)}{\left(\frac{2.254733632,5}{9000}\right)} = 1,6688$$

Dari grafik Johnson dan moreland didapatkan nilai ky = 1,5

Perhitungan kekakuan kolom dapat dilihat pada tabel 5.14 dibawah ini.

Tabel 5.14 Kekakuan Kolom

Tingkat	Bagian	G	Gx	Kx	Gy	Ky
1	Tepi	Ga	6.86	1.75	1.688	1.4
		Gb	1		1	
	Tengah	Ga	3.935	1.68	3.797	1.65
		Gb	1		1	
2	Tepi	Ga	6.86	2.25	1.688	1.5
		Gb	6.86		1.688	
	Tengah	Ga	3.935	2	3.797	1.9
		Gb	3.935		3.797	
3	Tepi	Ga	8.84	2.95	3.45	1.7
		Gb	6.86		1.688	
	Tengah	Ga	6.68	2.12	4.89	2.1
		Gb	3.935		3.797	

Dicoba Profil W14x99 (ry = 94,234 mm; rx = 156,718 mm ; Sx = 2572769,05 mm^3 ; Zx = 2834962,07 mm^3 ; A = 18774,156 mm^2 ; X_1 = 219950,5 Mpa ; $X_2 = 1,89.10^{-6}$; $\frac{bf}{2.tf} = 9,34$; $\frac{d}{tw} = 29,2$)

$$\frac{(K.L)x}{rx} = \frac{2,25 \times 4000}{156,718} = 57,428$$

$$\frac{(K.L)y}{ry} = \frac{1,5 \times 4000}{94,742} = 63,330 \rightarrow \text{yang dipakai}$$

Dari tabel AISC-LRFD hal 6-124 didapatkan nilai $\phi c.Fcr$

$\phi c.Fcr = 170,58$ Mpa

$\phi.Pn = \phi c.Fcr \times A = 170,58 \times 18774,156 = 3202,539$ KN

$$Lp = 1,76.ry.\sqrt{\frac{E}{fy}} = 1,76.94,234\sqrt{\frac{2.10^5}{250}} = 4690,998 \text{ mm} > Lb = 4000 \text{ mm}$$

Maka kuat nominal komponen struktur terhadap momen lentur adalah

ϕ Mnx $= \phi.Zx.Fy = 0,9.\ 2834962,07.\ 250.\ 10^{-6} = 637,866$ KN.m

ϕ Mny $= \phi.Zy.Fy = 0,9.\ 1369958,55.\ 250.\ 10^{-6} = 308,241$ KN.m

Selanjutnya dikontrol terhadap persamaan interaksi kolom aksial dan lentur

$$\frac{Pu}{\phi.Pn} = \frac{484,327}{3202,539} = 0,14 < 0,2$$

Rumus AISC-LRFD

$$\frac{Pu}{2.\phi.Pn} + \left(\frac{Mux}{\phi.Mnx} + \frac{Muy}{\phi.Mny}\right) \quad \text{..... (3.19)}$$

$$0,07 + \left(\frac{92.103}{637.866} + \frac{248.014}{308.241}\right) = 0,92 < 1,0 \rightarrow \textbf{AMAN}$$

Perhitungan perencanaan kolom terhadap lentur dan aksial dapat dilihat pada lampiran E

5.4.5 Perencanaan Kolom Terhadap Geser

Gaya geser (Vu,k) pada kolom K3 lantai 2 sebesar 119,628 KN

Kontrol rasio tinggi terhadap tebal panel (h/tw)

$$\frac{db}{tw} = \frac{359,664}{12,319} = 29,196 < 1,1.\sqrt{\frac{k_n E}{fy}} = 1,1.\sqrt{\frac{5,01.200000}{250}} = 69,67$$

Aw $= d.tw = 359,664.\ 12,319 = 4430,701$ mm^2

ϕ Vn = 0,9. 0,6.fy.Aw

= 0,9. 0,6. 250. 4430,701

= 996908 N = 996,908 KN

Rasio tegangan yang terjadi

$$\frac{Vu,k}{\phi Vn} = \frac{119,628}{996,908} = 0,12 < 1,0 \rightarrow \textbf{AMAN}$$

Perhitungan perencanaan kolom terhadap geser dapat dilihat pada lampiran E3

Gambar 5.13 Hasil Desain Kolom dan Balok Portal 1

Gambar 5.14 Hasil Desain Kolom dan Balok Portal 2

5.5 Perencanaan Sambungan

5.5.1 Sambungan Balok ke Kolom

Diambil contoh hitungan portal 1 lantai 2 tepi kiri, antara balok B1 dan K3.

Data profil desain yang dipakai

Balok W18X60

Ag = 11290,3 mm^2

d = 463,296 mm

tw = 105,41 mm

bf = 191,897 mm

tf = 17,653 mm

Zx = 2015608,872 mm^3

Zy = 337573,518 mm^3

Fy = 250 Mpa

Fu = 410 Mpa

Kolom W14X109

Ag = 20851,571 mm^2

d = 363,728 mm

tw = 13,335 mm

bf = 370,967 mm

tf = 21,844 mm

Zx = 3146316,288 mm^3

Zy = 1519080,833 mm^3

Fy = 250 Mpa

Fu = 410 Mpa

k = 39,688 mm

Prosedur desain

a. Menentukan beban yang bekerja berdasarkan kapasitas plastis balok

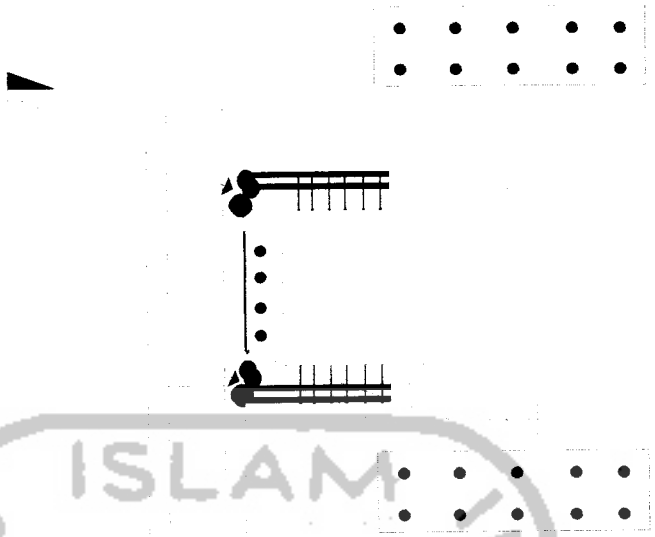
Momen kapasitas balok supaya terjadi strain hardening

Mp = 1,1. Zx. Fy

= 1,1 . 2015608,872. 250

= 554292439,8 N.mm

= 554,292 KN.m



Gambar 5.15 Rencana Sambungan Yang Digunakan

Gaya geser pada lokasi sendi plastis balok B1 hasil dari analisis struktur dapat dilihat pada lampiran A adalah sebagai berikut :

V_D = 86,248 KN

V_L = 36,467 KN

V_E = 51,143 KN

Gaya geser pada sendi plastis balok, merupakan nilai terkecil dari :

$$Vp = 1{,}2.V_D + 0{,}5.V_L + \frac{2.Mp}{L'}$$

$$= 1{,}2.86{,}248 + 0{,}5.36{,}467 + \frac{2.554{,}292}{8{,}54} = 122{,}669 \text{ KN}$$

$$Vp = 1{,}2.V_D + 0{,}5.V_L + \mu.V_E$$

$$= 1{,}2.\ 86{,}248 + 0{,}5.\ 36{,}467 + 4.\ 51{,}143 = 326{,}303\ KN$$

Momen pada muka kolom, Mf

$$Mf = Mpb + Vp.X = 554{,}292 + 122{,}669.0{,}831 = 656{,}288\ KN.m$$

b. Sambungan flange plate ke sayap kolom

Gaya tarik terfaktor, Tu

$$Tu = \frac{Mf}{0{,}95.d} \quad \text{..... (3.40)}$$

$$= \frac{656{,}288.10^3}{0{,}95.463{,}296} = 1491{,}119\ KN$$

Menenntukan ukuran flange plate yang menahan tarik dan desak :

Kondisi leleh tarik :

$$Tu \leq \phi.Fy.Ag \quad \text{..... (3.42)}$$

$$Ag_{min} = \frac{1491{,}119.10^3}{0{,}9.250} = 6627{,}196\ mm^2 \text{ (menentukan)}$$

Kondisi fraktur :

$$Tu \leq \phi.\ Fu.\ Ae \rightarrow Ae = U.\ An \quad \text{..... (3.43)}$$

Untuk $An \leq 0{,}85.\ Ag$

$U = 1 \rightarrow$ untuk pelat sambung/batang tarik pendek

$$Tu \leq \phi.\ Fu.\ U.\ 0{,}85.\ Ag$$

$$Ag_{min} = \frac{1491{,}119.10^3}{0{,}75.410.1.0{,}85} = 5704{,}903\ mm^2$$

Jika digunakan lebar flange plate, bpl = 165 mm

Maka tebal flange plate, tpl = $\frac{6627,196}{165}$ = 40,165 mm ≈ 45 mm

Menentukan jumlah baut yang diperlukan untuk menstranfer gaya tarik dan tekan pada bagian atas dan bawah balok :

Kuat geser satu baut A_{325}-X diameter ⅞ in

$\phi Rn = \phi.(0,6.\ Fu^b).\ m.\ A_b$ ..... (3.44)

= 0,75. (0,6. 838). 1. 387,96 = 146295,158 N = 146,295 KN

Kebutuhan baut minimal untuk menahan geser

$n_{min} = \frac{Tu}{\phi Rn} = \frac{1491,119}{146,295} = 10,193 \rightarrow 12$ buah ..... (3.45)

Kuat tumpu sayap balok

$\phi Rn = \phi.\ 2,4.\ Fu.\ d_b.\ t_p$ ..... (3.46)

= 0,75. 2,4. 410. 25. 17,653 = 325697,85 N

= 325,698 KN > 146,295 KN

Menentukan ukuran plat geser, jumlah baut dan panjang las :

$Vf = \frac{2.M_f}{Ln} + Vg$ ..... (3.50)

$= \frac{2.656,288}{8.54} + 1,2.86,248 + 0,5.36,467 = 298,58$ KN

Apabila digunakan baut dengan diameter ¾ in

$\phi Rn = \phi.(0,6.\ Fu^b).\ m.\ A_b$ ..... (3.44)

= 0,75. (0,6. 838). 1. 285,023 = 107482,157 N = 107,482 KN

Jumlah baut n = $\frac{V_f}{\phi Rn} = \frac{298,58}{107,482}$ = 2,78 → 4 buah ..... (3.52)

Dicoba plat ⅜ X 11,5

Kontrol geser leleh pada plat

$\phi Rn = \phi.(0,6.\ Fy).\ A_g$ ..... (3.53)

= 0,9. (0,6. 250). 2782,253 = 375604,088 N

= 375,604 KN > Vf = 298,58 KN → *Ok*

Kontrol geser fraktur pada plat :

$\phi Rn = \phi.(0,6.\ Fu).\ A_n$ ..... (3.54)

= 0,75. (0,6. 410). 1995,964 = 368255,312 N

= 368,255 KN >Vf = 298,58 KN → *Ok*

Menentukan panjang las fillet pada plat geser

a_{max} = tpl - 1/16 = ⅜ - 1/16 = 5/16 in

a_{min} = 3/16 in

a_{pakai} = ¼ in

te = 0,707. a = 0,707. 0,25 = 0,177 in = 5 mm

Kekuatan las terhadap geser

ϕR_{nw} = ϕ.(0,6.fuw). te ..... (3.55)

= 0,75. (0,6.485). 5 = 1091,25 N/mm (menentukan)

Kekuatan las terhadap geser fraktur dari logam dasar

ϕR_{nw} = ϕ.(0,6.fu). tpl ..... (3.56)

= 0,75. (0,6. 410). 10 = 1845 N/mm

Panjang las yang dibutuhkan

$$\text{P las} = \frac{Vf}{\phi Rnw} \quad \text{..... (3.57)}$$

$$= \frac{298{,}58.10^3}{1091{,}25} = 273{,}613 \text{ mm} \rightarrow \text{pakai } 275 \text{ mm}$$

5.5.3 Perencanaan Sambungan Kolom dengan Kolom

Diketahui profil kolom K3 lantai 1 adalah W14X159 dan lantai 2 adalah W14X99. Mu,k = 698,745 KN.m dan Pu,k = 657,569 KN

Kolom W14X159

Ag	= 30128,97 mm^2	Zx	= 4703087,368 mm^3
d	= 381 mm	Zy	= 2392511,344 mm^3
tw	= 18,923 mm	Fy	= 250 Mpa
bf	= 396,24 mm	Fu	= 410 Mpa
tf	= 30,226 mm	k	= 46,736 mm

Gaya pada sayap kolom

$$\text{Pu}_f = \frac{Mu,k}{0{,}95.d} \quad \text{..... (3.58)}$$

$$= \frac{698{,}745.10^3}{0{,}95.363{,}728} = 2022 \text{ KN}$$

a. Sambungan pada sayap

Kuat geser satu baut

Apabila digunakan baut tipe A325 dengan diameter ¾ in

$$\phi Rn = \phi.(0{,}6.\ Fu^b).\ m.\ A_b \quad \ldots\ldots (3.44)$$

$$= 0{,}75.\ (0{,}6.\ 838).\ 1.\ 285{,}023 = 107482{,}157\ N = 107{,}482\ KN$$

Jumlah baut yang diperlukan

$$n_{perlu} = \frac{Pu_f}{2.\varphi.Rn} \quad \ldots\ldots (3.59)$$

$$= \frac{2022}{2.107{,}482} = 9{,}4 \approx 10 \text{ buah}$$

Dicoba pelat lebar 10,5 in

Luas penampang bruto *flange plate*

$$Ag = \frac{Pu_f}{0{,}9.Fy} \quad \ldots\ldots (3.62)$$

$$= \frac{2022.10^3}{0{,}9.250} = 8986{,}66\ mm^2$$

$$t_{min} = \frac{Ag}{l} = \frac{8986{,}66}{10{,}5.25{,}4} = 33{,}696\ mm$$

Kontrol kekuatan pelat sambung

$$\phi Rn = \phi.fu.Ant > \frac{Pu_f}{2} \quad \ldots\ldots (3.63)$$

$$Agt = ¾\ .10{,}5 = 7{,}875\ in^2 = 5080{,}635\ mm^2$$

$$Ant = (10{,}5 - 2(¾ + \tfrac{1}{6})).\ ¾ = 6{,}5\ in^2 = 4193{,}54\ mm^2$$

$$0{,}85.Agt = 0{,}85.\ 5080{,}635 = 4318{,}54\ mm^2 > Ant = 4193{,}54\ mm^2$$

$$\phi Rn = \phi.fu.Ant > \frac{Pu_f}{2} \quad \text{..... (3.63)}$$

$$\phi Rn = 0{,}75.\ 410.\ 4193{,}54 = 1289{,}514\ KN > \frac{2022}{2} = 1011\ KN$$

AMAN

b. Sambungan pada badan

Gaya pada badan kolom

$$Puw = \frac{Pu,k.Aw}{A} \quad \text{..... (3.66)}$$

$$= \frac{657{,}569.30128{,}97}{6065{,}73} = 2266{,}2\ KN$$

Kuat geser satu baut

Apabila digunakan baut tipe A325 dengan diameter ¾ in

$$\phi Rn = \phi.(0{,}6.\ Fu^b).\ m.\ A_b \quad \text{..... (3.44)}$$

$$= 0{,}75.\ (0{,}6.\ 838).\ 1.\ 285{,}023 = 107482{,}157\ N = 107{,}482\ KN$$

Jumlah baut yang diperlukan

$$n_{perlu} = \frac{Pu_f}{2.\varphi.Rn} \quad \text{..... (3.59)}$$

$$= \frac{2266{,}2}{2.107{,}482} = 10{,}2 \approx 12\ buah$$

Dicoba pelat lebar 10,5 in

Luas penampang bruto *flange plate*

$$Ag = \frac{Pu_f}{0{,}9.Fy} \quad \text{..... (3.62)}$$

$$= \frac{2266{,}2.10^3}{0{,}9.250} = 10072 \text{ mm}^2$$

$$t_{min} = \frac{Ag}{l} = \frac{10072}{10{,}5.25{,}4} = 37{,}766 \text{ mm}$$

Kontrol kekuatan pelat sambung

$$\phi Rn = \phi.fu.Ant > \frac{Pu_f}{2} \quad \text{..... (3.63)}$$

$Agt = ¾ .10{,}5 = 7{,}875 \text{ in}^2 = 5080{,}635 \text{ mm}^2$

$Ant = (10{,}5 - 2(¾ + \frac{1}{16})).¾ = 6{,}5 \text{ in}^2 = 4193{,}54 \text{ mm}^2$

$0{,}85.Agt = 0{,}85.\ 5080{,}635 = 4318{,}54 \text{ mm}^2 > Ant = 4193{,}54 \text{ mm}^2$

$$\phi Rn = \phi.fu.Ant > \frac{Pu_f}{2} \quad \text{..... (3.63)}$$

$$\phi Rn = 0{,}75.\ 410.\ 4193{,}54 = 1289{,}514 \text{ KN} > \frac{2266{,}2}{2} = 1133{,}1 \text{ KN}$$

AMAN

5.5.4 Perencanaan Pelat Dasar Kolom

Momen dan gaya aksial yang bekerja

Mu,k x = 615,878 KN.m (Lampiran B1)

Mu,k y= 173,612 KN.m (Lampiran B1)

Pu, k = 704,969 KN (Lampiran B2)

Digunakan Profil kolom W14X159 dengan data sebagai berikut :

Ag	= 30128,97 mm^2	Zx	= 4703087,368 mm^3
d	= 381 mm	Zy	= 2392511,344 mm^3
tw	= 18,923 mm	Fy	= 250 Mpa
bf	= 396,24 mm	Fu	= 410 Mpa
tf	= 30,226 mm	k	= 46,736 mm

Eksentrisitas akibat momen

$$ey = \frac{Mu,kx}{Pu,k} = \frac{615,878}{704,969} = 0,874 \text{ m}$$

$$ey = \frac{Mu,ky}{Pu,k} = \frac{173,612}{704,969} = 0,246 \text{ m}$$

diasumsikan e > L/6, akibat e terjadi pembesaran gaya aksial, maka diberi sayap tambahan = 101,9 mm (menggunakan profil C12X45).

Kesetimbangan momen pada pusat gaya aksial T :

$$Pu,k\left(\frac{dc}{2}+\frac{bf_{ca}}{2}\right)+Mu,k_x+Mu,k_y = R\left(dc+bf_{ca}-\frac{tf}{2}\right) \quad ...(3.77)$$

$$T = \frac{704,969\left(\frac{0,381}{2}+\frac{0,396}{2}\right)+615,878+173,612}{0,381+0,396-\frac{0,0302}{2}}$$

= 1395,68 KN

Diasumsikan luas bidang tekan efektif penumpu akibat momen yang bekerja adalah (X.B), sehingga gaya tekan yang terjadi harus memenuhi :

$\phi P_p \geq Pu$ ..... (3.78)

$\phi(0,5.Fp.\ X.B) = Pu$

$$(X.B) = \frac{2.Pu}{\phi.Fp} = \frac{2.1395,68.1000}{\phi.0,5.25} = 372181,33\ mm^2$$

Coba B = 0,7 m, maka panjang bidang tekan

$$X = \frac{X.B}{0,7m} = \frac{372181,33}{700} = 531,69\ mm$$

Jarak dari pusat flens ke ujung pelat = 1/3.X ..... (3.79)

= 1/3. 531,69 = 177,23 mm

Panjang pelat dasar yang dibutuhkan

L = (2.X) + (dc – tf) ..... (3.80)

= (2.177,23) + (381-30,226) = 705,23 mm = 0,705 m

Diambil L = 710 mm = 0,71 m

Dipakai B = 0,7 m dan L = 0,71 m

$$m = \frac{L - 0,95.dc}{2}$$ ..... (3.81)

$$= \frac{0,71 - 0,95.0,381}{2} = 0,17\ m$$

$$n = \frac{B - 0,8.bf}{2}$$ ..... (3.82)

$$= \frac{0,7 - 0,8.0,396}{2} = 0,19\ m$$

Gambar 5.17 Desain Pelat Dasar Kolom

Tegangan pada ujung pelat :

$$fp = \frac{Pu}{B.L} \pm \frac{Mu,kx}{\frac{1}{6}B.L^2} \pm \frac{Mu,ky}{\frac{1}{6}B^2.L} < Fp \text{ (fy dalam Mpa) .... (3.83)}$$

$$\text{fp max} = \frac{1395,68}{0,7.0,71} + \frac{615,878}{\frac{1}{6}0,7.0,71^2} + \frac{173,612}{\frac{1}{6}0,7^2.0,71} = 14884,66 \text{ KN/m}^2$$

$$\text{fp min} = \frac{1395,68}{0,7.0,71} - \frac{615,878}{\frac{1}{6}0,7.0,71^2} - \frac{173,612}{\frac{1}{6}0,7^2.0,71} = 154,23 \text{ KN/m}^2$$

Cek kapasitas penumpu (pedestal)

$\phi Pp \geq Pu$ ..... (3.84)

$\phi Pp = \phi_c.Fp.A$ ..... (3.85)

= 0,6. 0,5. 25. 497000 = 3727500 N = 3727,5 KN > 1395,68

Perencanaan baut angkur arah y yang menahan Mu,kx

$$T = \frac{Mu,kx}{d} \quad \text{..... (3.92)}$$

$$= \frac{615,878}{0,381+0,102} = 1275,11 \text{ KN}$$

Kapasitas tarik satu angkur (ϕTn), digunakan baut angkur A325 D = 1 in

$$\phi Tn = \phi.0,75.fu.Ab \quad \text{..... (3.93)}$$

$$= \phi.\ 0,75.\ 838.\ 506,707 = 191079,39 \text{ N} = 191,08 \text{ KN}$$

Jumlah angkur minimum yang diperlukan

$$n = \frac{T}{\phi Tn} \quad \text{..... (3.94)}$$

$$= \frac{1275,11}{191,08} = 6,67 \approx 8$$

Perencanaan baut angkur arah x yang menahan Mu,ky

$$T = \frac{Mu,ky}{d} \quad \text{..... (3.92)}$$

$$= \frac{173,612}{0,381+0,102} = 559,45 \text{ KN}$$

Kapasitas tarik satu angkur (ϕTn), digunakan baut angkur A325 D = 1 in

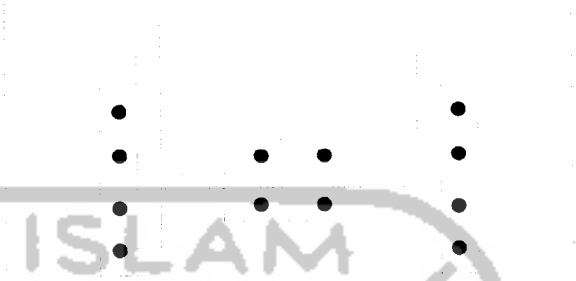
$$\phi Tn = \phi.0,75.fu.Ab \quad \text{..... (3.93)}$$

$$= \phi.\ 0,75.\ 838.\ 506,707 = 191079,39 \text{ N} = 191,08 \text{ KN}$$

Jumlah angkur minimum yang diperlukan

$$n = \frac{T}{\phi Tn} \quad \text{..... (3.94)}$$

$$= \frac{559,45}{191,08} = 2,9 \approx 4$$



Gambar 5.18 Desain Pelat Dasar Kolom

5.5.5 Perencanaan Pedestal (Kaki Kolom)

Dimensi pelat dasar kolom, L = 710 mm dan B = 700 mm

Dimensi pedestal, b = 800 mm dan h = 950 mm

Tulangan longitudinal

Rasio tulangan pakai, $\rho = 1\ \%$

$$Ast = 0,01.\ Ag \quad \text{..... (3.99)}$$

$$= 0,01.\ 800.\ 950 = 7600\ mm^2$$

Digunakan $D_{25} \rightarrow A_{\phi 25} = 490,874\ mm^2$

Jumlah tulangan longitudinal

$$n = \frac{Ast}{A_{\phi 22}} \quad \text{..... (3.100)}$$

$$= \frac{7600}{490{,}874} = 15{,}5 \approx 16 \text{ buah}$$

Dipakai tulangan $16D_{25}$ dipasang merata pada pedestal

Tulangan sengkang

Vu,k pakai = 184,367 KN (lihat lampiran B3)

$$Vs = \frac{Vu, k_{pakai}}{\varphi} \quad \text{..... (3.101)}$$

$$= \frac{184{,}367}{0{,}6} = 307{,}28 \text{ KN}$$

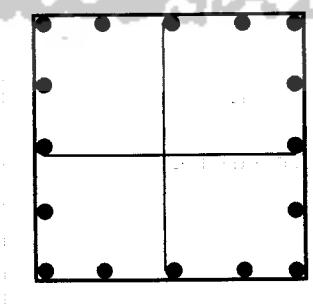
Dipakai tulangan sengkang $P_{10} \rightarrow A_{\phi 10} = 78{,}54 \text{ mm}^2$

Jarak antar tulangan

$$S = \frac{Av.fy.d}{Vs} \quad \text{..... (3.102)}$$

$$= \frac{1{,}5.78{,}54.400.1000}{307{,}28.1000} = 206{,}72 \text{ mm}$$

Dipakai sengkang 3 kaki, $1{,}5P_{10} - 200$



Gambar 5.19 Desain Pedestal (kaki kolom)