BAB IV

PERHITUNGAN KONSTRUKSI

4.1 Perencanaan Atap

Pada perencanaan ulang ini digunakan 3 macam rangka atap (kuda-kuda) yang direncanakan dengan menggunakan profil baja, di bawah ini gambar kuda - kuda

1. KK-1 $\alpha = 49°$ $\beta = 22°$

Gambar 4.1 Rencana Rangka KK-1

4.1.1 Perencanaan Gording Pada KK-1

1. Data-data

Jarak antar kuda-kuda = 6,0 m

Mutu baja Profil fy = 2400 kg/cm^2

Kuat tarik Fu = 3700 kg/cm^2

Sambungan dengan menggunakan las listrik dengan mutu sambungan setara dengan mutu baja propil

Gording dipakai baja jenis Light Lip Channel

Direncanakan terhadap bangunan di darat

2. Panjang batang

Struktur kuda – kuda baja dengan panjang batang sebagai berikut :

$\cos 49 = {}^{7.5}\!/_{r}$

r = 11.431 m

3. Jarak antar gording

Jarak antar gording = panjang batang / jumlah gording

$\frac{11.431}{11} = 1.0392$

Pembebanan Gording KK- 1a

A. Beban tetap

- Beban penutup atap = 10 x 2 x ½ x 1.0392 = 10.392 kg/m
  (genteng metal)
- Beban hidup = 20 x 2 x ½ x 1.0392 = 20.74 kg/m
- Berat gording (perkiraan) = 10,00 kg/m +
  = 41.132 kg/m

$q\perp$ = q cos α = 41.132 . cos 49° = 26.985 kg/m

q // = q sin α = 41.132 . sin 49° = 31.043 kg/m

B. Beban angin

Pada daerah daratan w = 25 kg/cm^2

- Angin tekan (wt) $\alpha < 65°$

  Diketahui sudut $\alpha = 49°$

  $C_1 = 0{,}02\ \alpha - 0{,}4 = 0{,}02 \cdot 49 - 0{,}4 = 0{,}58$

  $Wt = C_1 \cdot w \cdot$ jarak gording $= 0{,}58 \cdot 25 \cdot 1{,}0392 = 15{,}0684$ kg/m

- Angin hisap (wh)

  $C_2 = -0{,}4$

  $wh = C_2 \cdot w \cdot$ jarak gording $= -0{,}4 \cdot 25 \cdot 1{,}0392 = -10.392$ kg/m

C. Perhitungan momen

- Akibat beban tetap

  $q \perp$; $Mmaks = 1/8 \cdot q \perp \cdot b^2 = 1/8 \cdot 26{,}985 \cdot 6^2 = 121{,}4325$ kgm

  $= 12143{,}25$ kgcm

  $q//$; $Mmaks = 1/32 \cdot q// \cdot b^2 = 1/32 \cdot 31{,}043 \cdot 6^2 = 34{,}92$ kgm

  $= 3492$ kgcm

  - Jika sagrod yang digunakan di tengah maka momen maximum = $1/32 \cdot q// \cdot b^2$
  - Jika sagrod yang digunakan berada disepertiga panjang maka momen maximum = $1/90 \cdot q// \cdot b^2$

- Akibat beban angin

  $Mmaks = 1/8 \cdot w \cdot b^2 = 1/8 \cdot 15{,}0684 \cdot 6^2 = 67{,}807$ kgm

  $= 6780{,}7$ kgcm

D. Dimensi gording

Dicoba profil 125 x 50 x 20 x 3,2 (Light Lip Channel)

A = 7,807 cm^2 w = 6,13 kg/m < 10 kg/m

Ix = 137 cm^4 Sx = 21,9 cm^3

Iy = 20,6 cm^4 Sy = 6,22 cm^3

- Kontrol tegangan

$$\frac{fbx}{0,66Fy} + \frac{fby}{0,75Fy} \leq 1,0$$

$$fbx = \frac{M_{\perp}.max}{Sx} = \frac{12143,25}{21,9} = 554,486 \text{ kg/cm}^2$$

$$fby = \frac{M_{//}.max}{Sy} = \frac{3492}{6,22} = 561,414 \text{ kg/cm}^2$$

$$\frac{554,486}{0,66.2400} + \frac{561.414}{0,75.2400} = 0,662 \leq 1,0 \Rightarrow \text{ok!}$$

- Kontrol lendutan

$$\delta \perp = \frac{5}{384} \frac{q_{\perp}.L^4}{Elx} \leq \frac{L}{360}$$

$$= \frac{5}{384}.\frac{0,26985.600^4}{2,4.10^6.137} = 1,3849 \leq \frac{600}{360} = 1,667 \Rightarrow \text{ok!}$$

$$\delta // = \frac{5}{384} \frac{q_{//}.(L/(a+1))^4}{Ely} \leq \frac{L}{360}$$

$$= \frac{5}{384}.\frac{0,31043.(600/1+1))^4}{2,4.10^6.20.6} = 0,662 \leq \frac{600}{360} = 1,667 \Rightarrow \text{ok!}$$

Jadi profil C **125x50x20x3,2** dapat dipakai

E. Perencanaan sagrod dan tierod

- Beban sagrod

  - berat penutup atap x sisi miring = 10 .11.431

    = 114.31 kg/m

  - beban hidup x sisi miring = 20 . 11.431

    = 228,62 kg/m

  - jumlah gording satu sisi miring x berat gording= 11 . 6,13

    = 67,43 kg/m

  P = 114,31 + 228,62 + 67,43 = 410.36 kg/m

  P// = P . sin α . Ss = 410.36 sin 49° . 3 = 929.107 kg

- Dimensi sagrod

  $$\text{Asagrod} = \frac{P_{//}}{0.33 \cdot Fu} = \frac{1}{4} \cdot \pi \cdot D^2 \text{sagrod}$$

  $$= \frac{929.107}{0.33 \cdot 3700} = \frac{1}{4} \cdot \pi \cdot D^2$$

  $$\text{Dsagrod} = \sqrt{\frac{(929.107 \cdot 4)}{(0{,}33 \cdot 3700 \cdot \pi)}} = 0{,}98 \text{ cm} \Rightarrow 10 \text{ mm}$$

  Dsagrod pakai = 10+3 = 13 mm

- Dimensi tierod

  Beban tierod; T = P//.cos 49° = 929.107. cos 49° = 609.55 kg

  $$\text{Atierod} = \frac{T}{0{,}33Fu} = \frac{1}{4} \cdot \pi \cdot D^2 tierod$$

Dtierod $= \sqrt{\frac{(609.55 \cdot 4)}{(0,33 \cdot 3700 \cdot \pi)}} = 0,797$ cm ≈ 8 mm

Dtierod pakai = 8 + 3 = 11 mm

4.1.2 Perencanaan kuda-kuda

L = 15 m $\alpha = 49°$

Pembebanan kuda kuda

Digunakan profil WF. 200 x 150 x 6 x 9 mm

Berat profil = 30.6 kg/m , panjang profil digunakan = 22.862 m

Berat total kuda – kuda = 29.6×22.862 = 699.57 kg

Berat kuda-kuda yang digunakan:

- Berat total kuda-kuda = 699.57 kg
- Berat plat sambung, las & baut = (5 % x berat total kuda-kuda)

  = 35 kg

  Jumlah (Σ)= Berat total kuda-kuda + 5 % . Berat total kuda-kuda

  = 699.57 + 35 = 734,5 kg
- Panjang rangka kuda-kuda, L = 15 m

  $\frac{\Sigma}{L} = \frac{734,5}{15} = 49$ kg/m

  - Beban tetap

    - berat gording = 6.13 kg/m
    - berat eternit + penggantung = 18 kg/m^2
    - berat penutup atap = 10 kg/m^2
    - beban hidup = 20 kg/m^2
    - berat kuda-kuda = 49kg/m

Beban masing-masing joint

Perhitungan beban pada Program SAP 2000, berat sendiri kuda – kuda tidak diperhitungkan karena Program SAP 2000 menghitung secara otomatis berat sendiri kuda – kuda. Karena data pembebanan di bawah ini akan diinputkan ke program SAP 2000 maka berat sendiri kuda – kuda tidak diperhitungkan.

- P1 = P21

  Berat gording = 6.13 x 6.0 = 36,78 kg

  Berat penutup atap = 10 x 6.0 x ½ x 1.0392 = 31,176 kg

  Berat eternit + plafond = 18 x 6 x ½ x 1.0392 = 56,116 kg

  Beban mati = 124,072 kg

  = 1,24 KN

  Beban hidup = 20 x ½ x 1.0392 = 10,392 kg

  = 0,1 KN

- P2 – P20

  Berat gording = 6.13 x 6.0 = 36,78kg

  Berat penutup atap = 10 x 6.0 x 1.0392 = 62,352 kg

  Berat eternit + plafond = 18 x 6 x 1.0392 = 112,232 kg

  Beban mati = 211,364 kg

  = 2,11 KN

  Beban hidup = 20 x 1.0392 = 20,784 kg

  = 0,2 KN

- Beban angin

  Muatan angin di darat = 25 kg/m^2

  Koefisien angin menurut peraturan pembebanan untuk gedung 1983.

  untuk $\alpha < 65°$. Diketahui $\alpha = 49°$

  Tekan = C1 = 0.02 .α - 0,4 = 0.58

  Tarik = C2 = -0,4

  Beban yang bekerja

  Wt = C1 x w = 0,58 x 25 = 14,5 kg/m^2

  Wh = C2 x w = -0,4 x 25 = -10 kg/m^2

  - Sisi kiri

    W_1, W_{11} = 14,5 x ½ x 1,0392 x 6.0 = 45,205 kg

    $W_2 - W_{10}$ = 14,5 x 1,0392 x 6,0 = 90,41 kg

  - Sisi kanan

    W_{11}, W_{21} = 10 x ½ x 1,0392 x 6.0 = 31,176 kg

    $W_{12} - W_{20}$ = 10 x 1,0392 x 6.0 = 62,352 kg

  Syarat :

  - *30 % beban tetap + beban angin (angin kanan + angin kiri)*

    → Beban rencana = beban tetap

  - *30 % beban tetap + beban angin (angin kanan + angin kiri)*

    → Beban rencana = beban tetap + beban angin

4.1.3 Perencanaan Dukungan Arah lateral

Diketahui :

Lb = jarak antar gording = 1,04 m

Direncanakan dukungan pada jarak 2 Lb = 2,08 m

Lc = jarak antar kuda-kuda = 6 m

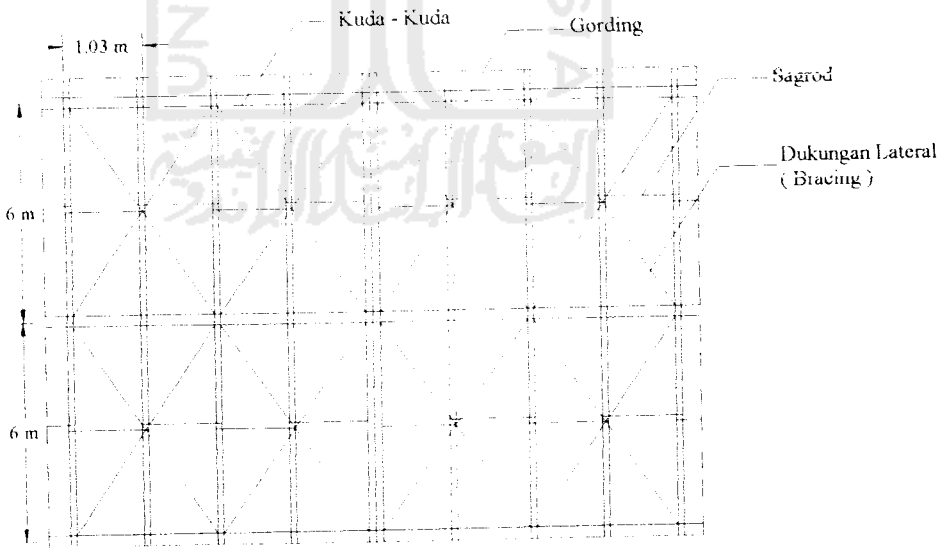
$L = \sqrt{2Lb^2 + Lc^2} = \sqrt{2{,}08^2 + 6^2} = 6{,}3$ m

Syarat $L/r \leq 300$, $\longrightarrow$ $r\ min \geq \frac{L}{300} = \frac{630}{300} = 2{,}1\ cm$

Keterangan :

- L ≤ 3 m → dipakai baja tulangan ∅ 12 mm
- L ≥ 3 m → dipakai baja tulangan ∅ 19 mm
- 3 m < L ≤ 5 m → dipakai baja tulangan ∅ 16 mm

L > 5 m maka digunakan baja tulangan ∅ 19 mm



Gambar 4.2 Tampak Atas Rencana Atap

4.1.4 Perencanaan Dimensi batang

- Perencanaan Rafter (batang utama)

Panjang batang = 11,431 m

Jarak antar gording = 1,0392 m

Fy = 2400 kg/cm^2 Fu = 3700 kg/cm^2

Dari perhitungan menggunakan program SAP 2000 didapat :

P maks = 39,88 KN

M maks = 27,63 KN m

S yang dibutuhkan = $\frac{M}{Fb} = \frac{27,63 \times 100 \times 100}{0,66.2400} = 174,43 cm^3$

Dicoba profil WF 200 x 150 x 6 x 9 , dari daftar pofil didapat :

Sx = 277 cm^3 ≥ 174,43 cm^3

A = 39,01 cm^2 ; tw = 6 mm

bf = 200 mm ; rx = 8,30 cm

d = 150 mm ; ry = 3,61 cm

tf = 9 mm ; Lb = 2,08 m (2 kali jarak gording)

dukungan lateral 2 kali jarak antar gording, maka Lb = 2,08 m

$$fa = \frac{P}{A} = \frac{39,88 \times 100}{37,66} = 105,89\ Kg/cm^2 = 1,53\ \text{ksi}$$

$$fb = \frac{M}{Sx} = \frac{27,63 \times 100 \times 100}{277} = 997,47\ Kg/cm^2 = 14,46\ \text{ksi}$$

Kontrol profil WF bagian sayap dan badan dengan rumus AISC

$$\frac{L}{bf} \leq \frac{76}{\sqrt{Fy}}$$

$$\frac{L}{bf}=\frac{2080}{200}=10,4\leq\frac{76}{\sqrt{36}}=12,66 \qquad \textit{ok}$$

$$\frac{bf}{2tf}\leq\frac{65}{\sqrt{Fy}}$$

$$\frac{200}{2.9}=10,5\leq\frac{65}{\sqrt{36}}=10,8 \qquad \textit{ok}$$

$$\frac{d}{tw}\leq\frac{640}{\sqrt{Fy}}\left(1-3,47\frac{fa}{Fy}\right)\geq\frac{257}{\sqrt{Fy}}$$

$$\frac{150}{6}\leq\frac{640}{\sqrt{36}}\left(1-3,47\frac{1,53}{36}\right)\geq\frac{257}{\sqrt{36}}$$

$25 < 90,9 > 42,8$ ***ok***

Lx = 1143,1 cm ; $(KL/r)x=\frac{1143,1}{8,30}=137,72$ (menentukan)

Ly = 208 cm ; $(KL/r)y=\frac{208}{3,61}=57,64$

$$Cc=\frac{6400}{\sqrt{Fy}}=\frac{6400}{\sqrt{2400}}=130,64$$

Kontrol kelangsingan :

$\frac{KL}{r} \geq Cc=\frac{6400}{\sqrt{Fy}}$ (Fy dalam Kg/cm^2) , maka

$$Fa=\frac{12}{23}\cdot\frac{\pi^2 E}{(KL/r)^2}=\frac{12}{23}\cdot\frac{\pi^2\ 2,1.10^6}{137,73^2}=570,05\ kg/cm^2$$

Lc = 6,4 ft = 1,95 m

Lu = 20,1 ft = 6,12 m (tabel A.3 Charles G. Salmon)

Lb = 2,08 > Lc = 1,95

Lb = 2,08 < Lc = 6,12

$Fb = 0.6\ Fy = 0.66 \times 2400 = 1440\ kg/cm^2$

$C_m = 1$ dianggap dukungan sederhana (tabel 12.10.1 Charles G. Salmon)

$$F'e = \frac{12}{23} \cdot \frac{\pi^2 E}{(KL/r)^2} = \frac{12}{23} \cdot \frac{\pi^2\ 2{,}1.10^6}{137{,}73^{\ 2}} = 570{,}05\ kg/cm^2$$

$$\frac{fa}{Fa} + \frac{fb_x}{Fb_x} \cdot \frac{C_m}{1 - \frac{fa}{F'_e}} \leq 1$$

$$\frac{105{,}89}{570{,}05} + \frac{997{,}47}{1548} \cdot \frac{1}{1 - \frac{105{,}89}{570{,}05}} = 0{,}185 + 0{,}644.1{,}22 = 0{,}97 < 1 \quad \textbf{OK}$$

Profil WF 200 x 150 x 6 x 9 mm dapat dipakai

- **Perencanaan Batang Sekunder**

Panjang batang = 6,47 m

Jarak antar gording = 1,078 m

$Fy = 2400\ kg/cm^2$ $Fu = 3700\ kg/cm^2$

Dari perhitungan menggunakan program SAP 2000 didapat :

P maks = 3,44 KN

M maks = 16,75 KN m

S yang dibutuhkan $= \frac{M}{Fb} = \frac{16{,}75 \times 100 \times 100}{0{,}66.2400} = 105{,}75\ cm^3$

Dicoba profil WF 148 x 100 x 6 x 9 , dari daftar pofil didapat :

$Sx = 138\ cm^3 \geq 105{,}75\ cm^3$

A = 26,84 cm^2 ; tw = 6 mm

bf = 148 mm ; rx = 6,17 cm

d = 100 mm ; ry = 2,37 cm

tf = 9 mm

dukungan lateral 2 kali jarak antar gording, maka Lb = 2,16 m

$$fa = \frac{P}{A} = \frac{3,44 \times 100}{26,84} = 12,82\ Kg/cm^2 = 0,185\ \text{ksi}$$

$$fb = \frac{M}{Sx} = \frac{16,75 \times 100 \times 100}{138} = 1213,76\ Kg/cm^2 = 17,65\ \text{ksi}$$

Kontrol profil WF bagian sayap dan badan dengan rumus AISC

$$\frac{L}{bf} \leq \frac{76}{\sqrt{Fy}}$$

$$\frac{L}{bf} = \frac{2160}{200} = 10,8 \leq \frac{76}{\sqrt{36}} = 12,66 \qquad \textbf{\textit{ok}}$$

$$\frac{bf}{2tf} \leq \frac{65}{\sqrt{Fy}}$$

$$\frac{148}{2.9} = 8,2 \leq \frac{65}{\sqrt{36}} = 10,8 \qquad \textbf{\textit{ok}}$$

$$\frac{d}{tw} \leq \frac{640}{\sqrt{Fy}}\left(1 - 3,47\frac{fa}{Fy}\right) \geq \frac{257}{\sqrt{Fy}}$$

$$\frac{100}{6} \leq \frac{640}{\sqrt{36}}\left(1 - 3,47\frac{0,185}{36}\right) \geq \frac{257}{\sqrt{36}}$$

$16,6 < 104,76 > 42,8 \qquad \textbf{\textit{ok}}$

Lx = 647 cm ; $(KL/r)x = \frac{647}{6,17} = 104,86$

Ly = 107,8 cm ; $(KL/r)y = \frac{107,8}{2,37} = 45,48$

$$Cc = \frac{6400}{\sqrt{Fy}} = \frac{6400}{\sqrt{2400}} = 130.64$$

Kontrol kelangsingan :

$$\frac{KL}{r} \leq Cc = \frac{6400}{\sqrt{Fy}}$$ (Fy dalam Kg/cm^2) , maka

$$Fa = \frac{\left(1 - \frac{(KL/r)^2}{2\,Cc^2}\right) Fy}{\frac{5}{3} + \frac{3\,(KL/r)}{8\,Cc} - \frac{(KL/r)^3}{8\,Cc^3}}$$

$$Fa = \frac{\left(1 - \frac{(104{,}86)^2}{2\times 130{,}64^2}\right) 2400}{\frac{5}{3} + \frac{3\,(104{,}86)}{8\times 130{,}64} - \frac{(104{,}86)^3}{8\times 130{,}64^3}} = \frac{1626{,}87}{1{,}67 + 0{,}30 - 0{,}06}$$

Fa = 851,73 kg/cm^2

Lc = 5,3 ft = 1,61 m

Lu = 16,6 ft = 5,06 m (tabel A.3 Charles G. Salmon)

Lb = 2,08 > Lc = 1,61

Lb = 2,08 < Lu = 5,06

Fb = 0,6 Fy = 0,66 x 2400 = 1440 kg/cm^2

$$F'e = \frac{12}{23} \cdot \frac{\pi^2 E}{(KL/r)^2} = \frac{12}{23} \cdot \frac{\pi^2\ 2{,}1.10^6}{104{,}86^2} = 983{,}45 \text{ kg/cm}^2$$

C_m = 1 dianggap dukungan sederhana (tabel 12.10.1 Charles G. Salmon)

$$\frac{fa}{Fa} + \frac{fb}{Fb} \cdot \frac{C_m}{1 - fa / F'_e} \leq 1$$

$$\frac{12{,}82}{851{,}73} + \frac{1213{,}76}{1440} \cdot \frac{1}{1 - \frac{12{,}82}{983{,}45}} = 0{,}015 + 0{,}843.1{,}01 = 0{,}869 < 1 \quad \textbf{\textit{OK}}$$

Profil WF 148 x 100 x 6 x 9 mm dapai dipakai

4.1.5 Perencanaan pelat kuda-kuda

P = 48,64 KN = 4864 kg

f'c = 22,5 Mpa = 225 kg/cm^2

$$\text{Aperlu} = \frac{P}{0{,}33.f'c} = \frac{48{,}64 \times 10^3}{0{,}33 \times 22{,}5} = 6550{,}8 \text{ mm}^2 = 65{,}2 \text{ cm}^2$$

Diambil ukuran pelat : 20 x 30 = 600 cm^2> Aperlu

$$q = \frac{P}{BxL} = \frac{4864}{20x30} = 8{,}10 \text{ kg/cm}^2$$

$$x = \frac{30-15}{2} = 7{,}5 \text{ cm}$$

M = ½ q. x^2 = ½ x 8,10 x 7,5^2 = 227.8 kgcm

Syarat :

$$0{,}66 \text{ fy} = \frac{M}{1/6.1.tp^2}$$

$$tp = \sqrt{\frac{10 \times 227{,}8}{2400}} = 0{,}974 \text{ cm} \approx 1 \text{ cm}$$

Sehingga dipakai pelat dengan tebal 1 cm

Pelat kuda-kuda berukuran : **20 x 30 x 1**

4.1.6 Perencanaan Sambungan

Sambungan pada struktur kuda – kuda baja yang digunakan pada Proyek Pembangunan Pasar Rakyat Teluk Kuantan Tahap – 2 ini adalah gabungan antara sambungan baut dan sambungan las. Untuk pelat penyambung digunakan sambungan las yang dilakukan di pabrik, dan untuk penyambungan utama digunakan baut dengan menggunakan baut mutu A325 D 19 mm

Diketahui :

profil WF 200 x 150 x 6 x 9 ; Bj 36 ksi

Sambungan terletak di puncak kuda – kuda dan digunakan baut A325

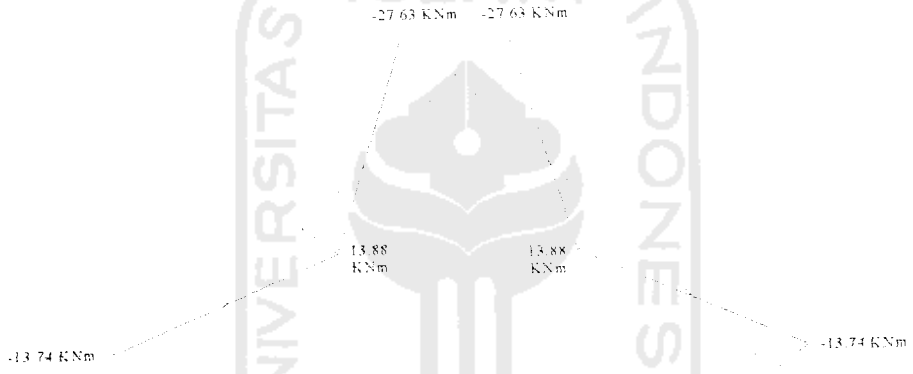
M = 27,63 KNm ; $Sx = 277\ cm^3$

D = 14,81 KN ; $F'_{t\ A325} = 44$ ksi $= 303$ mpa

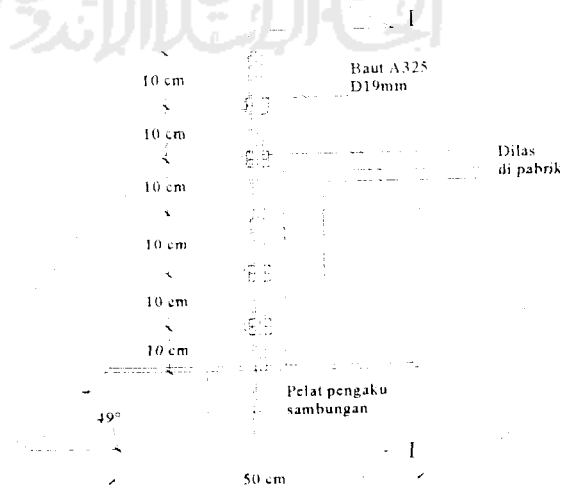
P = 39,9 KN ; $F_{v\ A325} = 30$ ksi $= 206$ mpa

A Pofil = 39,01 cm^2 ;

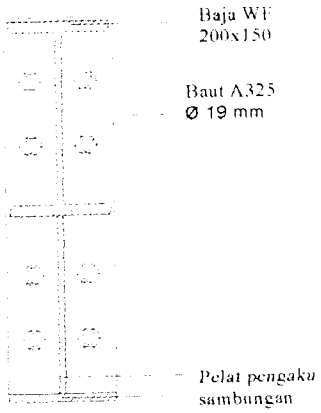
momen puncak kuda – kuda dapat dilihat pada gambar berikut ini :



Gambar 4.3 Bidang momen pada puncak kuda – kuda



Gambar 4.4 Sambungan Pada Puncak Kuda – kuda



Gambar 4.5 Pot I – I Sambungan Pada Puncak Kuda – kuda

$$f_{tb} = \frac{6M}{b.d^2} = \frac{6.27,63.10^6}{150.600^2} = 3,07\ mpa$$

reduksi tegangan tarik terhadap baut terluar (baut paling bawah)

$$f_{tb} = 3,07.\frac{200}{300} = 2,05\ mpa$$

gaya tarik dua baut yang paling besar (baut bawah)

$$T = f_{tb}\ .bp = 2,05.150.100 = 30750\ N$$

tegangan baut nominal ke dua baut paling bawah

$$f_t = \frac{T}{A_b} = \frac{30750}{2(\frac{\pi}{4}.19^2)} = 54,22\ mpa$$

komponen geser langsung

$$f_v = \frac{P}{\sum A} = \frac{39,9.10^3}{8.(\frac{\pi}{4}.19^2)} = 17,6\ mpa$$

Cek tegangan tarik ijin bila digabungkan dengan geser (tabel 4.10.1 Charles G. Salmon)

$F'_t \leq 379 - 1,8\ f_v \leq 303$

$F'_t = 379 - 1{,}8.17{,}6 = 347{,}3 > 303$

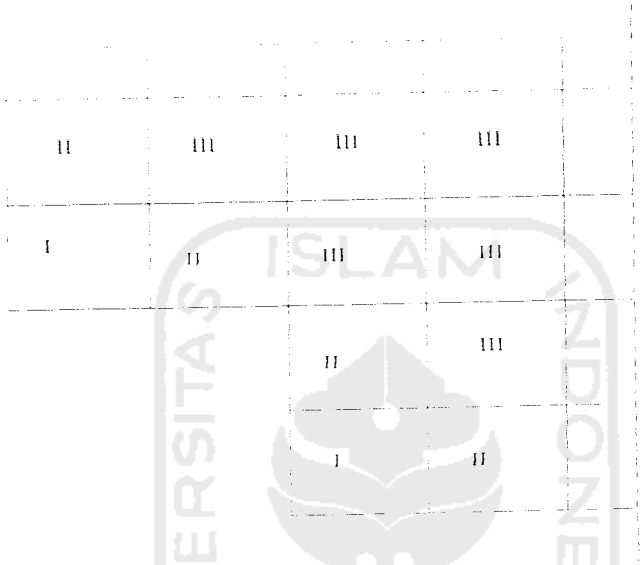
$F'_t = 303$ mpa $> f_t = 54{,}22$ mpa

8 baut A_{325} Ø 19 mm dapat digunakan

4.2 Perencanaan Pelat Lantai dan Pelat Talang Beton

4.2.1 Perencanaan Pelat Lantai

Pada Proyek Pembangunan Pasar Rakyat di Teluk Kuantan terdapat beberapa kondisi tepi pelat, dan dapat dilihat pada gambar :



Gambar 4.6 Beberapa Kondisi Tepi Pelat Lantai

4.2.1.1 Pembebanan Pelat Lantai

- fungsi bangunan : Pasar / Toserba (qL) = 2,5 KN/m^2
- spesifikasi bahan : mutu beton (f'c) = 22,5 Mpa
  mutu baja (fy) = 240 Mpa

- Perhitungan beban :

  Tebal pelat lantai (h) = 120 mm

  - Beban mati

    - Berat pelat beton = 0,12 . 24 = 2,88 KN/m^2
    - Berat pasir (5cm) = 0,05 . 18 = 0,90 KN/m^2
    - Berat spesi (3cm) = 0,03 . 21 = 0,63 KN/m^2

- Berat keramik (1cm) = 0,01 . 24 = 0,24 KN/m^2
- Perkiraan beban akibat partisi = 2,00 KN/m^2 +

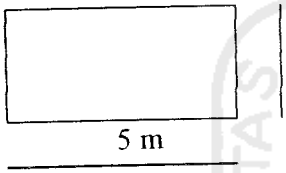
qD = 6.65 KN/m^2

- Beban hidup (qL) : qL = 2,5 KN/m^2

qu = 1,2.qD + 1,6.qL = 1,2 . 6,65 + 1,6 . 2,5 = 11,98 KN/m^2

- Menghitung distribusi momen

Contoh perhitungan pada panel III



ly = 5 m

lx = 3 m

ly/lx =1,667

Dari tabel 13.3.2.PBI 1971 (tumpuan tepi dianggap jepit elastis)

Didapat : clx = 58,7 ctx = 58,7

cly = 36 cty = 36

Mulx = 0,001.qu.lx^2.clx = 0,001 11,98. 3^2 . 58.7 = 6,32 KNm

Mutx = -0,001.qu.lx^2.ctx = -0,001 11,98. 3^2 . 58.7 = -6,32 KNm

Muly = 0,001.qu.lx^2.cly = 0,001 11,98. 3^2 . 36 = 3,88 KNm

Muty = -0,001.qu.lx^2.cty = -0,001 11,98. 3^2 . 36 = -3,88 KNm

4.2.1.2 Perhitungan Tulangan Pelat Lantai

- Perencanaan tulangan lx = tx

h = 120 mm

d = h – pb – ½ Øtul tx = 120 – 20 – ½ 8 = 96 mm

Mu = 6,32 KNm

Mu/∅ = 6,2 /0,8 = 7,9 KNm

$$Rn = \frac{Mu./\phi}{b.d^2} = \frac{7,9 .10^6}{1000.96^2} = 0,85 \text{ Mpa}$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.22,5} = 12,549$$

$$\rho_{perlu} = \frac{1}{m}\cdot\left(1-\sqrt{1-\frac{2\cdot m\cdot Rn}{fy}}\right) = \frac{1}{12,549}\cdot\left(1-\sqrt{1-\frac{2\ .12,549.\ 0,85}{240}}\right)$$

$$= 0,00365 \quad < \quad \rho_{min} = \frac{1,4}{fy} = \frac{1,4}{240} = 0,00583$$

$$\rho_b = \frac{0,85\cdot f'c}{Fy}\cdot\beta\cdot\left(\frac{600}{600+Fy}\right) = \frac{0,85\cdot 22,5}{22,5}\cdot 0,85\cdot\left(\frac{600}{600+240}\right) = 0,048$$

$$\rho_{maks} = 0,75\cdot\rho_b = 0,75\ .\ 0,048 = 0,036$$

1,33 ρ perlu = 1,33 . 0,00365 = 0,00486 < ρ min

ρ terpakai = 0,00486

As perlu = ρ terpakai.b.d = 0,00486. 1000 . 96 ≥ 0,002 . 1000.120

= 466.74 mm^2 ≥ 240 mm^2

As perlu pakai = 466,74 mm^2

Dipakai tulangan pokok ∅8 mm dengan A1∅ = 50 mm^2

Jarak tulangan : $s \le \frac{A_1\cdot b}{As_{perlu}} = \frac{50.1000}{466.74}$

= 107,12 mm ≤ 2.h = 240 mm

≤ 250 mm

Dipakai jarak (s) = 100 mm

$$\text{As ada} = \frac{A_{1\phi} \cdot b}{s_{terpakai}} = \frac{50.1000}{100} = 500 \text{ mm}^2$$

Kontrol kapasitas momen (Mn) :

$$a = \frac{As_{ada} \cdot fy}{0{,}85 \cdot f'c \cdot b} = \frac{500 \,.\, 240}{0{,}85 \,.\, 22{,}5 \,.1000} = 6{,}27 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \cdot \left(d - a/2\right) \geq 1{,}33\, Mu/\phi$$

$$= 500 \,.\, 240 \cdot \left(96 - 6{,}27/2\right) \geq 1{,}33 \,.\, 6{,}3125 \text{ KNm}$$

$$= 11{,}14 \text{ KNm} \geq 8{,}396 \text{ KNm}$$

⇒ Dipakai tulangan P8 – 100

- Perencanaan tulangan ly

h = 120 mm

d = h – pb – Øtul lx – ½ Øtul ly = 120 – 20 – 8 – 8/2 = 88 mm

Mu = 3,88KNm

Mu/∅ =3,88 /0,8 = 4.85 KNm

$$\text{Rn} = \frac{Mu./\phi}{b.d^2} = \frac{4{,}85 \,.\, 10^6}{1000 \,.\, 88^2} = 0{,}627 \text{ Mpa}$$

$$\text{m} = \frac{fy}{0{,}85.f'c} = \frac{240}{0{,}85 \,.\, 22{,}55} = 12{,}549$$

$$\rho_{perlu} = \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot Rn}{fy}}\right) = 0.0027 < \rho_{min} = \frac{1{,}4}{240} = 0{,}00583$$

$$\rho_b = \frac{0{,}85 \cdot f'c}{fy} \cdot \beta \cdot \left(\frac{600}{600 + fy}\right) = 0{,}048$$

$$\rho_{maks} = 0{,}75 \cdot \rho_b = 0{,}75 \,.\, 0{,}05376 = 0{,}036$$

1,33 ρ perlu = 1,33 . 0,0027 = 0,0035 < ρ min

ρ terpakai = 0,0035

As perlu = ρ terpakai.b.d = 0,0035 . 1000 .88 ≥ 0,002 . 1000.120

= 310,7 mm^2 ≥ 240 mm^2

As perlu pakai = 350 mm^2

Dipakai tulangan pokok ∅8 mm dengan A1∅ = 50,266 mm^2

Jarak tulangan : $s \leq \frac{A_1 \cdot b}{As_{perlu}}$ = 143 mm

≤ 2.h = 240 mm

≤ 250 mm

Dipakai jarak (s) = 140 mm

As ada = $\frac{A_{1\ \phi} \cdot b}{s_{terpakai}} = \frac{50,266.1000}{140}$ = 314 mm^2

Kontrol kapasitas momen (Mn) :

$$a = \frac{As_{ada} \cdot fy}{0,85 \cdot f'c \cdot b} = \frac{314\ .\ 240}{0,85\ .\ 22,55\ .\ 1000} = 4,51 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \cdot \left(d - a/2\right) \geq 1,33\, Mu/\phi$$

= 7,39 KNm ≥ 6,45 KNm

⇒ Dipakai tulangan P8 – 140

- Perencanaan tulangan ty

h = 120 mm

d = h – pb – ½ ∅tul ty = 120 – 20 – ½ 8 = 96 mm

Mu = 3,88 KNm

Mu/∅ =3.88 /0,8 = 4,85 KNm

$$Rn = \frac{Mu./\phi}{b.d^2} = \frac{4,85 \,.\, 10^6}{1000 \,.\, 96^2} = 0,86 \text{ Mpa}$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85 \,.\, 22,55} = 12,549$$

$$\rho_{perlu} = \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot Rn}{fy}}\right) = 0,0022 < \rho_{min} = \frac{1,4}{240} = 0,00583$$

$$\rho_b = \frac{0,85 \cdot f'c}{Fy} \cdot \beta \cdot \left(\frac{600}{600 + Fy}\right) = 0,048$$

$$\rho_{maks} = 0,75 \cdot \rho_b = 0,75 \,.\, 0,048 = 0,036$$

1,33 ρ perlu = 1,33 . 0,0022 = 0,0049

ρ terpakai = 0,0049

As perlu = ρ terpakai.b.d = 0,0049 . 1000 .96 ≥ 0,002 . 1000.120

= 284 mm^2 ≤ 240 mm^2

As perlu pakai = 300 mm^2

Dipakai tulangan pokok ∅8 mm dengan A1∅ = 50,266 mm^2

Jarak tulangan : $s \leq \frac{A_1 \cdot b}{As_{perlu}} = 167,6$ mm

≤ 2.h = 240 mm

≤ 250 mm

Dipakai jarak (s) = 160 mm

$$\text{As ada} = \frac{A_{1\phi} \cdot b}{s_{terpakai}} = 314 \text{ mm}^2$$

Kontrol kapasitas momen (Mn) :

$$a = \frac{As_{ada} \cdot fy}{0{,}85 \cdot f'c \cdot b} = \frac{314 \,.\, 240}{0{,}85 \,.\, 22{,}5 \,.\, 1000} = 3{,}94 \text{ mm}$$

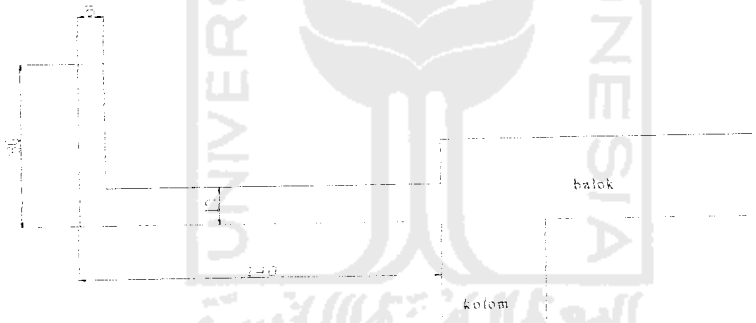
$$Mn = As_{ada} \cdot fy \cdot \left(d - a/2\right) \geq 1{,}33 \, Mu/\phi$$

$= 7{,}09\text{KNm} \geq 6{,}45 \text{ KNm}$

⇒ Dipakai tulangan P8 – 160

4.2.2 Perencanaan Talang Beton

Pada Proyek Pembangunan Pasar Rakyat di Teluk Kuantan menggunakan talang beton yang dapat dilihat seperti pada gambar :



Gambar 4.7 Talang Beton

Tebal pelat, h = 10 mm

Berat lisplang (P_D) = 0.8 . 0.08 . 23 = 1,472 KN/m'

Beban hidup (P_L) = 1 KN/m'

Pu = 1.2 . P_D + 1.6 . P_L

= 1,2 . 1,472 + 1,6 . 1

= 3,366 KN/m'

Berat pelat (W_D) $= 0,10 \cdot 23 = 2,3$ KN/m^2

$W_L = 1$ KN/m^2

$W_U = 1,2 \cdot 2,3 + 1,6 \cdot 1$

$= 4,36$ KN/m^2

$Mu = \frac{1}{2} \cdot 4,36 \cdot 1,4^2 + 3,666 \cdot 1,4 = 9,4$ KNm/m'

$Mn = \frac{Mu}{\phi} = \frac{9,4}{0,8} = 11,75$ KNm/m'

Digunakan tulangan P 8 mm, penutup beton = 15 mm

ds = Pb + ½ D = 15 + ½ . 8 = 19 mm

maka d = h – ds = 100 – 19 = 81 mm

$$Rn = \frac{Mn}{b \cdot d^2} = \frac{11,75 \cdot 10^6}{0,85 \cdot 81^2} = 2,106 \text{ Mpa}$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.22,5} = 12,549$$

$$\rho_{perlu} = \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot Rn}{fy}}\right) = \frac{1}{12,549} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot 12,549 \cdot 2,106}{240}}\right)$$

$$= 0,0093 > \rho_{min} = \frac{1,4}{240} = 0,00583$$

$$\rho_b = \frac{0,85 \cdot f'c}{Fy} \cdot \beta \cdot \left(\frac{600}{600 + Fy}\right) = \frac{0,85 \cdot 22,5}{22,5} \cdot 0,85 \cdot \left(\frac{600}{600 + 240}\right) = 0,048$$

$\rho_{maks} = 0,75 \cdot \rho_b = 0,75 \cdot 0,048 = 0,036$

ρ pakai = 0.0093

Luas tulangan pokok As = ρ. b. d = 0,0093 . 1000 . 81 = 753,3 mm^2

$$s \leq \frac{50 \,.\, 1000}{753.3} = 66{,}37 \ \ mm$$

Dipakai tulangan P8 – 60

Luas tulangan susut Asst = 0,002 b.h = 0,002.1000.100 = 200 mm^2

$$s \leq \frac{50 \,.\, 1000}{200} = 250 \ \ mm$$

Dipakai tulangan P8 – 200

Cek geser :

Vu = 4,36 . 1,4 + 3,366 = 9,47 KN

$$\phi Vu = 0.6\left[\tfrac{1}{6}\sqrt{fc'} \,.\, bw.d\right] = 0{,}6\left[\tfrac{1}{6}\ \sqrt{22{,}5}\ \ 1000.81\right] = 38421 \ \ N$$

$$= 38{,}42 \ \ KN > Vu = 9.47 \quad ok$$

4.3 Perencanaan Struktur dengan daktilitas penuh

Pada perencanaan ulang gedung Pasar Rakyat Teluk Kuantan Tahap – 2 ini, perencanaan portal dianalisis dengan SAP2000 dengan analisis struktur tiga (3) Dimensi. 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 pasar / toserba, maka menurut Peraturan Pembebanan Indonesia 1983 (Tabel 3.3) untuk perencanaan beban gempa, beban hidup direduksi sebesar 0,8. Adapun perhitungan gaya geser dasar horizontal total akibat gempa adalah sebagai berikut :

4.3.1 Berat total bangunan

Perhitungan berat bangunan pada blok B

(gambar dapat dilihat pada lampiran)

Tabel 4.1 Perhitungan Berat Bangunan

Lantai	Elemen	b (m) (a)	h (m) (b)	l (m) (c)	Faktor berat (KN)... (d)	Faktor reduksi (e)	Total beban (KN) (a)x(b)x(c)x(d)x(e)
L. Dasar							
	B. Mati						
	Sloof	0.4	0.6	391	24	1	2,252.16
	Kolom	0.4	0.4	116	24	1	445.44
	Dinding	1	2	122	2.5	1	610.00
Total beban (Wt1)							**3,307.60**
Lt 1							
	B. Mati						
	pelat	25	0.12	36	24	1	2,592.00
	balok	0.3	0.43	216	24	1	668.74
		0.3	0.33	175	24	1	415.80
		0.25	0.18	125	24	1	135.00
	kolom	0.4	0.4	168	24	1	645.12
	dinding	1	4	121	2.5	1	1,210.00
	keramik	25	0.01	36	24	1	216.00
	spesi	25	0.03	36	21	1	567.00
	pasir	25	0.05	36	18	1	810.00
	penggantung	25	1	36	0.18	1	162.00
	B. Hidup	25	1	36	2.5	0.8	1,800.00
Total beban (Wt2)							**9,221.66**
Lt 2							
	B. Mati						

	pelat	25	0.12	36	24	1	2,592.00
	balok	0.3	0.43	216	24	1	668.74
		0.3	0.33	175	24	1	415.80
		0.25	0.18	125	24	1	135.00
	kolom	0.4	0.4	168	24	1	645.12
	dinding	1	4	121	2.5	1	1,210.00
	keramik	25	0.01	36	24	1	216.00
	spesi	25	0.03	36	21	1	567.00
	pasir	25	0.05	36	18	1	810.00
	penggantung	25	1	36	0.18	1	162.00
	B. Hidup	20	1	36	2.5	0.8	1,440.00
		5	1	18	2.5	0.8	180.00
Total beban (Wt3)							**9,041.66**
Lt 3							
	B. Mati						
	pelat	25	0.12	36	24	1	2,592.00
	balok	0.3	0.43	216	24	1	668.74
		0.3	0.33	175	24	1	415.80
		0.25	0.18	125	24	1	135.00
	kolom	0.4	0.4	168	24	1	645.12
	dinding	1	4	121	2.5	1	1,210.00
	keramik	25	0.01	36	24	1	216.00
	spesi	25	0.03	36	21	1	567.00
	pasir	25	0.05	36	18	1	810.00
	Talang beton	1.4	0.07	50	24	1	117.60
	penggantung	25	1	36	0.18	1	162.00
	B. Hidup	20	1	36	2.5	0.8	1,440.00
		5	1	18	2.5	0.8	180.00
Total beban (Wt4)							**9,159.26**
Lt 4							

	B. Mati						
	ring balk	0.2	0.33	70	24	1	110.88
		0.2	0.23	144	24	1	158.98
	kolom	0.3	0.3	47.25	24	1	102.06
	dinding	1	1.2	122	2.5	1	366.00
	penggantung	15	1	36	0.18	1	97.20
	kuda2 (WF200)	1	1	160	0.306	1	48.96
	(WF 148)	1	1	90.58	0.211	1	19.11
	gording	1	1	1364	0.0613	1	83.61
	atap	35.8	1	36	0.1	1	128.88
	B. Hidup	35.8	1	36	0.2	0.8	206.21
Total beban (Wt5)							**1,489.09**
W. TOTAL							**32,219.26**

W total = **Wt1 + Wt2 + Wt3 + Wt4 + Wt5**

= 32.219,26 KN

4.3.2 Waktu getar bangunan

$Tx = Ty = 0{,}06\ H^{(3/4)} = 0{,}06\ .\ 16.5^{(3/4)} = 0{,}491$ dt

4.3.3 Koefisien getar bangunan

Tx = Ty = 0,491 dt; Zona 5 dan jenis tanah lunak → C = 0,03

4.3.4 Faktor keutamaan I dan faktor jenis struktur K

I = 1,0 K = 1,0

4.3.5 Gaya geser horizontal akibat gempa

Vx = Vy = C.I.K.Wt = 0,03 . 1,0 . 1,0 . 32.219,26

= 966.577 KN

4.3.6 Distribusi gaya horizontal akibat gempa ke sepanjang tinggi gedung

a. Arah x

H/A = 15.78 / 25 = 0.631 < 3,0

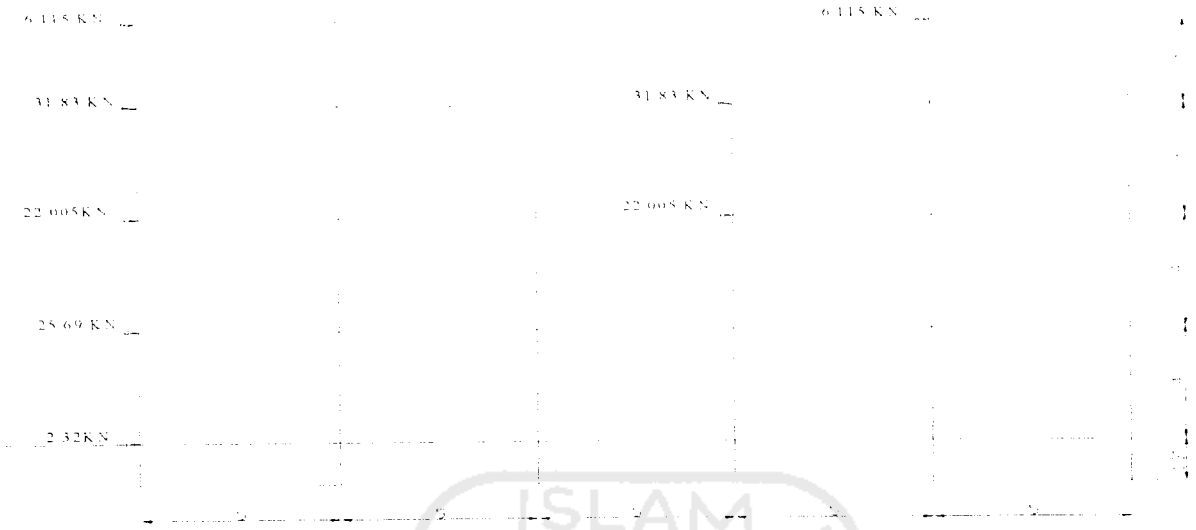
b. Arah y

H/A = 15.78 / 36 = 0.438 < 3,0

Tabel 4.2 Distribusi Gaya Geser Dasar Horizontal Total Akibat Gempa

Lantai	Hi (m)	Wi (KN)	Wi.Hi (KNm)	$F_i = \frac{W_i.H_i}{\sum W_i.H_i} \cdot V$	Untuk Tiap Portal	
					1/7. Fi x (KN)	1/6 Fiy (KN)
5	15.78	1,489.09	23,497.84	85.63	12.23	14.27
4	13.35	9,159.26	122,276.12	445.62	63.66	74.27
3	9.35	9,041.66	84,539.52	308.09	44.01	51.35
2	5.35	9,221.66	49,335.88	179.80	25.69	29.97
1	1.35	3,307.60	4,465.26	16.27	2.32	2.71
			260,616.78			

Gambar 4.8 Beban Gempa Arah – X Pada Portal A



Gambar 4.9 Beban Gempa Arah – X Pada Portal B

Gambar 4.10 Beban Gempa Arah – Y Pada Portal 12

4.4 Desain Balok

4.4.1 Desain Balok Anak

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

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

4.4.1.1 Desain Tulangan Lentur

Berikut ini contoh hitungan balok anak pada batang – 192

- f'c = 22,5 Mpa
- fy deform = 360 Mpa
- tul pokok = 16 mm
- tulangan sengkang = 10 mm

⟶ untuk f'c ≤ 30 Mpa ⇒ $\beta_1 = 0{,}85$

f'c > 30 Mpa ⇒ $\beta_1 = 0{,}85\text{-}0{,}008\ (f'c\text{-}30) \geq 0{,}65$

A. Tulangan Tumpuan

Mu = 74,14 KNm (-)

$$Mu/\phi = \frac{74{,}14}{0{,}8} = 92{,}67 \text{ KNm}$$

Dimensi rencana balok anak $250/400$, maka :

Tinggi efektif balok ($d_{\text{diketahui}}$) = $h_{\text{diketahui}}$ – Pb - $\varnothing_{\text{sengkang}}$ - ½ $\varnothing_{\text{tul.rencana}}$

d = 400 – 40 – 10 – ½ 16 = 340 mm

$$\rho b = \frac{0{,}85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0.85.22{,}5}{360}.0{,}85\left(\frac{600}{600+360}\right) = 0{,}0282$$

$\rho_{maks} = 0,75 \cdot \rho b = 0,75 \cdot 0,0282 = 0,0212$

$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{360} = 0,0039$

$\rho_{pakai} = 0,5 \cdot \rho_{maks} = 0,5 \cdot 0,0212 = 0,0106$

$m = \frac{fy}{0,85.f'c} = \frac{360}{0,85.22,5} = 18.82$

$Rn = \rho\, fy\, (1 - \frac{1}{2}\rho\, m) = 0,0106 \cdot 360\, (1 - \frac{1}{2} \cdot 0,0106 \cdot 18,82) = 3,431$ MPa

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

$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{92,67.10^6}{3,431.250}} = 328,7$ mm < d = 340 mm, maka dipakai tulangan sebelah

$Rn_{ada} = \frac{Mu/\phi}{b.d^2_{ada}} = \frac{92,67.10^6}{250.340^2} = 3,2$ MPa

$\rho_{ada} = \frac{Rn_{ada}}{Rn}\rho = \frac{3,2}{3,431}0,0106 = 0,0098 > \rho_{min} = 0,0039$

$\rho_{pakai} = 0,0098$

$As = \rho_{ada}.b.d_{ada} = 0,0098. 250 \cdot 340 = 833$ mm^2

Dipakai diameter tulangan D16, maka : $A_1Ø = 201$ mm^2

$n = \frac{As}{A_1\phi} = \frac{833}{201} = 4,1$ batang

Dipakai tulangan memanjang **5D16**, maka :

$As_{ada} = 5 \cdot 201 = 1005$ mm^2 > As = 833 mm^2

Jarak bersih antar tulangan = $\frac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tul.}{(n-1)}$

$= \frac{250 - 2.40 - 2.10 - 5.16}{(5-1)}$

= 17,5 mm < 25 mm maka dipakai tulangan dua lapis

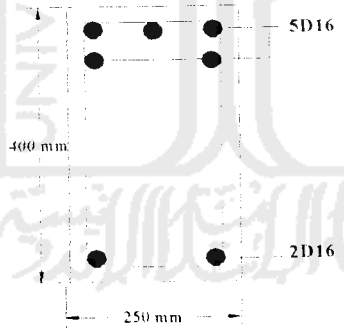
Kontrol Kapasitas Lentur yang terjadi :

$a = \frac{As_{ada}\, fy}{0,85.f'c\ b} = \frac{1005\ .360}{0,85.22,5.250} = 75,67$ mm

$Mn = As_{ada} \cdot fy \cdot \left(d - a/2\right) \geq Mu/\phi$

$= 1005\ .\ 360\ (\ 340 - \frac{63,02}{2}\ )\ 10^{-6} = 92,95$ KNm

$= 1009,32$ KNm > $Mu/\phi = 92,67$ KNm (Ok!)



Gambar 4.11 Tulangan Tumpuan Balok Anak Frame 192

B. Tulangan Lapangan

Mu = 46,32 KNm

$Mu/\phi = \frac{46,32}{0,8} = 57,9$ KNm

Tinggi efektif balok ($d_{diketahui}$) = $h_{diketahui}$ – Pb - $\emptyset_{sengkang}$ - ½ $\emptyset_{tul.rencana}$

d = 400 – 40 – 10 – ½ 16 = 340 mm

$$\rho b = \frac{0{,}85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0{,}85.22{,}5}{360}.0{,}85\left(\frac{600}{600+360}\right) = 0{,}0282$$

$\rho_{maks} = 0{,}75 \;.\; \rho b = 0{,}75 \;.\; 0{,}0282 = 0{,}0212$

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{360} = 0{,}0039$$

$\rho_{pakai} = 0{,}5 \;.\; \rho_{maks} = 0{,}5 \;.\; 0{,}0212 = 0{,}0106$

$$m = \frac{fy}{0{,}85.f'c} = \frac{360}{0{,}85.22{,}5} = 18{,}82$$

$Rn = \rho\, fy\,(1 - \frac{1}{2}\rho\, m) = 0{,}0106 \;.\; 360\;(1 - \frac{1}{2} \;.\; 0{,}0106 \;.\; 18{,}82) = 3{,}431$ MPa

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

$$d_{perlu} = \sqrt{\frac{Mu}{\frac{\phi}{Rn.b}}} = \sqrt{\frac{57{,}9 \;.10^6}{3{,}431.250}} = 259{,}8 \text{ mm} < d = 340 \text{ mm}$$, maka dipakai tulangan sebelah

$$Rn_{ada} = \frac{Mu}{\frac{\phi}{b.d^2_{ada}}} = \frac{57{,}9.10^6}{250.340^2} = 2{,}00 \text{ MPa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}\rho = \frac{2{,}00}{3{,}431}0{,}0106 = 0{,}0061 > \rho_{min} = 0{,}0039$$

$\rho_{pakai} = 0{,}0061$

$As = \rho_{ada}.b.d_{ada} = 0{,}0061.250 \;.\; 340 = 519 mm^2$

Dipakai diameter tulangan D16, maka : $A_1\emptyset = 201\ mm^2$

$$n = \frac{As}{A_1\phi} = \frac{519}{201} = 2{,}5 \text{ batang}$$

Dipakai tulangan memanjang **3D16**, maka :

$As_{ada} = 3 \cdot 201 = 603 \text{ mm}^2 > As = 518 \text{ mm}^2$

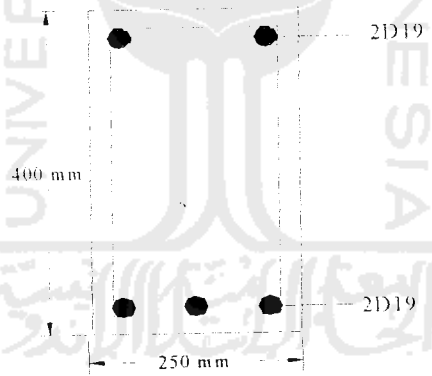
Kontrol Kapasitas Lentur yang terjadi :

$$a = \frac{As_{ada}\, fy}{0{,}85.f'c\, b} = \frac{603 .360}{0{,}85.22{,}5.250} = 45{,}40 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \cdot \left(d - a/2\right) \geq Mu/\phi$$

$$= 603 \cdot 360 \left(340 - \frac{45{,}40}{2} \right) 10^{-6} = 68{,}88 \text{ KNm}$$

$$= 68{,}88 \text{ KNm} > Mu/\phi = 57{,}9 \text{ KNm} \qquad \text{(Ok!)}$$



Gambar 4.12 Tulangan Lapangan Frame192

4.4.1.2 Perencanaan Tulangan Geser Balok

- **Gaya Geser Dukungan**

  Vu dukungan = 74,25 KN

  maka $Vu/\phi = \frac{74{,}25}{0{,}6} = 123{,}75$ KN

- **Gaya Geser Tengah Bentang**

Vu tengah bentang = 5,95KN

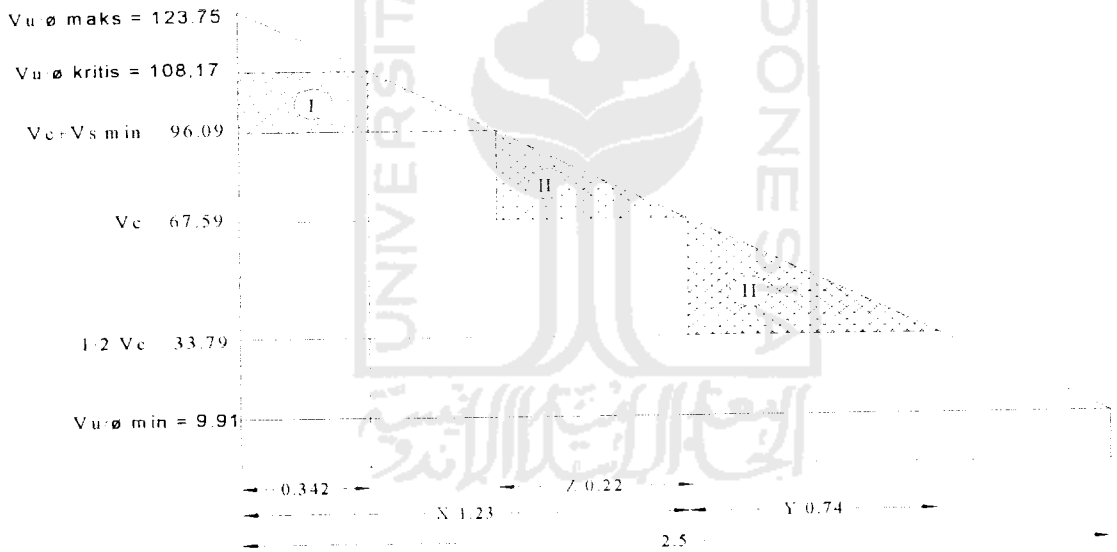
maka $Vu/\phi = \frac{5,95}{0,6} = 9,91\text{KN}$

Tegangan Geser Beton (Vc) :

$$Vc = \left(\frac{1}{6}\sqrt{fc'}\right).b.d = \left(\frac{1}{6}\sqrt{22,5}\right).250.342 = 67593\ \text{N} = 67,59\ \text{KN}$$

$Vu/\phi = 123,75 > Vc = 67,59$, maka perlu tulangan geser.

$$Vs_{min} = \frac{1}{3}.b.d = \frac{1}{3}.250.342.10^{-3} = 28,5\ \text{KN}$$



Gambar 4.13 Diagram Tegangan Geser Balok Anak

$$Vu/\phi kritis = \frac{((2,5-0,342)(123,75-9,91))}{2,5} + 9,91 = 108,17\ \text{KN}$$

$$x = \frac{(123,75-67,59).2,5}{123,75-9,91} = 1,23\ \text{m}$$

$$y = \frac{(67{,}59 - 33{,}8).2{,}5}{123{,}75 - 9{,}91} = 0{,}74 \text{ m}$$

$$z = \frac{9{,}91 \,.\, 1{,}23}{123{,}75 - 67{,}59} = 0{,}22 \text{ m}$$

Daerah I :

$(Vc + Vs_{min}) = 96{,}09 < Vu/\phi_{kritis} = 108{,}17 < 3\ Vc = 202{,}77$ KN

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

Jarak sengkang :

$$s \leq \frac{Av.fy.d}{Vu/\phi - Vc} = \frac{157.240.340}{123{,}75 - 67{,}59} \,.\, 10^{-3} = 229 \text{ mm}$$

$$\leq d/2 = 340/2 = 170 \text{ mm}$$

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

Jadi dipakai tulangan sengkang **P10 – 170 mm**

Daerah II :

$$Vu/\phi = Vc + Vs_{min} = 96{,}09 \text{ KN}$$

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

$$s \leq \frac{Av.fy.d}{Vs_{min}} = \frac{157.240.340}{28{,}5} \,.\, 10^{-3} = 452 \text{ mm}$$

$$\leq d/2 = 340/2 = 170 \text{ mm}$$

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

Jadi dipakai tulangan sengkang **P10 – 170 mm**

4.4.2 Desain Balok Induk

4.4.2.1 Portal Arah – X

Berikut ini contoh perhitungan balok BI portal – X as A (bentang 7 - 8) lantai 1

A. Tulangan Tumpuan

Momen tumpuan diambil yang paling besar dari semua kombinasi, dan momen yang digunakan adalah yang terbesar dari kedua tumpuan (kiri & kanan)

Dipakai dimensi rencana 300/400

fc' = 22,5 Mpa fy = 360 Mpa

- Tulangan tumpuan momen negatif

Mu = 96.74 Nm (-)

$$\frac{Mu}{\phi} = \frac{96,74}{0,8} = 120,93 \text{ KNm}$$

$$\rho b = \frac{0,85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5}{360}.0,85\left(\frac{600}{600+360}\right) = 0,0282$$

$$\rho_{maks} = 0,75 \,.\, \rho b = 0,75 \,.\, 0,0282 = 0,0212$$

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{360} = 0,0039$$

$$\rho_{pakai} = 0,5 \,.\, \rho_{maks} = 0,5 \,.\, 0,0212 = 0,0106$$

$$m = \frac{fy}{0,85.f'c} = \frac{360}{0,85.22,5} = 18.82$$

$$Rn = \rho\, fy\,(1 - \tfrac{1}{2}\rho\, m) = 0,0106 \,.\, 350\,(1 - \tfrac{1}{2} \,.\, 0,0106 \,.\, 18,82) = 3,431 \text{ MPa}$$

d = 400 – 40 – 10 – ½ 19 = 340 mm

$$d_{perlu} = \sqrt{\frac{Mu}{\phi}} = \sqrt{\frac{120,93 \ . \ 10^6}{3,431 \ . \ 300}} = 342.76 \text{ mm} > d_{perlu} = 340 \text{ mm}$$

maka dipakai tulangan rangkap

$As1 = \rho_1 \ . \ b \ . \ d_{diketahui}$

$As1 = 0,0106.300.340 = 1081,2 \text{ mm}^2$

Dipakai tulangan Ø19 dengan $A1\emptyset = 284 \text{ mm}^2$

$$\text{jumlah tulangan (n)} = \frac{As}{A1\phi} = \frac{1081,2}{284} = 3,4 \text{ batang}$$

dipakai **4D19**, maka $As_{ada} = 2 \ . \ 284 = 1136 \text{ mm}^2 > As$

$$a = \frac{As1.fy}{0,85.fc'.b} = \frac{1081,2 \ . \ 360}{0,85.22,5.300} = 67,84 \text{ mm}$$

$$Mn_1 = As_1 \cdot fy \cdot \left(d_{diketahui} - a/2\right) < Mu/\phi$$

$$Mn_1 = 1081,2.360. \left(340 - 67,84/2 \right) .10^{-6} = 119,14 \text{ KNm}$$

$$< Mu/\phi = 120,93 \text{ KNm}$$

$$Mn_2 = Mu/\phi - Mn1$$

$$Mn_2 = 120,93 - 119,14 = 1,79 \text{ KNm}$$

$$fs' = 600 \cdot \left\{ 1 - \frac{0,85 \cdot f'c \cdot \beta_1}{(\rho - \rho') \cdot fy} \cdot \frac{d'}{d_{diketahui}} \right\} \leq fy$$

$$fs' = 600 \cdot \left\{ 1 - \frac{0,85 \cdot 22,5 \cdot 0,85}{0.0106 \cdot 360} \cdot \frac{60}{340} \right\} = 148,9 \text{ Mpa} \leq 360 \text{ Mpa}$$

$$As' = \frac{Mn_2}{fs' \cdot \left(d_{diketahui} - d'\right)}$$

$$As' = \frac{1,79.10^6}{148,9 \cdot (340-60)} = 42,93 \text{ mm}^2$$

Dipakai tulangan Ø19 dengan A1Ø = 284 mm^2

$$\text{jumlah tulangan } (n') = \frac{As'}{A1\phi} = \frac{42,93}{284} = 0,15 \text{ batang}$$

dipakai **2D19**, maka $As_{ada} = 2 \cdot 284 = 568 \text{ mm}^2 > As'$

$As = As1 + As' = 1136 + 568 = 1704 \text{ mm}^2$

Kontrol kapasitas lentur

$$\rho = \frac{As}{b \cdot d_{diketahui}}$$

$$\rho = \frac{1704}{300 \cdot 340} = 0.0167$$

$$\rho' = \frac{As'}{b \cdot d_{diketahui}}$$

$$\rho' = \frac{568}{300.340} = 0.0055$$

$$fs' = 600 \cdot \left\{1 - \frac{0,85 \cdot 22,5 \cdot 0,85}{(0.0167 - 0.0055) \cdot 360} \cdot \frac{60}{340}\right\} = 173,10 \text{ Mpa} \leq 360 \text{ Mpa}$$

$$a = \frac{As \cdot fy - As' \cdot fs'}{0,85 \cdot f'c \cdot b} = \frac{1704 \cdot 360 - 568 \cdot 173,10}{0,85 \cdot 22,5 \cdot 300} = 89,78 \text{ mm}$$

$$Mn = (As \cdot fy - As' \cdot fs') \cdot \left(d_{diketahui} - a/2\right) + (As' \cdot fs') \cdot (d_{diketahui} - d')$$

$$Mn = (1704 \cdot 360 - 568 \cdot 173,10) \cdot \left(340 - 89,78/2\right) + (568 \cdot 173,10) \cdot (340 - 60)$$

$$Mn = 179,5 \text{ KNm} > Mu/\phi = 120,93 \text{ KNm} \quad \text{-OK-}$$

$$\text{Jarak bersih antar tulangan} = \frac{h - 2.Pb - 2.\phi\ sengkang - n.\phi\ tul.}{(n-1)}$$

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

= 41 mm > 25 mm, maka dipakai tulangan 1 lapis

- Tulangan tumpuan momen positif

Dikarenakan koefisien gempa dasar lokasi proyek ini ada di wilayah 5 maka beban gempa yang terjadi terhadap proyek ini tidak terlalu besar, maka penggunaan momen positif tidak relevan lagi untuk digunakan pada perhitungan luas tulangan yang akan digunakan. Untuk menghitung tulangan yang digunakan pada bangunan tahan gempa SNI mengisyaratkan untuk mengambil nilai terbesar dari keterangan berikut :

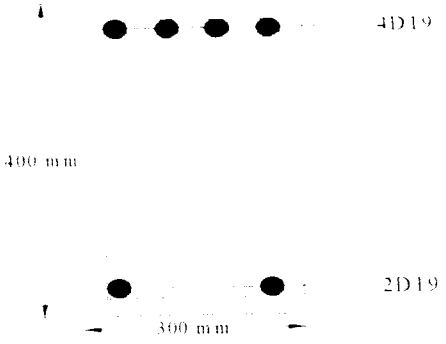
1. Jumlah tulangan momen positif diambil ≥ 50 % jumlah tulangan momen negative pada tumpuan
2. Jumlah tulangan momen positif diambil ≥ 30 % jumlah tulangan momen positif pada lapangan.

Dikarenakan besarnya momen negative tumpuan dengan momen positif lapangan selisihnya tidak terlalu besar maka peraturan no 2 tidak perlu digunakan.

Maka jumlah tulangan tumpuan momen positif = 50% . jmlh tul tumpuan

= 50% . 4 tulangan D19 = 2 tulangan D19

Maka digunakan tulangan **2D19**



Gambar 4.14 Tulangan Tumpuan Balok Induk

B. Tulangan Lapangan

Mu = 41,21 KNm (+)

$$\frac{Mu}{\phi} = \frac{41,21}{0,8} = 51,51 \text{ KNm}$$

$$\rho b = \frac{0,85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5}{360}.0,85\left(\frac{600}{600+360}\right) = 0,0282$$

$$\rho_{maks} = 0,75 \;.\; \rho b = 0,75 \;.\; 0,0282 = 0,0212$$

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{360} = 0,0039$$

$$\rho_{pakai} = 0,5 \;.\; \rho_{maks} = 0,5 \;.\; 0,0212 = 0,0106$$

$$m = \frac{fy}{0,85.f'c} = \frac{360}{0,85.22,5} = 18,82$$

$$Rn = \rho\, fy\,(1 - \tfrac{1}{2}\rho\, m) = 0,0106 \;.\; 350\;(1 - \tfrac{1}{2} \;.\; 0,0106 \;.\; 18,82) = 3,431 \text{ Mpa}$$

$$d_{perlu} = \sqrt{\frac{Mu}{\phi}} = \sqrt{\frac{51,51 \;.\; 10^6}{3,431 \;.\; 300}} = 223,7 \text{ mm} < d_{perlu} = 340 \text{ mm},$$

maka dipakai tulangan sebelah.

$$Rn_{ada} = \frac{\frac{Mu}{\phi}}{b.d^2} = \frac{51,51.10^6}{300.340^2} = 1,48 \text{ Mpa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}.\rho_{pakai} = \frac{1,48}{3,431}.0,0106 = 0,0046 > \rho_{min} = 0,0046$$

$$\rho_{ada} = \rho_{pakai} = 0,0046$$

$As_{perlu} = \rho_{ada}.b.d = 0,0039.300.340 = 469mm^2$

Dipakai tulangan Ø19 dengan A1Ø $= 284 \text{ mm}^2$

$$\text{jumlah tulangan (n)} = \frac{As_{perlu}}{A1\phi} = \frac{469}{284} = 1,6 \text{ batang}$$

dipakai **2D19**, maka $As_{ada} = 2 \,.\, 284 = 568 \text{ mm}^2 > As_{perlu}$

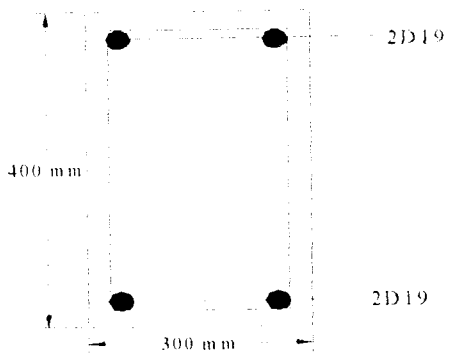
Kontrol kapasitas momen nominal :

$$a = \frac{As_{ada}.fy}{0,85.fc'.b} = \frac{568 \,.\, 360}{0,85.22,5.300} = 35.63 \text{ mm}$$

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

$$= 568 \,.\, 360 \,.\, (340 - {}^{35,63}/_{2}) .10^{-6}$$

$$= 65,88 \text{ KNm} > {}^{Mu}/_{\phi} = 51,51 \text{ KNm} \rightarrow \text{OK!}$$



Gambar 4.15 Tulangan Lapangan Balok Induk

C. Momen Nominal Aktual Balok

1) Momen Aktual Balok Negatif Tumpuan kiri = kanan

tulangan atas = 4D19 dengan $As_{ada} = 1136\ mm^2$

tulangan bawah = 2D19 dengan $As_{ada} = 567\ mm^2$

$$\rho = \frac{As_{ada}}{b.d_{pakai}} = \frac{1136}{300.340} = 0{,}011$$

$$\rho' = \frac{As'_{ada}}{b.d_{pakai}} = \frac{567}{300.340} = 0{,}0055$$

$$\rho_1 = \rho - \rho' = 0{,}011 - 0{,}0055 = 0{,}0055$$

$$fs' = 600\left\{1 - \frac{0{,}85.f'c.\beta_1\ d'}{(\rho - \rho').fy\ d}\right\} = 600\left\{1 - \frac{0{,}85.22{,}5.0{,}85\ \ 60}{0{,}0055.360\ \ 340}\right\}$$

$= 269{,}3$ Mpa

$fs' < fy$ dipakai $fs' = 269{,}3$ Mpa

$$a = \frac{(As_{ada}.fy) - (As'_{ada}.fs')}{0{,}85.fc'.b} = \frac{(1136.360) - (567.269{,}3)}{0{,}85.22{,}5.300}$$

$= 44{,}66\ mm^2$

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

$$= (1136.360 - 567.269{,}3).(340 - {}^{44{,}66}/_{2}) . 10^{-6} = 81{,}41\ KNm$$

$$Mn_2 = (As'_{ada}.fs').(d - d') = (567.269{,}3).(340 - 100) . 10^{-6} = 36{,}64\ KNm$$

$$Mnak^{-} = Mn1 + Mn_2 = 81{,}41 + 36{,}64 = 118{,}05\ KNm$$

2) Momen Aktual Balok Positif

$$\rho_{aktual} = \frac{As_{ada}}{b.d_{pakai}} = \frac{567}{300.340} = 0{,}0055$$

$Rn = \rho.fy(1-1/2.\rho..m) = 0{,}0055.360.(1\text{-}1/2.\ 0{,}0055.18{,}82) = 1{,}87$ Mpa

$Mnak^{+} = Rn.b.d^2 = 1{,}87.300.340^2.10^{-6} = 64{,}85$ KNm

D. 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 $Vu,b = 1{,}05\ (\ V_{D,b} + V_{L,b} + 4/k\ .V_{E,b}\ )$

$V_D = 53{,}31$ KN ; $V_L = 7{,}10$ KN ; $V_E = 18{,}40$ KN

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

$$Vu,b = 0{,}7.1{,}25\left[\frac{118{,}5 + 64{,}85}{5}\right] + 1{,}05(53{,}31 + 7{,}10) = 95{,}52 \text{ KN}$$

Dengan syarat tidak lebih besar dari :

$Vu,b = 1{,}05\ (53{,}31 + 7{,}10 + 4/1\ .\ 18{,}40\ ) = 140{,}71$ KN

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

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

$$= \left[63{,}43 - 0{,}7.1{,}25\left(\frac{118{,}5 + 64{,}85}{5}\right)\right] +$$

$$\frac{5-0{,}39}{5}\left[95{,}52 - \left[63{,}43 - 0{,}7.1{,}25\left[\frac{118{,}5 + 64{,}85}{5}\right]\right]\right]$$

$= 90{,}51$ KN

Gambar 4.16 Diagram Tegangan Geser Balok

1) dalam daerah sendi plastis

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

Vu,b = 90,51 KN

Vc = 0

$$\frac{Vu,b}{\phi} = \frac{90,51}{0,6} = 150,85 \text{ KN}$$

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

Jarak sengkang :

$$s \leq \frac{Av.fy.d}{\frac{Vu,b}{\phi} - Vc} = \frac{157.240.340}{150,85 - 0} \cdot 10^{-3} = 84,92 \text{ mm}$$

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

Jadi dipakai tulangan geser **P10 – 80 mm**

2) Diluar sendi plastis

Diambil jarak sejauh 2h = 2.400 = 800 mm

$$Vu,b = 90,51 - \frac{90,51.(0,8-0,34)}{(2,5-0,34)} = 71,23 \text{ KN}$$

$$Vc = 1/6.\sqrt{fc'}.b.d = 1/6.\sqrt{22,5}.300.340.10^{-3} = 80.64 \text{ KN}$$

$$Vs = \frac{Vu,b}{\phi} - Vc = \frac{71,23}{0,6} - 80,64 = 38,07 \text{ KN}$$

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

Jarak sengkang :

$$s \leq \frac{Av.fy.d}{Vs} = \frac{157.240.340}{38,07} . 10^{-3} = 336 \text{ mm}$$

$$\leq d/2 = 340/2 = 170 \text{ mm}$$

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

Jadi dipakai tulangan geser **P10 – 170 mm**

E. Perencanaan Tulangan Torsi

Tu = 1,39 KNm

$\sum x^2.y = 300^2 . 350 = 31,5.10^6 \text{ mm}^3$

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 :

$$Tu,b = \phi\left(\frac{1}{9}\sqrt{f'c}.\sum x^2.y\right) = 0,6.\left(\frac{1}{9}\sqrt{22,5}.31,5.10^6\right.$$

= 9,96 KNm > Tu = 1,39 KNm , tulangan torsi diabaikan.

4.4.2.2 Portal Arah – Y

Berikut ini contoh perhitungan balok BI portal – Y as 10 (bentang A - B) lantai 1

A. Tulangan Tumpuan

Momen tumpuan diambil yang paling besar dari semua kombinasi, dan momen yang digunakan adalah yang terbesar dari kedua tumpuan (kiri & kanan)

Dipakai dimensi rencana 350/550

fc' = 22,5 Mpa fy = 360 Mpa

- Tulangan tumpuan momen negatif

Mu = 195,34 Nm (-)

$$\frac{Mu}{\phi} = \frac{195,34}{0,8} = 244,2 \text{ KNm}$$

$$\rho b = \frac{0,85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5}{360}.0,85\left(\frac{600}{600+360}\right) = 0,0282$$

$$\rho_{maks} = 0,75 \;.\; \rho b = 0,75 \;.\; 0,0282 = 0,0212$$

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{360} = 0,0039$$

$$\rho_{pakai} = 0,5 \;.\; \rho_{maks} = 0,5 \;.\; 0,0212 = 0,0106$$

$$m = \frac{fy}{0,85.f'c} = \frac{360}{0,85.22,5} = 18.82$$

Rn $= \rho\ fy\ (1 - \frac{1}{2}\rho\ m) = 0{,}0106\ .\ 350\ (1 - \frac{1}{2}\ .\ 0{,}0106\ .\ 18{,}82) = 3{,}431$ MPa

$$d_{perlu} = \sqrt{\frac{Mu}{\phi} / Rn.b} = \sqrt{\frac{244{,}2\ .\ 10^6}{3{,}431\ .\ 350}} = 451 \text{ mm}$$

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

$= 550 - 100 = 450$ mm $> d_{perlu}$, maka dipakai tulangan rangkap

$As1 = \rho_1\ .\ b\ .\ d_{diketahui}$

$As1 = 0{,}0106.350.450 = 1670\ mm^2$

Dipakai tulangan Ø19 dengan A1Ø $= 284\ mm^2$

jumlah tulangan (n) $= \frac{As}{A1\phi} = \frac{1670}{284} = 5{,}8$ batang

dipakai **6D19**, maka $As_{ada} = 6\ .\ 284 = 1704\ mm^2 > As$

$$a = \frac{As1.fy}{0{,}85.fc'.b} = \frac{1670\ .\ 360}{0{,}85.22{,}5.350} = 89{,}81 \text{ mm}$$

$$Mn_1 = As_1 \cdot fy \cdot \left(d_{diketahui} - a/2\right) < Mu/\phi$$

$$Mn_1 = 1670.360.\ (\ 450 - 89{,}81/2\ ).10^{-6} = 243{,}54 \text{ KNm}$$

$$< Mu/\phi = 244{,}2 \text{ KNm}$$

$Mn_2 = Mu/\phi$ - Mn1

$Mn_2 = 244{,}2 - 243{,}54 = 0{,}66$ KNm

$$fs' = 600 \cdot \left\{1 - \frac{0{,}85 \cdot f'c \cdot \beta_1}{(\rho - \rho') \cdot fy} \cdot \frac{d'}{d_{diketahui}}\right\} \leq fy$$

$$fs' = 600 \cdot \left\{1 - \frac{0{,}85 \cdot 22{,}5 \cdot 0{,}85}{0.0106 \cdot 360} \cdot \frac{60}{450}\right\} = 259{,}19 \text{ Mpa} \leq 360 \text{ Mpa}$$

$$As' = \frac{Mn_2}{fs' \cdot (d_{diketahui} - d')}$$

$$As' = \frac{0{,}66.10^6}{259{,}19 \cdot (450 - 60)} = 6{,}53 \text{ mm}^2$$

Dipakai tulangan Ø19 dengan A1O = 284 mm^2

jumlah tulangan (n') = $\frac{As'}{A1\phi} = \frac{6{,}53}{284} = 0{,}02$ batang

dipakai **2D19**, maka $As_{ada} = 2 \,.\, 284 = 568 \text{ mm}^2 > As'$

As = As1 + As' = 1704 + 568 =2272 mm^2

Kontrol kapasitas lentur

$$\rho = \frac{As}{b \cdot d_{diketahui}}$$

$$\rho = \frac{2272}{300 \cdot 450} = 0.0168$$

$$\rho' = \frac{As'}{b \cdot d_{diketahui}}$$

$$\rho' = \frac{568}{300.450} = 0.0042$$

$$fs' = 600 \cdot \left\{ 1 - \frac{0{,}85 \cdot 22{,}5 \cdot 0{,}85}{(0.0168 - 0.0042) \cdot 360} \cdot \frac{100}{450} \right\} = 122{,}15 \text{ Mpa} \leq 360 \text{ Mpa}$$

$$a = \frac{As \cdot fy - As' \cdot fs'}{0{,}85 \cdot f'c \cdot b} = \frac{2272 \cdot 360 - 568 \cdot 122{,}15}{0.85 \cdot 22.5 \cdot 350} = 111{,}82 \text{ mm}$$

$$Mn = (As \cdot fy - As' \cdot fs') \cdot \left(d_{diketahui} - a/2\right) + (As' \cdot fs') \cdot (d_{diketahui} - d')$$

$$Mn = (2272 \cdot 360 - 568 \cdot 122{,}15) \cdot \left(450 - 111{,}82/2\right) + (568 \cdot 122{,}15) \cdot (450 - 100)$$

Mn = 319,27 KNm > Mu/ϕ = 244,2 KNm -OK-

Jarak bersih antar tulangan = $\frac{b - 2.Pb - 2.\phi\ sengkang - n.\phi\ tul.}{(n-1)}$

$= \frac{350 - 2.50 - 2.10 - 6.19}{(6-1)}$

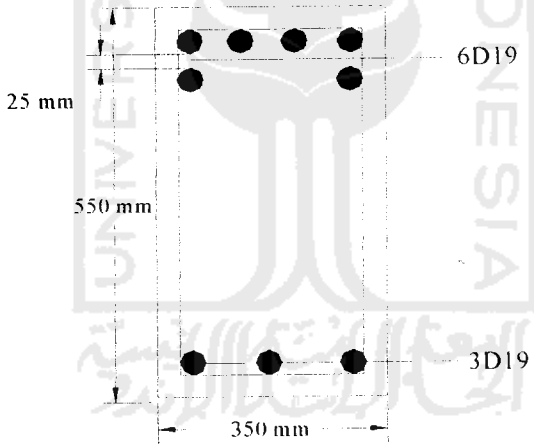
= 23,2 mm < 25 mm, maka dipakai tulangan 2 lapis

- Tulangan tumpuan momen positif

  jumlah tulangan tumpuan momen positif = 50% . jmlh tul tumpuan

  = 50% . 6 tulangan D19 = 3 tulangan D19

  Maka digunakan tulangan **3D19**



Gambar 4.17 Tulangan Tumpuan Balok Induk

B. Tulangan Lapangan

Mu = 152,6 KNm

$\frac{Mu}{\phi} = \frac{152,6}{0,8} = 190,7$ KNm

$$\rho b = \frac{0{,}85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0{,}85.22{,}5}{360}.0.85\left(\frac{600}{600+360}\right) = 0{,}0282$$

$$\rho_{maks} = 0{,}75\ .\ \rho b = 0{,}75\ .\ 0{,}0282 = 0{,}0212$$

$$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{360} = 0{,}0039$$

$$\rho_{pakai} = 0{,}5\ .\ \rho_{maks} = 0{,}5\ .\ 0{,}0212 = 0{,}0106$$

$$m = \frac{fy}{0{,}85.f'c} = \frac{360}{0{,}85.22{,}5} = 18.82$$

$$Rn = \rho\ fy\ (1 - \tfrac{1}{2}\rho\ m) = 0{,}0106\ .\ 350\ (1 - \tfrac{1}{2}\ .\ 0{,}0106\ .\ 18{,}82) = 3{,}431\ MPa$$

$$d_{perlu} = \sqrt{\frac{Mu}{\phi}} = \sqrt{\frac{190{,}7\ .\ 10^6}{3{,}431\ .\ 350}} = 399\ mm$$

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

= 550 – 100 = 450 mm > d_{perlu} maka dipakai tulangan sebelah.

$$Rn_{ada} = \frac{Mu}{\phi} = \frac{190{,}7.10^6}{350.450^2} = 2{,}69\ Mpa$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}.\rho_{pakai} = \frac{2{,}69}{3{,}431}.0{,}0106 = 0{,}0083 > \rho_{min} = 0{,}0039$$

ρ_{pakai} = 0,0083

$As_{perlu} = \rho_{ada}.b.d = 0{,}0083.350.450 = 1307{,}25\ mm^2$

Dipakai tulangan Ø19 dengan $A1Ø = 284\ mm^2$

$$\text{jumlah tulangan (n)} = \frac{As_{perlu}}{A1\phi} = \frac{1307{,}25}{284} = 4{,}6\ \text{batang}$$

dipakai **5D19**, maka $As_{ada} = 5\ .\ 284 = 1420\ mm^2 > As_{perlu}$

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

$$= \frac{350 - 2.50 - 2.10 - 5.19}{(5-1)}$$

= 33,75 mm > 25 mm, maka dipakai tulangan 1 lapis

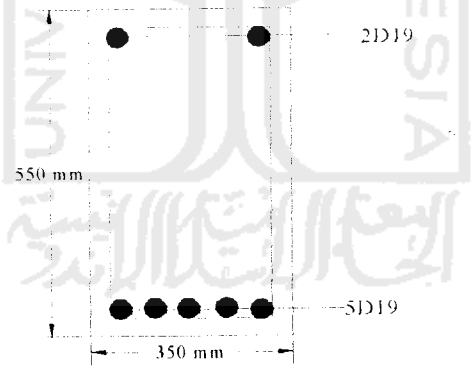
Kontrol kapasitas momen nominal :

$$a = \frac{As_{ada}.fy}{0{,}85.fc'.b} = \frac{1420\ .\ 360}{0{,}85.22{,}5.350} = 76{,}37\ mm$$

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

$$= 1420\ .\ 360\ .\ (\ 450 - 76{,}37/2\ ).10^{-6}$$

$$= 210{,}52\ KNm > Mu/\phi = 190{,}7\ KNm \rightarrow OK!$$



Gambar 4.18 Tulangan Lapangan Balok Induk

C. Momen Nominal Aktual Balok

1) Momen Aktual Balok Negatif Tumpuan kiri = kanan

tulangan atas = 6D19 dengan $As_{ada} = 1704\ mm^2$

tulangan bawah = 3D19 dengan $As_{ada} = 851\ mm^2$

$$\rho = \frac{As_{ada}}{b.d_{pakai}} = \frac{1704}{350.450} = 0{,}0108$$

$$\rho' = \frac{As'_{ada}}{b.d_{pakai}} = \frac{851}{350.450} = 0{,}0054$$

$$\rho_1 = \rho - \rho' = 0{,}0108 - 0{,}0054 = 0{,}0054$$

$$fs' = 600\left\{1 - \frac{0{,}85.f'c.\beta_1}{(\rho - \rho').fy}\frac{d'}{d}\right\} = 600\left\{1 - \frac{0{,}85.22{,}5.0{,}85}{0{,}0054.360}\frac{100}{450}\right\}$$

$= 514{,}9$ Mpa

fs' > fy dipakai fs' = 360 Mpa

$$a = \frac{(As_{ada}.fy) - (As'_{ada}.fs')}{0{,}85.fc'.b} = \frac{(1704.360) - (851.360)}{0{,}85.22{,}5.350}$$

$= 45{,}87\ mm^2$

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

$$= (1704.360 - 851.360).(450 - 45.87/2).\ 10^{-6} = 131{,}14\ KNm$$

$$Mn_2 = (As'_{ada}.fs').\ (d - d') = (851.360).(450 - 100)\ .\ 10^{-6} = 107{,}23\ KNm$$

$$Mnak^- = Mn1 + Mn_2 = 131{,}14 + 107{,}23 = 238{,}37\ KNm$$

2) Momen Aktual Balok Positif

$$\rho_{aktual} = \frac{As_{ada}}{b.d_{pakai}} = \frac{1420}{350.450} = 0{,}009$$

$$Rn = \rho.fy(1 - 1/2.\rho..m) = 0{,}009.360.(1\text{-}1/2.\ 0{,}009.18{,}82) = 2{,}96\ Mpa$$

$$Mnak^+ = Rn.b.d^2 = 2{,}96.350.450^2.10^{-6} = 210{,}18\ KNm$$

D. 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 $Vu,b = 1,05\ (\ V_{D,b} + V_{L,b} + 4/k\ .V_{E,b}\ )$

$V_D = 89,31$ KN ; $V_L = 26,79$ KN ; $V_E = 15,12$ KN

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

$$Vu,b = 0,7.1,25\left[\frac{238,37 + 210,18}{6}\right] + 1,05(89,31 + 26,79) = 187,32\ KN$$

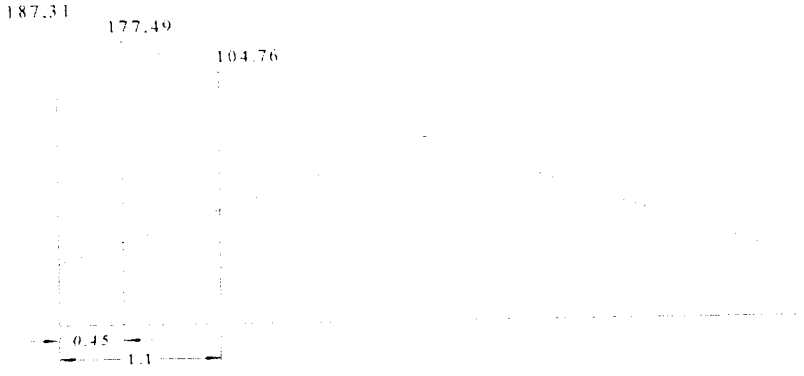
Dengan syarat tidak lebih besar dari :

$$Vu,b = 1,05\ (89,31 + 26,79 + 4/1\ .\ 15,12\ ) = 187,31\ KN$$

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

$$= \left[121,90 - 0,7.1,25\left(\frac{238,37 + 210,18}{6}\right)\right] + \frac{6 - 0,45}{6}\left[187,31 - \left[121,90 - 0,7.1,25\left[\frac{238,37 + 210,18}{6}\right]\right]\right]$$

$$= 177,49\ KN$$



Gambar 4.19 Diagram Tegangan Geser Balok

1) dalam daerah sendi plastis

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

Vu,b = 177,49 KN

Vc = 0

$$\frac{Vu,b}{\phi} = \frac{177,49}{0,6} = 295,81 \text{ KN}$$

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

Jarak sengkang :

$$s \leq \frac{Av.fy.d}{\frac{Vu,b}{\phi} - Vc} = \frac{157.240.450}{295,81 - 0} \cdot 10^{-3} = 57,3 \text{ mm}$$

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

Jadi dipakai tulangan geser **P10 – 50 mm**

2) Diluar sendi plastis

Diambil jarak sejauh 2h = 2.550 = 1100 mm

$$Vu,b = 177,49 - \frac{177,49.(1,1-0,45)}{(3-0,45)} = 132,25 \text{ KN}$$

Vu,b = 132,25 KN

$$Vc = 1/6.\sqrt{fc'}.b.d = 1/6.\sqrt{22,5}.350.450.10^{-3} = 124,51 \text{ KN}$$

$$Vs = \frac{Vu,b}{\phi} - Vc = \frac{132,25}{0,6} - 124,51 = 95,91 \text{ KN}$$

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

Jarak sengkang :

$$s \leq \frac{Av.fy.d}{Vs} = \frac{157.240.450}{95,91} . 10^{-3} = 176,8 \text{ mm}$$

$$\leq d/2 = 450/2 = 225 \text{ mm}$$

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

Jadi dipakai tulangan geser **P10 – 170 mm**

E. Perencanaan Tulangan Torsi

Tu = 3,86 KNm

$$\sum x^2.y = 350^2 . 600 = 77,76.10^6 \text{ mm}^3$$

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 :

$$Tu,b = \phi\left(\frac{1}{9}\sqrt{f'c}.\sum x^2.y\right) = 0{,}6.\left(\frac{1}{9}\sqrt{22{,}5}.77{,}76.10^6\right)$$

$= 24{,}58$ KNm > Tu = 3,86 KNm , tulangan torsi diabaikan.

4.5 Perencanaan Kolom

4.5.1 Perhitungan Momen dan Gaya Aksial Rencana

Contoh perhitungan pada kolom As – 8 portal A lantai 1

1. Momen untuk portal arah X

$M_{Dy\ atas}$ = -2,23 KNm

$M_{Dy\ bawah}$ = 1,36 KNm

$M_{Ly\ atas}$ = -0,48 KNm

$M_{Ly\ bawah}$ = 0,24 KNm

$M_{Ey\ atas}$ = 30,56 KNm

$M_{Ey\ bawah}$ = 40,32 KNm

Kombinasi pembebanan :

Atas

$1,2\ M_{Dy} + 1,6\ M_{Ly}$ = 1,2 .2,23 + 1,6.0,48 = 3,4 KNm

$1,05\ (M_{Dy} + M_{Ly} + M_{Ey})$ = 1,05 (2,23 + 0,48 + 30,56) = 35 KNm

Bawah

$1,2\ M_{Dy} + 1,6\ M_{Ly}$ = 1,2 . 1,36 + 1,6. 0,24 = 0,016 KNm

$1,05\ (M_{Dy} + M_{Ly} + M_{Ey})$ = 1,05 (1,36 + 0,24 + 40,32) = 44 KNm

2. Momen untuk portal arah Y

Data Momen

$M_{Dx\ atas}$ = 30,84 KNm

$M_{Dx\ bawah}$ = -15,61 KNm

$M_{Lx\ atas}$ = 9,32 KNm

$M_{Lx\ bawah}$ = -3,92 KNm

$M_{Ex\ atas}$ = -44,42KNm

$M_{Ex\ bawah}$ = 55,79 KNm

Kombinasi pembebanan :

Atas

$1,2\ M_{Dx} + 1,6\ M_{Lx}$ = 1,2 .30,84 + 1,6.9,32 = 52 KNm

$1,05\ (M_{Dx} + M_{Lx} + M_{Ex})$ = 1,05 (30,84 + 9,32 + 44,42) = 84,6 KNm

Bawah

$1,2\ M_{Dx} + 1,6\ M_{Lx}$ = 1,2 . 15,61 + 1,6. 3,92 = 25 KNm

$1,05\ (M_{Dx} + M_{Lx} + M_{Ex})$ = 1,05 (15,61 + 3,92 + 55,79) = 75,32 KNm

3. Gaya Aksial

Data Gaya Aksial

$P_{D\ atas}$ = -627,41 KNm

$P_{D\ bawah}$ = -410,57KNm

$P_{L\ atas}$ = -111,72 KNm

$P_{L\ bawah}$ = -111,72 KNm

$P_{Ex\ atas}$ = -7,91 KNm

$P_{Ex\ bawah}$ = -7,91 KNm

$P_{Ey\ atas}$ = 35,28 KNm

$P_{Ey\ bawah}$ = 35,28 KNm

Kombinasi pembebanan :

Atas

$1,2\ P_D + 1,6\ P_L$ = 1,2 .627,41 + 1,6.111,72 = 932 KNm

$1,05\ (P_D + P_L + P_E)$ = 1,05 (627,41 + 111,72 + 35,28) = 813 KNm

Bawah

$1,2\ P_D + 1,6\ P_L$ $= 1,2\ .410,57 + 1,6.111,72 = 671$ KNm

$1,05\ (P_D + P_L + P_E)$ $= 1,05\ (410,57 + 111,72 + 35,28) = 585$ KNm

4.5.2 Kriteria Kolom dan Pembesaran Kolom

Menghitung Kekakuan Kolom

1. Arah X

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

$= 4700\ \sqrt{22,5}$

$= 22294$ Mpa

rencana dimensi kolom = 400x400 mm

Ic (Inersia kolom) $= \frac{1}{12}.400^3.400 = 2,13.10^9\ mm^4$

$\beta d = \frac{1,2M_D}{1,2M_D + 1,6M_L} = \frac{1,2.2,23}{1,2.2,23 + 1,6.0,48} = 0,80$

$EI = \frac{E_c.I_c}{2,5(1+\beta.d)} = \frac{22294.x2,13.10^9}{2,5(1+0,8)} = 1,05.\ 10^{13}\ Nmm^2$

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 :

$I_{cr} = \frac{1}{2}\left(\frac{1}{12}.b.h^3\right) = \frac{1}{2}\left(\frac{1}{12}.300.400^3\right) = 0,8.10^9\ mm^4$

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

   Lc (panjang bersih kolom) = 3,4 m

   Lg (panjang bersih balok) = 4,6 m

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

$$\psi_{atas} = \frac{\left(\frac{1{,}05.10^{13}}{3400} + \frac{1{,}05.10^{13}}{3400}\right)}{\left(\frac{22294.0{,}8.10^{9}}{4600} + \frac{22294.0{,}8.10^{9}}{4600}\right)} = 0{,}79$$

ψ_{bawah} = 0 (ujung jepit)

Dari Nomogram portal tanpa pengaku, didapat k = 1,2

$$\frac{k.lu}{r} = \frac{1{,}2.3400}{0{,}3.400} = 34 > 22 \text{ (termasuk kolom langsing)}$$

Beban tekuk Euler yang terjadi adalah

$$Pc = \frac{\pi.^2 EI}{(k.Lc)^2} = \frac{\pi.^2 1{,}05.10^{13}}{(1{,}2.3400)^2} = 6225421 \text{ N}$$

Menghitung faktor pembesaran momen δ_{by}

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

Cm = 1 (untuk portal tanpa pengaku)

$$\delta_{by} = \frac{1}{1-\left(\frac{932000}{0{,}65.6225421}\right)} \geq 1$$

$$= 1{,}3 \geq 1$$

Menghitung faktor pembesaran momen δ_{sy}

Σ Pu = (611000+932000+734626+737134+996396+733459)

= 4744615 N

Σ Pc = (6225421 x 6)

= 37352526 N

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

$$= \frac{1}{1-\left(\frac{4744615}{37352526}\right)} \geq 1$$

= 1,14 $\geq$ 1

Momen akibat pembesaran momen

Muy, bawah = δ_{by} Mby + δ_{sy} Msy

= (1,3 . 0,016) + (1,14 . 44)

= 50,2 KNm

Muy, atas = δ_{by} Mby + δ_{sy} Msy

= (1,3 . 3,4) + (1,14 . 35)

= 44,3 KNm

2. Arah Y

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

= $4700\sqrt{22,5}$

= 22294 Mpa

rencana dimensi kolom = 400x400 mm

$$\frac{k.lu}{r} = \frac{1,05.3400}{0,3.400} = 29,75 \quad < \quad 22 \text{ (termasuk kolom langsing)}$$

Beban tekuk Euler yang terjadi adalah

$$Pc = \frac{\pi.^2 EI}{(k.Lc)^2} = \frac{\pi.^2 1,108.10^{13}}{(1,05.3400)^2} = 8580312 \text{ N}$$

Menghitung faktor pembesaran momen δ_{by}

$$\delta_{by} = \frac{1}{1-\left(\frac{932000}{0,65.8580312}\right)} \geq 1$$

$$= 1,2 \geq 1$$

Menghitung faktor pembesaran momen δ_{sy}

Σ Pu = (932000+1466812+1432058+1421661+1412827+1448501 +992966)

= 9106855 N

Σ Pc = (6225421 x 7) = 43577947 N

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

$$= \frac{1}{1-\left(\frac{9106855}{43577947}\right)} \geq 1$$

$$= 1,26 \geq 1$$

Momen akibat pembesaran momen

Muy, bawah $= \delta_{by}$ Mby + δ_{sy} Msy

$= (1,2 . 25) + (1,26 . 75,32)$

= 125 KNm

Muy, atas $= \delta_{by}$ Mby + δ_{sy} Msy

= (1,2 . 52) + (1,26 . 84,6)

= 169 KNm

4.5.3 Analisis Gaya Aksial dan Momen akibat balok

Contoh perhitungan pada kolom As – 8 portal A lantai 1

h = 4 m

hn = 3,4 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 = 1,25.180,5 = 225,625 KNm

$Mkap_{(kanan)}$= 1,25. Mnak = 1,25. 180,5 = 225,625 KNm

menghitung gaya aksial rencana :

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

$$= 0,7.\ 1.\ \frac{225,625}{4} + \frac{225,625}{4} + 1,05(627,41 + 111,72)$$

= 621,18 KN

tidak perlu melebihi :

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

= 1,05 (627,41 + 111,72 + 4.35,28)

= 924,26 KN

menghitung α :

$M_{Ey\ atas}$ = 30,56 KNm

$M_{Ey\ bawah}$ = 40,32 KNm

$$\alpha\ ka = \frac{M_{E.k(lti+latas)}}{M_{E.k(lti+latas)} + M_{E.k(ltibawah)}} = \frac{30,56}{30,56 + 40,32} = 0,43$$

$$\alpha\ kb = \frac{M_{E.k(ltibawah)}}{M_{E.k(lti+latas)} + M_{E.k(ltibawah)}} = \frac{40,32}{30,56 + 40,32} = 0,57$$

menghitung momen rancang kolom :

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

$$= \frac{3,4}{4}.1,0\ .\ 0,43.0,7.\left(\frac{5}{4,6}.225,625 + \frac{5}{4,6}.225,625\right)$$

= 153,43 KNm

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

$$= \frac{3,4}{4}.1,0.0,57\ .\ 0,7.\left(\frac{5}{4,6}.225,625 + \frac{5}{4,6}.225,625\right)$$

= 191,30 kNm

tidak perlu melebihi :

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

$$= 1,05\ (2,23 + 0,48 + \frac{4}{1}(40,32))$$

= 172,2 KNm

b. .Perhitungan Arah Y

$Mkap_{(kiri)}$ = 1,25. Mnak = 1,25.0 = 0 KNm

$Mkap_{(kanan)}$ = 1,25. Mnak = 1,25.237,85 = 297,31 KNm

menghitung gaya aksial rencana :

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

$$= 0,7.\ 1.\ \frac{0}{0} + \frac{297,31}{4} + 1,05(627,41 + 111,72)$$

= 850,41 KN

tidak perlu melebihi :

Pu,k_x = 1,05 (N_D+ N_L+ 4.N_E)

= 1,05 (627,41 + 111,72 + 4.35,28)

= 924,26 KN

menghitung α :

$M_{Ex\ atas}$ = -44,42KNm

$M_{Ex\ bawah}$ = 55,79 KNm

$$\alpha\ ka = \frac{M_{E.k(lti+latas)}}{M_{E.k(lti+latas)} + M_{E.k(ltibawah)}} = \frac{44,42}{44,42 + 55,79} = 0.44$$

$$\alpha\ kb = \frac{M_{E.k(ltibawah)}}{M_{E.k(lti+latas)} + M_{E.k(ltibawah)}} = \frac{55,79}{44,42 + 55,79} = 0.56$$

menghitung momen rancang kolom :

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

$$= \frac{3,4}{4}.1,0.0,44\ .\ 0,7.\left(\frac{5}{4,6}.0+\frac{5}{4,6}.297,31\right)$$

$$= 84,60\ \text{KNm}$$

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

$$= \frac{3,4}{4}.1,0.0,56\ .\ 0,7.\left(\frac{5}{4,6}.0+\frac{5}{4,6}.297,31\right)$$

$$= 107,67\ \text{KNm}$$

tidak perlu melebihi :

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

$$= 1,05\ (30,84+9,32\ +\frac{4}{1}(55,79))$$

$$= 263\ \text{KNm}$$

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 :

$\text{Pu,k}_x = 593,05\ \text{KN}$

$\text{Pu,k}_y = 621,18\ \text{KN}$

$\text{Mu,k}_x = 1,2\ .\ 107,67 = 129,20\ \text{KNm}$

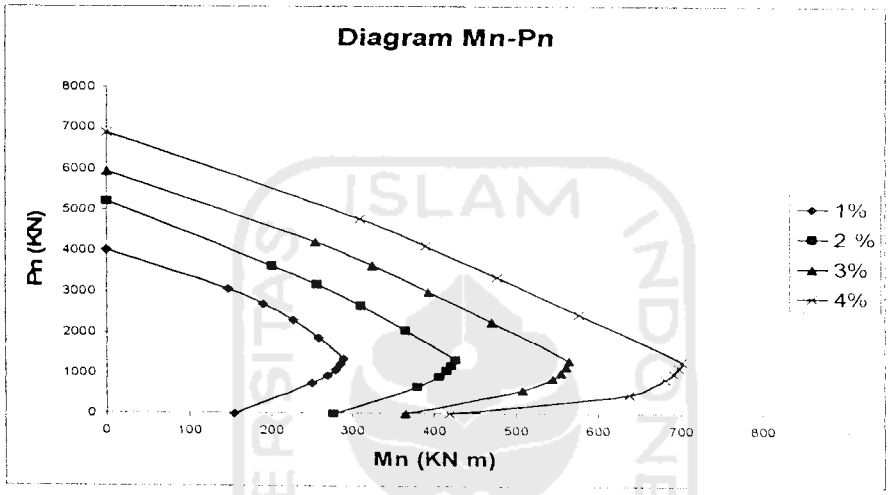
$\text{Mu,k}_y = 1,3\ .\ 191,30 = 248,7\ \text{KNm}$

$$\frac{Pu,k_x}{\phi} = \frac{593,05}{0,65} = 912,4\ \text{KN}$$

$$\frac{Pu, k_y}{\phi} = \frac{621,18}{0,65} = 955,7 \text{ KN}$$

$$\frac{Mu, k_x}{\phi} = \frac{129,20}{0,65} = 198,77 \text{ KNm}$$

$$\frac{Mu, k_y}{\phi} = \frac{248,7}{0,65} = 382,6 \text{ KNm}$$



Gambar 4.20 Grafik Mn – Pn Kolom

a. Arah x

$$\frac{Pu, k_y}{\phi} = \frac{621,18}{0,65} = 955,7 \text{ KN}$$

$$\frac{Mu, k_y}{\phi} = \frac{248,7}{0,65} = 382,6 \text{ KNm}$$

Dari grafik Mn vs Pn didapat $\rho_g = 2\ \%$

$Ast = 0,02.400.400 = 3200\ mm^2$

$As = As' = 0,5.Ast = 1600\ mm^2$

dipakai 5D22 dengan $As_{ada} = As'_{ada} = 1901\ mm^2$

Cek eksentrisitas balance (e_b)

$$Xb = \frac{600.d}{600 + fy} = \frac{600.340}{600 + 360} = 212{,}5 \text{ mm}$$

$ab = \beta_1 . Xb = 0{,}85 . 212{,}5 = 180{,}625$ mm

$$f's = 600\frac{(Xb - d')}{Xb} = 600\frac{(180 - 60)}{180} = 400 \text{ MPa} > fy = 360 \text{ MPa}$$

Dengan demikian digunakan $f's = fy = 360$ MPa

$Ccb = 0{,}85.f'c.b.ab = 0{,}85.22{,}5.360.180{,}625 = 1243603$ N

$Csb = As'(fs' - 0{,}85.f'c) = 1901.(360 - 0{,}85.22{,}5) = 648003$ N

$Tsb = As.fy = 1901 .360 = 684360$ N

$Pnb = Ccb + Csb - Tsb = 1243603 + 648003 - 684360$

$= 1207246$ N $= 1207$ KN

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

$$= 1243603\left[\frac{400}{2} - \frac{180{,}625}{2}\right] + 648003\left(\frac{400}{2} - 60\right)$$

$$+ 684360\left(340 - \frac{400}{2}\right)$$

$= 322938524$ Nmm $= 332{,}93$ KNm

$$eb = \frac{Mnb}{Pnb} = \frac{332{,}93}{1207} = 0{,}27 \text{ m}$$

$$e = \frac{Mu_k, y / \phi}{Pu_k / \phi} = \frac{382{,}6}{955{,}7} = 0{,}4 \text{ m}$$

karena $e > e_b$ ⟶ kolom mengalami patah tarik

Kontrol tegangan pada daerah desak :

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

$$= \frac{1901.360}{\frac{400}{(340-60)} + 0{,}5} + \frac{400.400.22{,}5}{\frac{3.400.400}{340^2} + 1{,}18}$$

$$= 1029990 \text{ N} = 1029{,}9 \text{ KN}$$

$$Pn = 1029{,}9 \text{ KN} > \frac{Pu_k}{\phi} = 955{,}7 \text{ KN} \text{ ..... } \mathbf{Ok\ !}$$

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

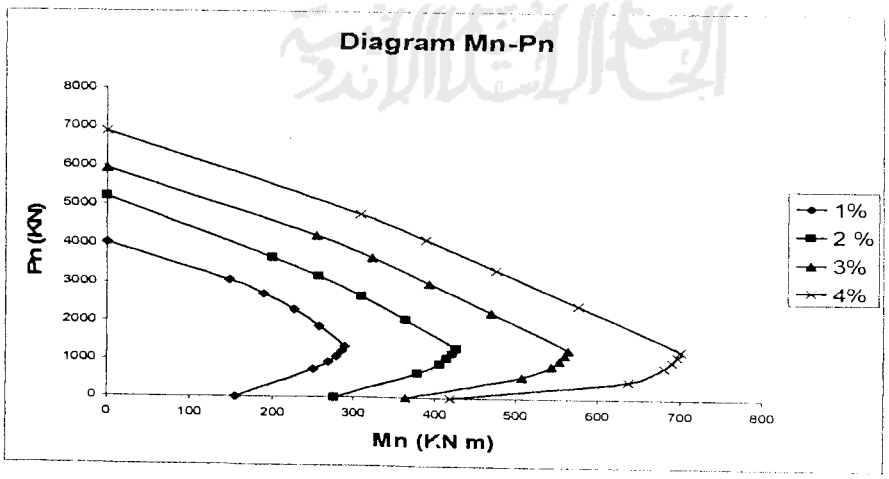
$$= 1029{,}9.\ 400$$

$$= 411{,}9 \text{ KNm} > \frac{Mu, k_y}{\phi} = 382{,}6 \text{ KNm}$$

b. Arah y

$$\frac{Pu, kx}{\phi} = \frac{593{,}05}{0{,}65} = 912{,}4 \text{ KN}$$

$$\frac{Mu, k_x}{\phi} = \frac{129{,}20}{0{,}65} = 198{,}77 \text{ KNm}$$



Gambar 4.21 Grafik Mn – Pn Kolom

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

$Ast = 0{,}01.400.400 = 1600\ mm^2$

$As = As' = 0{,}5.Ast = 800\ mm^2$

dipakai 3D22 dengan $As_{ada} = As'_{ada} = 1140\ mm^2$

Cek eksentrisitas balance (e_b)

$$cb = \frac{600.d}{600 + fy} = \frac{600.340}{600 + 360} = 215{,}5\ mm$$

$ab = \beta_1 . cb = 0{,}85 . 212{,}5 = 180{,}7\ mm$

$$f'sb = 600\frac{(cb - d')}{cb} = 600\frac{(180{,}7 - 60)}{180{,}7} = 401\ MPa > fy = 360\ MPa$$

digunakan $f'sb = fy = 360\ MPa$

$Ccb = 0{,}85.f'c.b.ab = 0{,}85.22{,}5.360.180{,}7 = 1244119\ N$

$Csb = As'(fs' - 0{,}85.f'c) = 1140.(360 - 0{,}85.22{,}5) = 388597\ N$

$Tsb = As.fy = 1140.360 = 410400\ N$

$Pnb = Ccb + Csb - Tsb = 1244119 + 388597 - 410400$

$= 1222316 = 1222{,}3\ KN$

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

$$= 1244119.\left[\frac{400}{2} - \frac{180{,}7}{2}\right] + 388597.\left(\frac{400}{2} - 60\right) + 410400\left(340 - \frac{400}{2}\right)$$

$= 248277228\ Nmm = 248{,}27\ KNm$

$$eb = \frac{Mnb}{Pnb} = \frac{248{,}27}{1222{,}3} = 0{,}203$$

$$e = \frac{Mu_{,kx}/\phi}{Pu_{,k}/\phi} = \frac{198.77}{912{,}4} = 0{,}22$$

karena $e > e_b$,kolom mengalami patah tarik

Kontrol kapasitas kolom terhadap patah tarik :

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

$$= \frac{1140.360}{\frac{220}{(340-60)} + 0{,}5} + \frac{400.400.22{,}5}{\frac{3.400.220}{340^2} + 1{,}18}$$

$= 1346640{,}5\ N = 1346{,}64\ KN$

$$Pn = 1346{,}64\ KN > \frac{Pu_k}{\phi} = 912{,}4\ KN \text{ ..... } \mathbf{Ok!}$$

$Mn = Pn \cdot e$

$= 1346{,}64 \cdot 0{,}223$

$$= 300{,}3\ KNm > \frac{Mu, k_x}{\phi} = 198{,}77\ KN$$

4.5.5 Perencanaan Tulangan Geser Kolom

$Mu_{,k}$ atas = 153,43 KNm

$Mu_{,k}$ bwh = 191,30 kNm

$V_{D,}k = 11{,}61\ KN$

$V_{L,}k = 3{,}31 KN$

$V_{Ex,}k = 25{,}03\ KN$

$V_{Ey,}k = 17{,}72\ KN$

$h_n = 3{,}60\ m$

$$V_{u,k} = \frac{Mu, k_{atas} + Mu, k_{bawah}}{h_n} = \frac{153{,}43 + 191{,}30}{3{,}60} = 96\ KN$$

tetapi tidak perlu lebih besar dari :

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

$$= 1{,}05\ (\ 11{,}61 + 3{,}31 + \frac{4}{1}.(25{,}03 + 0{,}3.17{,}72)$$

$$= 143{,}12\ \text{KN}$$

$$\frac{V_{U,k}}{\phi} = \frac{96}{0{,}6} = 160\ \text{KN}$$

di daerah sejauh *lo*

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

$$Vs = \frac{V_{U,k}}{\phi} = 160\ \text{KN}$$

Dipakai tulangan geser Ø P10 mm, maka :

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

$$\text{Jarak (s)} < \frac{Av.fy.d}{Vs} = \frac{157.240.340}{160.10^3} = 80{,}07\ \text{mm}$$

$$< d/4 = 85\ \text{mm}$$

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

Digunakan sengkang $\mathbf{P_{10\text{-}80}}$ **mm**

di luar daerah *lo*

$$Vc = \left(1 + \frac{P_U.k}{14.A_g}\right).\frac{1}{6}\sqrt{fc'}.b.d = \left(1 + \frac{621{,}18.10^3}{14.400.400}\right).\frac{1}{6}\sqrt{22{,}5}.400.340$$

$$= 137{,}4\ \text{KN} < \frac{V_{U,k}}{\phi} = 160\ \text{KN}, \text{ maka perlu tulangan geser.}$$

$$Vs = \frac{V_{U,k}}{\phi} - Vc = 160 - 137{,}4 = 22{,}6\ \text{KN}$$

Dipakai tulangan geser Ø P10 mm, maka :

Av = 2 . ¼ .π . 10^2 = 157 mm^2

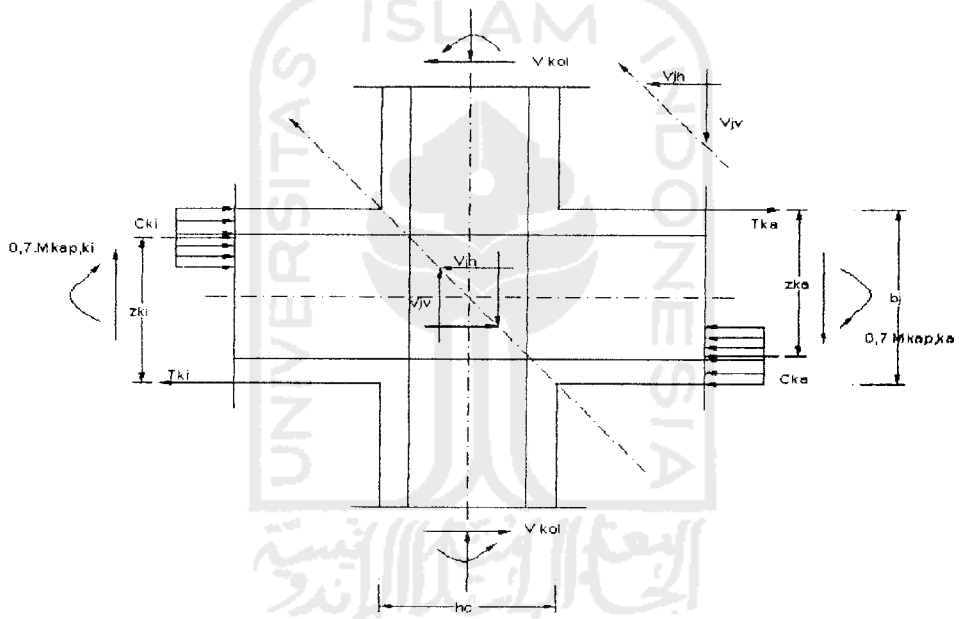
Jarak (s) $< \frac{Av.fy.d}{Vs} = \frac{157.240.340}{22,6.10^3} = 566$ mm

< d/2 = 170 mm

< 16.D = 160 mm

Digunakan sengkang **$P_{10\text{-}170}$ mm**

4.5.6 Pertemuan Balok Kolom



Gambar 4.22 Joint Balok Kolom Dalam

a. Perhitungan gaya-gaya dalam

1). Sumbu X

b_j = bc = 400 mm

= bb + 0,5.hc = 350 + 0,5.400 = 550 mm

b_j pakai = 400 mm

h_c = 400 mm

$$V_{kol,x} = \frac{0,7 \ . \left(\frac{lki}{lki'} . M_{kap,ki} + \frac{lka}{lka'} M_{kap,ka} \right)}{\frac{1}{2} . (h_{k,a} + h_{k,b})}$$

$$V_{kol,x} = \frac{0,7 \ . \left(\frac{5}{4,6} . 225,625 + \frac{5}{4,6} . 225,625 \right)}{\frac{1}{2} . (3,5 + 3,5)} = 99 \text{ KN}$$

$z_{ki,x}$ = 0,9.d = 0,9.340 = 306 mm = 0,306 m

$z_{ka,x}$ = 0,9.d = 0,9.340 = 306 mm = 0,306 m

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

= 0,7.(225,625)/0,306 = 516,1 KN

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

= 0,7. (225,625)/0,306 = 516,1 KN

$V_{jh,x} = C_{ki,x} + T_{ka,x} - V_{kol,x}$ = 516.1 + 516,1 – 99

= 933,3 KN

Kontrol tegangan geser horizontal :

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

$$v_{jh,x} = \frac{933,3}{0,4.0,4} = 5833 \text{ KN/m}^2$$

= 5,8 N/mm^2 < 1,5. $\sqrt{22,5}$ = 7,115 N/mm^2 .....**Ok !**

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

$$V_{ch,x} = 2/3 . \sqrt{\left\{ \left(621,18.10^3 / 400.400 \right) - 0,1.22,5 \right\}} .400.400$$

= 136282 N = 136,3 KN

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

$= 933,3 - 136,3 = 797$ KN

2). Arah Y

$b_j = bc = 400$ mm

$= bb + 0,5.hc = 350 + 0,5.400 = 550$ mm

b_j pakai = 400 mm

$h_c = 400$ mm

$$V_{kol,x} = \frac{0,7 \ . \left(\frac{lki}{lki'} . M_{kap,ki} + \frac{lka}{lka'} M_{kap,ka} \right)}{\frac{1}{2} . (h_{k,a} + h_{k,b})}$$

$$V_{kol,x} = \frac{0,7 \ . \left(\frac{5}{4,6} . 0 + \frac{5}{4,6} . 237,85 \right)}{\frac{1}{2} . (3,5 + 3,5)} = 51,70 \text{ KN}$$

$z_{ki,y} = 0,9.d = 0,9.340 = 306$ mm $= 0,306$ m

$z_{ka,y} = 0,9.d = 0,9.340 = 306$ mm $= 0,306$ m

$C_{ka,y} = T_{ka,y} = 0,7.M_{kap,by\text{-}ka})/z_{ka,y}$

$= 0,7.(237,85)/0,306 = 544,1$ KN

$V_{jh,y} = C_{ki,y} + T_{ka,y} - V_{kol,y}$

$= 0 + 650,8 - 51,70 = 599,1$ KN

Kontrol tegangan geser horizontal :

$$v_{jh,y} = \frac{V_{jh,y}}{b_j.h_c} \leq 1,5\sqrt{f'c}$$

$$v_{jh,y} = \frac{599,1}{0,4.0,4} = 3744,37 \text{ KN/m}^2$$

$= 3,744$ N/mm^2 $< 1,5.\sqrt{22,5} = 7,115$ N/mm^2 ..... **Ok !**

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

$$V_{ch,y} = 2/3. \sqrt{\left\{\left(593,05.10^3/400.400\right) - 0,1.22,5\right\}} .400.400$$

$$= 128734 \text{ N} = 128,7 \text{ KN}$$

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

$$= 599,1 - 128,7 = 470,4 \text{ KN}$$

b. Penulangan Geser Horisontal

$$V_{sh,mak} = V_{sh,x} = 797 \text{ KN}$$

$$A_{jh} = \frac{V_{sh,mak}}{fy} = \frac{797.10^3}{360} = 2213,88 \text{ mm}^2$$

Digunakan sengkang rangkap Ø P10 dengan Av = 314 mm^2

Jumlah lapis sengkang = $\frac{2213,88}{314}$ = 7,05lapis

digunakan sengkang rangkap Ø 8P10

c. Penulangan geser vertikal

$$V_{cv} = \frac{A_{sc'}}{A_{sc}} V_{jh,mak} \left(0,6 + Pu,k/Ag.f'c\right)$$

$$V_{cv} = 1 \cdot 933,3 .10^3 \left(0,6 + 621,18.10^3/400.400.22,5\right)$$

$$= 721021 \text{ N} = 721 \text{ KN}$$

$$V_{jv} = bj/h_c.V_{jh,mak}$$

$$= (0,4/0,4). 933,3 = 933,3 \text{ KN}$$

$$V_{sv} = V_{jv} - V_{cv} = 933,3 - 721 = 212,3 \text{ KN}$$

$$A_{jv} = \frac{V_{sv}}{fy} = \frac{212{,}3.10^3}{360} = 590 \text{ mm}^2$$

Digunakan sengkang Ø P10 dengan Av = 157 mm^2

Jumlah lapis sengkang = $\frac{590}{157}$ = 3,75 lapis

digunakan sengkang **6P10**

4.5.7 Perencanaan Tulangan Lentur Kolom dengan Biaksial Momen

Perencanaan kolom biaksial momen ini hanya dijadikan cek, sedangkan perencanaan sebenarnya pada kolom adalah dengan menghitung arah X dan arah Y sebagaimana telah dijelaskan dimuka. Adapun perhitungan kolom dengan cara biaksial momen adalah sebagai berikut :

Pu,k_y = 621,18 KN

Mu,k_x = 1,23 . 107,67 = 132,43 KNm

Mu,k_y = 1,11 . 191,30 = 210,43 KNm

$$Pn = \frac{Pu}{\phi} = \frac{621{,}18}{0{,}7} = 887{,}4 \text{ KN}$$

$$Mnx = \frac{Mux}{\phi} = \frac{132{,}43}{0{,}7} = 189{,}18 \text{ KNm}$$

$$Mny = \frac{Muy}{\phi} = \frac{210{,}43}{0{,}7} = 300{,}61 \text{ KNm}$$

$$\frac{Mny}{Mnx} = \frac{300{,}61}{189{,}18} = 1{,}59$$

Gunakan Mox untuk perencanaan

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

Digunakan $\beta = 0{,}65$ untuk perencanaan

b = 400 mm, h = 400 mm, d = 330 mm, d' = 70 mm

$$Mox = 189{,}18 + 300{,}61.\frac{0{,}4}{0{,}4}.\frac{1-0{,}65}{0{,}65}$$

$$= 351{,}04 KNm$$

Kontrol kapasitas Pn pada penampang yang diasumsikan

Dianggap $\rho = \rho' = 0{,}015$

$As = As' = \rho.b.d = 0{,}015 \cdot 400 \cdot 330 = 1980\ mm^2$

Pakai 6D22 dengan $As_{ada} = As'_{ada} = 2281\ mm^2$

Cek eksentrisitas balance (e_b)

$$Xb = \frac{600.d}{600+fy} = \frac{600.330}{600+360} = 206{,}25\ mm$$

$ab = \beta_1 \cdot Xb = 0{,}85 \cdot 206{,}25 = 175{,}31$ mm

$$f's = 600\frac{(Xb-d')}{Xb} = 600\frac{(206{,}25-70)}{206{,}25} = 396{,}36\ MPa > fy = 360\ MPa$$

Dengan demikian digunakan f's = 360 MPa

$Ccb = 0{,}85.f'c.b.ab = 0{,}85.25.400.175{,}31 = 1490135$ N

$Csb = As'(fs' - 0{,}85.f'c) = 2281.(360 - 0{,}85.22{,}5) = 777536$ N

$Tsb = As.fy = 2281.360 = 821160$ N

$Pnb = Ccb + Csb - Tsb = 1490135 + 777536 - 821160$

$= 1446511$ N $= 1446{,}511$ KN

Karena Pnb > Pn, maka patah yang terjadi adalah patah tarik

$$\rho = \frac{As}{bd} = \frac{2281}{400.330} = 0{,}017$$

$$e = \frac{Mox}{Pn} = \frac{351{,}04}{887{,}4} = 0{,}396 \text{ m}$$

$$e' = e + \frac{d - d'}{2} = 396 + \frac{330 - 70}{2} = 526$$

$$1 - \frac{e'}{d} = 1 - \frac{526}{330} = -0{,}59$$

$$m = \frac{fy}{0{,}85 f'c} = \frac{360}{0{,}85.22{,}55} = 18{,}82$$

Kontrol tegangan pada daerah desak :

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

$$= \frac{2281.360}{\frac{396}{(330 - 70)} + 0{,}5} + \frac{400.400.22{,}5}{\frac{3.400.396}{330^2} + 1{,}18}$$

$$= 955289 \text{ N} = 955{,}28 \text{ KN}$$

Kontrol keserasian regangan

$$a = \frac{Pn}{0{,}85.f'c.b} = \frac{955{,}28}{0{,}85.22{,}5.400} = 124{,}87 \text{ mm}$$

$$c = \frac{600}{600 + fy}.d = \frac{600}{600 + 360}.330 = 206{,}25$$

$$f's = 600\frac{(c - d')}{c} = 600\frac{(206{,}25 - 70)}{206{,}25} = 396 > fy = 360 \text{ Mpa}$$

f's > fy, mka f's = fy = 360

Menghitung Momen tahanan nominal actual Moyn untuk lentur uniaksial ekuivalen terhadap sumbu y bila Mox = 0

$$a = \frac{Pn}{0{,}85.f'c.b} = \frac{887{,}4.10^3}{0{,}85.22{,}5.400} = 116 \text{ mm}$$

$$c = \frac{600}{600 + fy}.d = \frac{600}{600+360}.330 = 206{,}25$$

$$f's = 600\frac{(c-d')}{c} = 600\frac{(206{,}25-70)}{206{,}25} = 396 > fy = 360 \text{ Mpa}$$

f's > fy, mka f's = fy = 360

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

$$= \frac{2281.360}{\frac{396}{(330-70)} + 0{,}5} + \frac{400.400.22{,}5}{\frac{3.400.396}{330^2} + 1{,}18}$$

$$= 955289 \text{ N} = 955{,}28 \text{ KN}$$

Menghitung Momen tahanan nominal actual Moyn untuk lentur uniaksial ekuivalen terhadap sumbu x bila Mox = 0

Dalam kondisi ini, b = 400 mm; h = 400 mm; d = 330mm dan As =As'.

$$a = \frac{Pn}{0{,}85.f'c.b} = \frac{887{,}4.10^3}{0{,}85.22{,}5.400} = 116 \text{ mm}$$

$$c = \frac{600}{600 + fy}.d = \frac{600}{600+360}.330 = 206{,}25$$

$$f's = 600\frac{(c-d')}{c} = 600\frac{(206{,}25-70)}{206{,}25} = 396 > fy = 360 \text{ Mpa}$$

f's = fy = 360 Mpa

$$fs = 600\frac{(d-c)}{d} = 600\frac{(330-206{,}25)}{330} = 225 \text{ Mpa}$$

Pn = (0,85 . f'c . b.a) + (As'(fs' – 0.85.f'c)) – As.fs

=(0,85 . 22,5 . 400 . 116)+(2281.(360-0,85.22,5))–(2281.225)

= 1018,9 KN > 887,4 KN ..... Oke!

$$Moxn = 0{,}85.f'c.b.ab\left[\frac{h}{2}-\frac{ab}{2}\right] + As'.fs'\left(\frac{h}{2}-d'\right) + As.fs\left(d-\frac{h}{2}\right)$$

$$= 0{,}85 \cdot 22{,}5 \cdot 400 \cdot 116\left[\frac{400}{2}-\frac{116}{2}\right] + 2281.360\left(\frac{400}{2}-70\right)$$

$$+ \ 2281.225\left(330-\frac{400}{2}\right)$$

= 299,5 KNm

Mencari Mny dengan memasukkan Mny/Mony dan factor β pada grafik

Untuk β = 0,65

$$\frac{Mny}{Moyn} = \frac{300{,}61}{955{,}28} = 0.31$$

dari grafik didapat β = 0,98

Mnx = 0,98.299,5 = 252,74 KNm

Mnx = 293,51 > 189,18KNm ..... oke!

4.6 Perencanaan Pondasi

4.6.1 Daya Dukung Tiang Pancang Tunggal

Berikut ini contoh perhitungan daya dukung tiang pancang tunggal pada join 17 dengan menggunakan data sondir S4.

Data tiang :

Dimensi	= 250 x 250 mm	;	Mx	= 23,61 KNm
Panjang	= 4.5 m	;	My	= 28,16 KNm
Fc' tiang	= 22,5 mpa	;	fc' pile cap = 22,5 mpa	
P tiang	= 1645 KN			

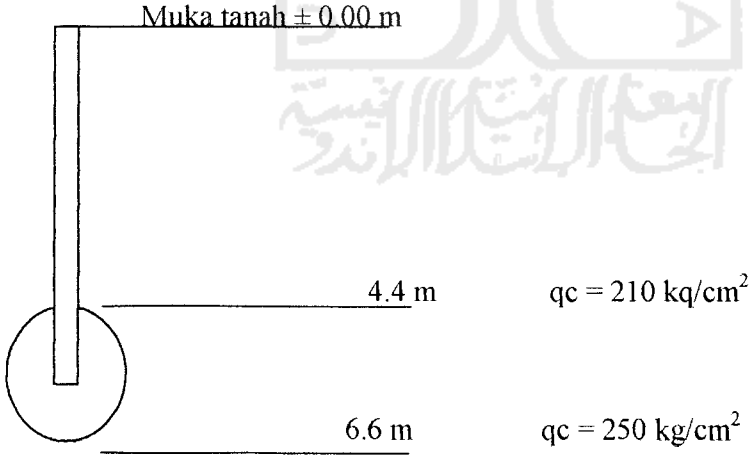
Data sondir :

Kedalaman 8 D di atas ujung tiang : 210 kg/cm^2

Kedalaman 3D di bawah ujung tiang : >210 kg/cm^2 diambil 250 kg/cm^2

JHP pada kedalaman 1,35 m : 70 kg/cm

JHP pada kedalaman 6 m : > 296.67 kg/cm diperkirakan 300 kg/cm



Gambar 4.23 Tiang Pancang Tunggal

Daya dukung ujung tiang

$$qc = \frac{qc_1 + qc_2}{2} = \frac{210 + 250}{2} = 230\ kg/cm^2$$

$$Qp = A_p \times qc = (25 \times 25) \times 230 = 143750\ kg$$

daya dukung selimut tiang

$$Qs = A_s \times JHP = (4 \times 25) \times (300 - 70) = 23000\ kg$$

$$Qu = Qp + Qs = 143750 + 23000 = 166750\ kg$$

$$Qall = \frac{Qp}{Sf} + \frac{Qs}{Sf}$$

$$Qall = \frac{143750}{3} + \frac{23000}{5} = 52516\ kg = 525\ KN$$

berat tiang = 0,25 . 0,25 . 4,5 .24 =6,75 KN

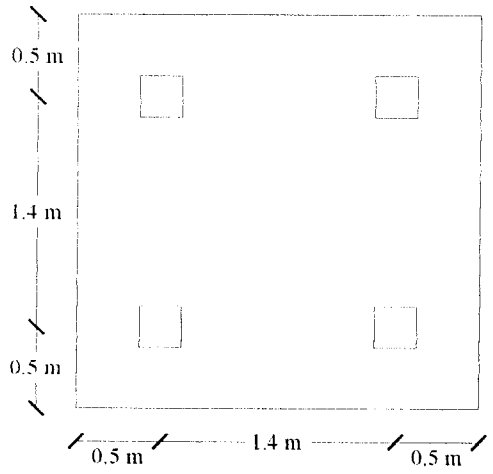
daya dukung tiang yang diijinkan = 525 – 7 = 518,25 KN

beban yang bekerja :

Gaya normal kolom		= 1645 KN
Berat tanah urug	= 0,5 . 2,4 . 2,4 . 18	= 52 KN
Berat poer	= 2,4 . 2,4 . 1 . 24	= 138 KN +
	Total P kolom	= 1835 KN

4.6.2 Daya dukung Tiang Pancang Kelompok

Dari analisis daya dukung tiang pancang tunggal dicoba digunakan 4 tiang pancang dengan formasi tiang pancang seperti gambar berikut :



Formasi Pile Cap
Dengan 4 Tiang D 25 cm

Gambar 4.24 Formasi Tiang Pancang Pada Pile Cap

Dihitung efisiensi tiang pada formasi kelompok tiang dengan rumus Conversi – Labbare

$$\theta = arc\tan {}^{d}/_{s}$$

$$\theta = arc\tan {}^{25}/_{140} = 10{,}12°$$

$$E_g = 1 - \theta \frac{(n-1)\,m + (m-1)n}{90°.m.n}$$

$$E_g = 1 - 10{,}12°.\frac{(2-1)\,2 + (2-1)2}{90°.2.2} = 0{,}887$$

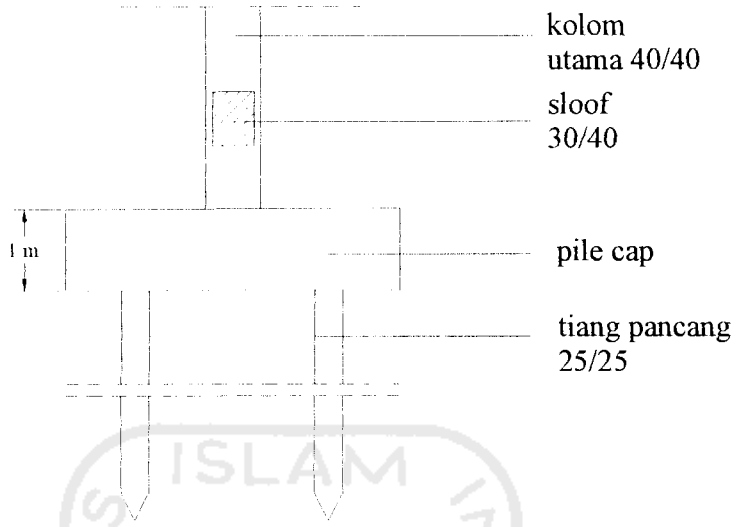
Kapasitas dukung ultimit tiang kelompok

$Q_{ug} = Q_{ult} . n . E_g$

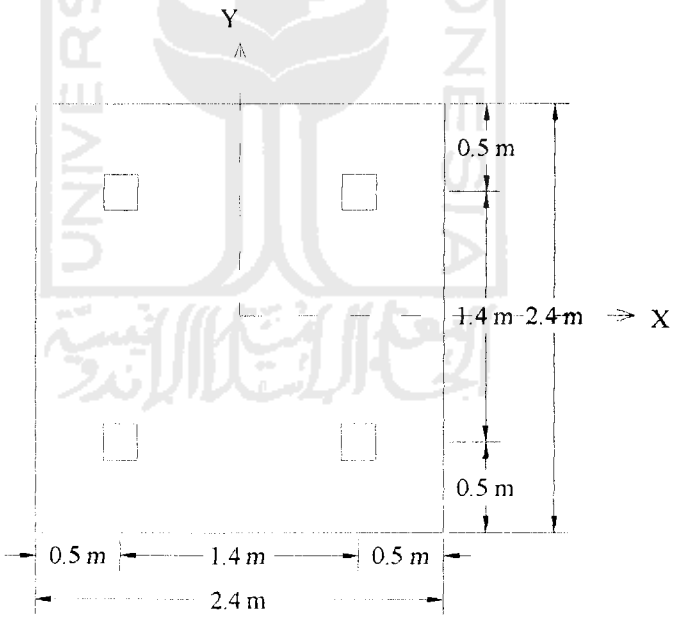
$Q_{ug} = 166750 \text{ kg} . 4 . 0{,}887 = 591629 \text{ kg} = 5916 \text{ KN}$

$$Q_{ijin\,g} = \frac{Q_{ug}}{SF} = \frac{5916}{3} = 1972\,KN > \text{P total} = 1833 \text{ KN}$$

4.6.3 Perencanaan pile cap



Gambar 4.25 Rencana Pile Cap



Gambar 4.26 Susunan Kelompok Tiang Pada Pile Cap

$$\sum X^2 = \sum\left(n.\ Xi^2\right) \quad = 4\ .\ 0{,}7^2 = 1{,}96$$

$$\sum Y^2 = \sum\left(n.\ Yi^2\right) \quad = 4\ .\ 0{,}7^2 = 1{,}96$$

Beban yang bekerja pada pile cap :

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

$$P\max = \frac{1835}{4} + \frac{23{,}61.0{,}7}{1{,}96} + \frac{28{,}16.0{,}7}{1{,}96} = 477\ \text{KN}$$

$$P\min = \frac{1835}{4} - \frac{16{,}53}{1{,}96} - \frac{19{,}71}{1{,}96} = 440\text{KN}$$

perencanaan tebal pile cap

$$D = tp - pb - 0.5\phi tul = 1000 - 80 - \tfrac{1}{2}\ .\ 19 = 910\ \text{mm}$$

Kontrol geser satu arah (geser lentur)

$$Vu(x,y) = n\ tiang\ arah(x,y).P_{max} = 2\ .\ 477 = 954\ \text{KN}$$

$$Vc = \frac{1}{6}\sqrt{fc'}\ .B.D = \frac{1}{6}\sqrt{22{,}5}.2400.910.10^{-3} = 1726\ \text{KN}$$

$$1726\ \text{KN} \geq \frac{Vu}{\phi} = \frac{954}{0{,}6} = 1590\ \text{KN} \quad \text{- OK -}$$

Kontrol geser dua arah (geser pons)

$$\beta_0 = \frac{hc}{bc} = \frac{0{,}4}{0{,}4} = 1$$

$$b_0 = 2.\left((hc + D) + (bc + D)\right)$$

$$b_0 = 2.\left((400 + 910) + (400 + 910)\right) = 5240\ \text{mm}$$

$$Vc1 = \left[1 + \frac{2}{\beta 0}\right].\left[\frac{\sqrt{fc'}}{6}\right].bo\,D$$

$$Vc1 = \left[1+\frac{2}{1}\right].\left[\frac{\sqrt{22,5}}{6}\right].5240.910.10^{-3}=11309 \text{ KN}$$

$$Vc2 = 0,33\ \sqrt{f'c}\ .bo.D$$

$$Vc2 = 0,33.\sqrt{22,5}.5240.910.10^{-3} = 7464 \text{ KN}$$

$$Vu(x,y) = 2\ .\ 477 = 954 \text{ KN}$$

$$Vc = 4983 \text{ KN} \geq Vu/\phi = \frac{954}{0,6} = 1590 \text{ KN}\textbf{.....Ok !}$$

Perencanaan tulangan

$$Mu = 477\ .\ (0,7-0,2).\ 2 = 477$$

$$\frac{Mu}{\phi} = \frac{477}{0,8} = 596,25 \text{ KNm}$$

Digunakan tulangan pokok $\varnothing_{19}$ mm, maka : $A_{1\varnothing} = 284mm^2$

Tebal pile cap = 1000 mm, selimut beton (Pb) = 80 mm

$$d = tf - Pb - 0,5.\ \varnothing_{tul.\ pokok} = 1000 - 80 - 0,5.\ 19 = 910 \text{ mm}$$

$$m = \frac{fy}{0,85.f'c} = \frac{360}{0,85.22,5} = 18,82$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm :

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{596,25.10^6}{1000.910^2} = 0,72 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{360} = 0,0038$$

$$\rho b = \frac{0,85.f'c.\beta_1}{fy}.\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5.0,85}{360}.\left(\frac{600}{600+360}\right) = 0,0282$$

$\rho_{mak} = 0,75.\ \rho b = 0,75.\ 0,0244 = 0,0211$

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

$$= \frac{1}{18,82}\left(1-\sqrt{1-\frac{2.18,82.0,72}{360}}\right) = 0,00204 < \rho_{mak} = 0,0211$$

$< \rho_{min} = 0,0038$

$1,33\ .\ 0,00204 = 0,0027 < \rho_{min} = 0,0038$

$\rho_{pakai} = 0,0027$

$As_{perlu} = \rho_{pakai}\ .\ b\ .\ d = 0,0027\ .\ 1000\ .\ 910 = 2457\ mm^2$

$0,002.b.h = 0,002.1000.400 = 800 mm^2 < As_{perlu}$, maka, $As_{perlu} = 2457\ mm^2$

Jarak antar tulangan :

$$s \leq \frac{A_{01}.b}{As_{perlu}} = \frac{284.1000}{2457} = 115\ mm$$

$s \leq 2.h = 2\ .600 = 1200\ mm$

$s \leq 250\ mm$

→ Dipakai Tulangan Pokok : D_{19} – 100 mm

$$As_{ada} = \frac{A_{10}.1000}{s} = \frac{284.1000}{100} = 2840\ mm^2$$

• Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0,85.f'c.b} = \frac{2840.360}{0,85.22,5.1000} = 53,4\ mm$$

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

$$= 2840.\ 360\ (910 - {}^{53,4}/_{2})\ .\ 10^{-6}$$

$$= 903\ \text{KNm} \geq {}^{Mu}/_{\phi} = 596,25\ \text{KNm} \ldots\ldots\ldots \textbf{Ok !}$$

Perencanaan Tulangan Bagi Pondasi

$As_{bagi} = 0,002.\ b.\ h = 0,002.\ 1000.\ 600 = 1200\ mm^2$

Digunakan tulangan bagi ∅16 mm, maka: $A_{1\varnothing} = 201\ mm^2$

Jarak antar tulangan susut :

$$s \leq \frac{A_{01}.b}{As_{susut}} = \frac{201.1000}{1200} = 167,5\ mm$$

→ Dipakai Tulangan Susut : P_{12} – 160 mm

4.7 Perencanaan Tangga

4.7.1 Perencanaan Tangga Entrance

4.7.1.1 Spesifikasi Struktur

1. Tinggi lantai (h) = 4 m = 400 cm
2. Lebar Bordes (LB) = 1,62 m = 162 cm
3. Tinggi optrade rencana diambil 18 cm

   Jumlah Optrade = 400/18 = 22,2 dipakai 22 Buah

   Tinggi Optrade pakai = 400/22 = 18 cm

   Jumlah Antrade = 22 – 2 = 20

   Diambil Panjang Antrade = 40 cm
4. Sudut kemiringan Tangga = 18/40 = arc tg α $\rightarrow$ α = 24,23°
5. Dimensi Tangga

   Panjang Tangga (Lo)

   Lo = (panjang antrade x jumlah antrade/2) + LB

   = (40 x 20/2) + 162 = 562cm

   Lebar bersih tangga = 500 cm
6. Tebal Pelat bordes Diambil 20 cm

   tebal pelat sisi miring (h') = 20 . sin (90-24,23) = 18,24 cm

4.7.2 Pembebanan

1. Pembebanan Bordes

Beban mati

- Berat sendiri pelat = 0,2 .24 = 4,80 KN/m^2
- Berat spesi = 0,03 .24 = 0,72 KN/m^2
- Berat keramik = 0,01 .24 = 0,24 KN/m^2

qD = 5,8 KN/m^2

Beban hidup

QL = 300 kg/m = 3 KN/m^2

Qu = 1,2 qD + 1,6 qL = 1,2 . 5,8 + 1,6 . 3 = 11,76 KN/m^2

Untuk lebar = 1,62 m → qu = 11,76 . 1 = 19,05 KN/m

2. Pembebanan Tangga

Beban mati

- berat sendiri tangga = (0,18 + 0,18/2).24 = 6,54 KN/m^2
- Berat spesi = 0,03 . 24 = 0,72 KN/m^2
- Lantai keramik = 0,01 . 24 = 0,24 KN/m^2

QD = 7,5 KN/m^2

Beban hidup

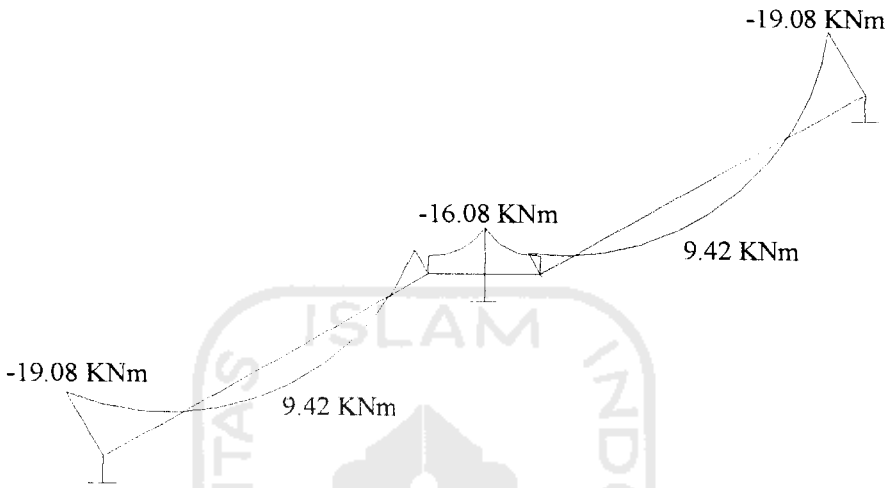
QL = 300 kg/m = 3 KN/m^2

Qu = 1,2 qD + 1,6 qL = 1,2 . 7,5 + 1,6 . 3 = 13,8 KN/m^2

Untuk lebar = 1 m → qu = 13,8 . 1 = 13,8 KN/m

4.7.3 Penulangan Tangga

Perhitungan analisis struktur tangga menggunakan perhitungan program SAP 2000, data – data perhitungan dapat dilihat pada lampiran.



Gambar 4.27 Bidang Momen Tangga Entance

1. Perhitungan pelat bordes

Mu maks = 16,08 KNm

$Mu/\phi = \frac{16,08}{0,8} = 20,1$ KNm

Digunakan tulangan ∅10 mm, sehingga luas tampang 1 tulangan pokok :

A1∅ = 79 mm2

Tebal pelat bordes = 200 mm, selimut beton (pb) = 20 mm

d = h – pb – 0,5 ∅tul pokok = 200 – 20 – ½ .10 = 175 mm

$$\rho b = \frac{0,85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5}{240}.0,85\left(\frac{600}{600+240}\right) = 0,0483$$

$\rho_{maks} = 0,75 \; . \; \rho b = 0,75 \; . \; 0,0483 = 0,0363$

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{240} = 0,0058$$

$$\rho_{pakai} = 0,5 \; . \; \rho_{maks} = 0,5 \; . \; 0,0363 = 0,01815$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.22,5} = 12,55$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{20,1.10^6}{1000.175^2} = 0,66 \text{ MPa}$$

$$\rho_{perlu} = \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot Rn}{fy}}\right) = \frac{1}{12,55} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot 12,55 \cdot 0,66}{240}}\right)$$

$\rho_{perlu} = 0,0028 < \rho_{min} = 0,0058$

$1,33 \; \rho_{perlu} = 0,0037 < \rho_{min} = 0,0058$

sehingga ρ pakai = $1,33 \; \rho_{perlu} = 0,0037$

As perlu = ρ pakai . b .d $\geq 0,002$. b. h

$= 0,0037 \; . \; 1000 \; . \; 175 \geq 0,002 \; . \; 1000 \; . \; 200$

$= 648 \text{ mm}^2 \geq 400 \text{ mm}^2$

As pakai = 648 mm^2

Jarak tulangan : $s \leq \frac{A_1 \cdot b}{As_{perlu}} = \frac{79.1000}{648} = 121,91 \text{mm}$

$\leq 2.h = 2 \; .200 = 400$ mm

≤ 250 mm

Dipakai jarak (s) = 120 mm

Dipakai Tulangan Pokok D10 – 120

$$\text{As ada} = \frac{A_{1\phi} \cdot b}{s_{terpakai}} = \frac{79.1000}{120} = 658{,}33 \text{ mm}^2$$

Kontrol kapasitas lentur pelat bordes:

$$a = \frac{As_{ada} \cdot fy}{0{,}85 \cdot f'c \cdot b} = \frac{658{,}33.240}{0{,}85 \cdot 22{,}5.1000} = 8{,}26 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \cdot \left(d - a/2\right) \geq 1{,}33\, Mu/\phi$$

$$= 658{,}33 \cdot 240 \cdot \left(175 - 8{,}26/2\right).10^{-6} \geq 1{,}33.20.1$$

$$= 26{,}99 \text{ KNm} \geq 26{,}733 \text{ KNm} \quad \ldots\ldots \text{ (Ok..!)}$$

Tulangan bagi pelat bordes

As bagi = 0,002 . b .h = 0,002 . 1000 . 200 = 400 mm^2

Digunakan ∅8mm dengan A1∅ = 50 mm^2

$$\text{Jarak antar tulangan (s)} = \frac{A_{1\Phi}.b}{As.bagi} = \frac{50.1000}{400} = 125 \text{ mm}$$

Dipakai P8-120 mm

2. Perhitungan Pelat Tangga

Mu maks = 19,08 KNm

$$Mu/\phi = \frac{19{,}08}{0.8} = 23{,}85 \text{ KNm}$$

Digunakan tulangan ∅10 mm, sehingga luas tampang 1 tulangan pokok :

A1∅ = 79 mm2

Tebal pelat tangga = 18,24 mm, selimut beton (pb) = 20 mm

d = h – pb – 0,5 ∅tul pokok = 182– 20 – ½ .10 = 157 mm

$$\rho b = \frac{0,85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5}{240}.0,85\left(\frac{600}{600+240}\right) = 0,0483$$

$$\rho_{maks} = 0,75 \;.\; \rho b = 0,75 \;.\; 0,0483 = 0,0363$$

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{240} = 0,0058$$

$$\rho_{pakai} = 0,5 \;.\; \rho_{maks} = 0,5 \;.\; 0,0363 = 0,01815$$

$$m = \frac{fy}{0,85.f'c} = \frac{240}{0,85.22,5} = 12,55$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{23,85.10^6}{1000.157^2} = 0,96 \text{ MPa}$$

$$\rho_{perlu} = \frac{1}{m}\cdot\left(1-\sqrt{1-\frac{2\cdot m\cdot Rn}{fy}}\right) = \frac{1}{12,55}\cdot\left(1-\sqrt{1-\frac{2\cdot 12,55\cdot 0,96}{240}}\right)$$

$\rho_{perlu} = 0,0041 < \rho_{min} = 0,0058$

$1,33\ \rho_{perlu} = 0,0055 < \rho_{min} = 0,0058$

sehingga ρ pakai = $1,33\ \rho_{perlu} = 0,0055$

As perlu = ρ pakai . b .d $\geq 0,002$. b. h

$= 0,0055 \;.\; 1000 \;.\; 157 \geq 0,002 \;.\; 1000 \;.\; 182$

$= 863,5 \geq 364$

As pakai = 863,5 mm^2

Jarak tulangan : $s \leq \frac{A_1 \cdot b}{As_{perlu}} = \frac{79.1000}{863,5} = 91,5$ mm

$\leq 2.h = 2 \;.182 = 364$ mm

≤ 250 mm

Dipakai jarak (s) = 90 mm

Dipakai Tulangan Pokok D10 – 90

$$\text{As ada} = \frac{A_{1\phi} \cdot b}{s_{terpakai}} = \frac{79.1000}{90} = 878 \text{ mm}^2$$

Kntrol kapasitas lentur pelat tangga:

$$a = \frac{As_{ada} \cdot .fy}{0{,}85 \cdot f'c \cdot b} = \frac{878.240}{0{,}85 \cdot 22{,}5.1000} = 11{,}02 \text{ mm}$$

$$Mn = As_{ada} \cdot fy \cdot \left(d - a/2\right) \geq 1{,}33 Mu/\phi$$

$$= 878 \cdot 240 \cdot \left(157 - 11{,}02/2\right).10^{-6} \geq 1{,}33.23{,}85$$

$$= 31{,}92 \text{ KNm} \geq 31{,}72 \text{ KNm} \quad \ldots\ldots \text{ (Ok..!)}$$

Tulangan bagi pelat tangga

As bagi = 0,002 . b .h = 0,002 . 1000 . 182 = 364 mm^2

Digunakan ∅8mm dengan A1∅ = 50 mm^2

$$\text{Jarak antar tulangan (s)} = \frac{A_{1\Phi}.b}{As.bagi} = \frac{50.1000}{364} = 137{,}4 \text{ mm}$$

Dipakai P8-130 mm

4.7.4 Perencanaan Balok Bordes

Dimensi rencana balok :

Tinggi (h) = 450 mm

Lebar (b) = 250 mm

Tinggi efektif balok ($d_{\text{diketahui}}$) = $h_{\text{diketahui}}$ – Pb - $Ø_{\text{sengkang}}$ - ½ $Ø_{\text{tul.rencana}}$

d = 450 – 30 – 10 – ½ 16 = 400 mm

Pembebanan :

- berat pelat bordes = 11,76 . 1,62 = 19,05 KN/m
- berat sendiri = 1,2.0,25.0,45.24 = 3,24 KN/m +

qu = 22,30KN/m

Momen tumpuan :

$$Mu = -\frac{1}{12} qu.L^2 = -\frac{1}{12}.22,30.5,0^2 = -46,46 \text{ KNm}$$

Momen lapangan :

$$Mu = \frac{1}{24} qu.L^2 = \frac{1}{24}.22,30.5,0^2 = 23,23 \text{ KNm}$$

a. Perencanaan tulangan lentur balok bordes

- Tulangan Tumpuan

Mu = 46,46 KNm

$$Mu/\phi = \frac{46,46}{0,8} = 50,1 \text{ KNm}$$

$$\rho b = \frac{0,85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5}{360}.0,85\left(\frac{600}{600+360}\right) = 0,0282$$

$\rho_{maks} = 0,75 \cdot \rho b = 0,75 \cdot 0,0282 = 0,0212$

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{360} = 0,0038$$

$\rho_{pakai} = 0,5 \cdot \rho_{maks} = 0,5 \cdot 0,0212 = 0,0106$

$$m = \frac{fy}{0,85.f'c} = \frac{360}{0,85.22,5} = 18,82$$

$Rn = \rho\, fy\, (1 - \frac{1}{2}\rho\, m)$

$= 0,0106 \cdot 360\ (1 - \frac{1}{2}.0,0106 .18,82) = 3,43$ MPa

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

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{50{,}1.10^6}{3{,}43.250}} = 241{,}7 \text{ mm} < d = 400 \text{ mm}$$, maka dipakai tulangan sebelah.

$$Rn_{ada} = \frac{Mu/\phi}{b.d_{ada}^2} = \frac{51{,}1.10^6}{250.400^2} = 1{,}28 \text{ MPa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}\rho = \frac{1{,}28}{3{,}43}0{,}0106 = 0{,}004 > \rho_{min} = 0{,}0038 \quad \text{(Ok!)}$$

$$As = \rho_{ada}.b.d_{ada} = 0{,}004 \cdot 250 \cdot 400 = 400 \text{ mm}^2$$

Dipakai diameter tulangan D16, maka : $A_1Ø = 201 \text{ mm}^2$

$$n = \frac{As}{A_1\phi} = \frac{400}{201} = 1{,}99 \text{ batang}$$

Dipakai tulangan memanjang **2D16**, maka :

$As_{ada} = 2 \cdot 201 = 402 \text{mm}^2 > As = 400 \text{ mm}^2$

Kontrol Kapasitas Lentur yang terjadi :

$$a = \frac{As_{ada}\, fy}{0{,}85.f'c\, b} = \frac{402.360}{0{,}85.22{,}5.250} = 30{,}27 \text{ mm}$$

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

$$= 402 \cdot 360 \left(400 - \frac{30{,}27}{2}\right). 10^{-6} \geq Mu/\phi$$

$$= 55{,}6 \text{ KNm} > Mu/\phi = 50{,}1 \text{ KNm} \quad \text{(Ok!)}$$

- Tulangan Lapangan

Mu = 23,23 KNm

$$Mu/\phi = \frac{23,23}{0,8} = 29,04 \text{ KNm}$$

$$\rho b = \frac{0,85.f'c}{fy}.\beta_1\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5}{360}.0,85\left(\frac{600}{600+360}\right) = 0,0282$$

$\rho_{maks} = 0,75 \; . \; \rho b = 0,75 \; . \; 0,0282 = 0,0212$

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{360} = 0,0038$$

$\rho_{pakai} = 0,5 \; . \; \rho_{maks} = 0,5 \; . \; 0,0212 = 0,0106$

$$m = \frac{fy}{0,85.f'c} = \frac{360}{0,85.22,5} = 18,82$$

$$Rn = \rho \; fy \; (1 - \tfrac{1}{2}\rho \; m)$$

$$= 0,0106 \; . \; 360 \; (1 - \tfrac{1}{2}.0,0106 \; .18,82)$$

$$= 3,43 \text{ MPa}$$

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

$$d_{perlu} = \sqrt{\frac{Mu/\phi}{Rn.b}} = \sqrt{\frac{29,04.10^6}{3,43.250}} = 184 \text{ mm} < d = 400 \text{ mm, maka dipakai}$$

tulangan sebelah.

$$Rn_{ada} = \frac{Mu/\phi}{b.d_{ada}^2} = \frac{29,04.10^6}{250.400^2} = 0,73 \text{ MPa}$$

$$\rho_{ada} = \frac{Rn_{ada}}{Rn}\rho = \frac{0,73}{3,43}0,0106 = 0,0022 < \rho_{min} = 0,0038$$

$\rho_{pakai} = \rho_{min} = 0{,}0038$

$As = \rho_{ada}.b.d_{ada} = 0{,}0038 \cdot 250 \cdot 400 = 380\ mm^2$

Dipakai diameter tulangan D16, maka : $A_1Ø = 201\ mm^2$

$$n = \frac{As}{A_1\phi} = \frac{380}{201} = 1{,}89 \text{ batang}$$

Dipakai tulangan memanjang **2D16**, maka :

$As_{ada} = 2 \cdot 201 = 402mm^2 > As = 380\ mm^2$

Kontrol Kapasitas Lentur yang terjadi :

$$a = \frac{As_{ada}\, fy}{0{,}85.f'c\ b} = \frac{402.360}{0{,}85.22{,}5.250} = 30{,}27 \text{ mm}$$

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

$$= 402 \cdot 360\,(400 - \frac{30{,}27}{2}).\ 10^{-6} \geq Mu/\phi$$

$$= 55{,}6 \text{ KNm} > Mu/\phi = 29{,}04 \text{ KNm} \qquad \text{(Ok!)}$$

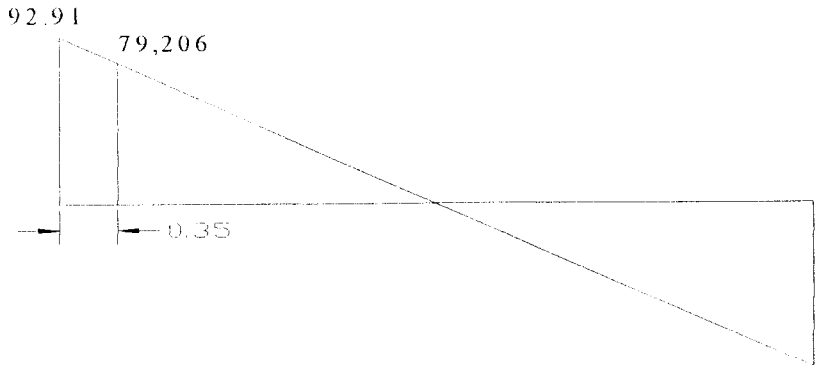
b. Perencanaan tulangan geser balok bordes

Gaya Geser Dukungan

$$\text{Vu dukungan} = \frac{1}{2}.q_U.L$$

$$= \frac{1}{2} \cdot 22{,}30 \cdot 5{,}0 = 55{,}75 \text{ KN}$$

maka $Vu/\phi = \frac{55{,}75}{0{,}6} = 92{,}91$ KN



Gambar 4.28 Diagram Geser Balok Bordes

$$\text{Vu pakai} = \left(\frac{2{,}5 - 0{,}35}{2{,}5}\right).92{,}91 = 79{,}206 \text{ KN}$$

Tegangan Geser Beton (Vc) :

$$\text{Vc} = \left(\frac{1}{6}\sqrt{\text{fc}'}\right).\text{b.d} = \left(\frac{1}{6}\sqrt{22{,}5}\right).250.400 = 78174 \text{ N} = 78{,}174 \text{ KN}$$

Vu pakai = 79,206 > *Vc* = 78,174 , maka perlu tulangan geser.

Vs = Vu pakai – Vc = 79,206 – 78,174 = 1,032 KN

Dipakai sengkang P10, maka Av = $2.\frac{1}{4}.\pi.10^2 = 157 \text{ mm}^2$

Jarak sengkang $s \leq \frac{\text{Av.fy.d}}{\text{Vs}}$

$$s \leq \frac{157.240.400}{1032}$$

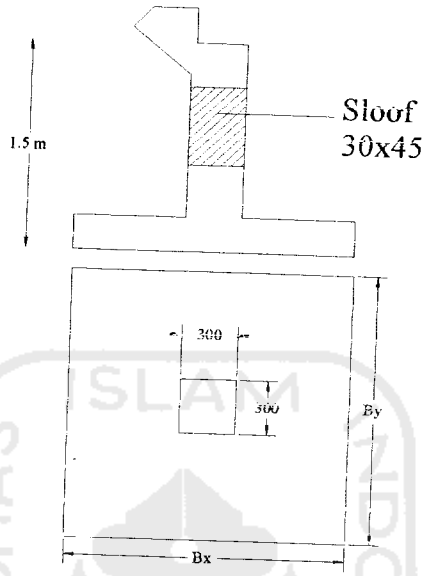
$$s \leq 14604 \text{ mm}$$

$$\leq {}^{\text{d}}\!/_{2} = {}^{400}\!/_{2} = 200 \text{ mm}$$

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

jadi dipakai sengkang **P10 – 200 mm.**

4.7.5 Perencanaan Pondasi Tangga



Gambar 4.29 Pondasi Tangga

Pembebanan :

QD = 7,5 KN/m^2

QL = 300 kg/m = 3 KN/m^2

Qu = 1,2 qD + 1,6 qL = 1,2 . 7,5 + 1,6 . 3 = 13,8 KN/m^2

Untuk lebar = 1 m → qu = 13,8 . 1 = 13,8 KN/m

- Akibat beban tangga = $\frac{2}{\sin 24{,}23^o}$. 3 . 13,8 . ½ = 168,13 KN

- Berat balok diatas pondasi = 1,2x0,30x0,45x5x24 x ½ = 9,72 KN

Pu = 177,84 KN

σ tanah = 250 KN/m^2 γbtanah = 18 KN/m^3

Fc' = 22,5 Mpa γbatu = 22 KN/m^3

fy = 360 Mpa

My = 19,08 KNm

Asumsi tebal pelat (tf) = 400 mm

$\sigma_{netto\ tanah} = \sigma_{tanah} - \Sigma(h.\ \gamma_{beton}) - \Sigma(h.\ \gamma_{tanah})$

$= 250 - (0,4.24) - (1,5.18)$

$= 213\ kN/m^2$

1. Tinjauan Terhadap Beban Tetap

Digunakan pondasi penampang bujur sangkar, dicoba dengan nilai Bx = By = 1,2 m

Luas penampang pelat pondasi :

$A = Bx\ .\ By = 1,2 \times 1,2 = 1,44\ m^2$

Kontrol luas pelat pondasi dan tegangan yang terjadi :

$$\sigma_{terjadi} = \frac{P}{A} \pm \frac{6.My}{Bx^2.By}$$

$$\sigma_{terjadi\ max} = \frac{177,84}{1,44} + \frac{6.19,08}{1,2^2.1,2}$$

$= 189,75\ KN/m^2 < \sigma_{nettotanah} = 213\ KN/m^2$ .....**Ok !**

2. Tinjauan Terhadap Beban Sementara

Eksentrisitas yang terjadi :

$$ex = \frac{My}{P} = \frac{19,08}{177,84} = 0,107\ m$$

$B/6 = 1,2/6 = 0,2 > ex$ (beban eksentrisitas di dalam teras), maka

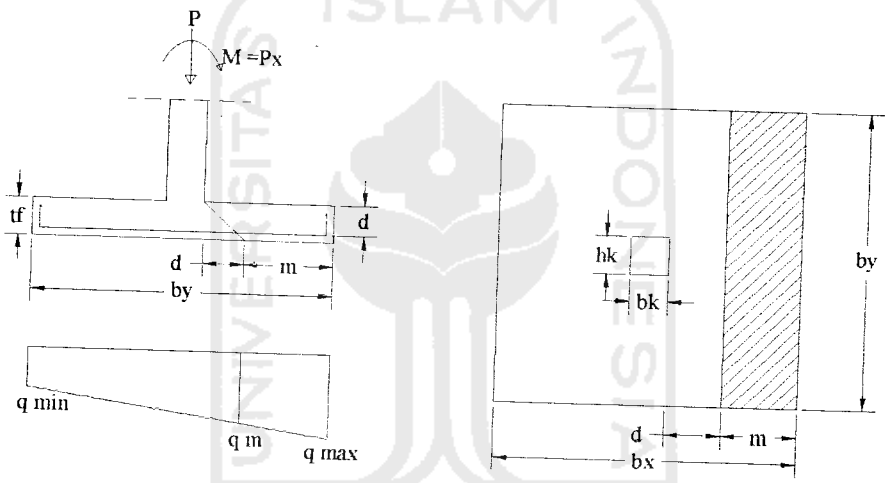
Kontrol tegangan yang terjadi :

$$\sigma_{terjadi} = \frac{P}{A}\left(1 + \frac{6.ex}{Bx}\right)$$

$$= \frac{177,84}{1,44}\left(1 + \frac{6.0,107}{1,2}\right)$$

$= 189,5 \text{ KN/m}^2 < 1,5.\sigma_{netto} = 1,5.\ 214 = 321 \text{ KN/m}^2$ **.....Ok !**

B. Perencananaan Geser Satu Arah



Gambar 4.30 Pondasi Dengan Geser Satu Arah

P = 177,84 KN

My = 19,08 KNm

Jarak pusat tulangan tarik ke serat tekan beton :

$d = tf - Pb - \frac{1}{2}.\varnothing_{tul.\ pokok} = 400 - 70 - \frac{1}{2}.\ 19 = 320,5 \text{ mm} = 0,300 \text{ m}$

$$m = \frac{Bx - bk - 2.d}{2} = \frac{1,2 - 0,30 - 2.0,30}{2} = 0,15 \text{ m}$$

• Tegangan kontak yang terjadi :

$$\sigma_{terjadi} = \frac{P}{A} \pm \frac{6.My}{Bx^2.By}$$

$$\sigma_{terjadi\ max} = \frac{177,84}{1,44} + \frac{6.19,08}{1,2^2.1,2}$$

$$q_{tjd\ mak} = 189,75\ KN/m^2$$

$$q_{tjd\ min} = 57,25\ KN/m^2$$

$$q_{tjd\,m} = \frac{(q_{tjd\ mak} - q_{tjd\ min}).(Bx - m)}{Bx} + q_{tjd\ min}$$

$$= \frac{(189,75 - 57,25).(1,2 - 0,15)}{1,2} + 57,25$$

$$= 173,2\ KN/m^2$$

Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

$$Vu = q_{tjd\,m} . m . Bx = 173,2 . 0,15 . 1,2 = 31,2\ KN$$

$$Vu/\phi = 31,2/0,6 = 60\ KN$$

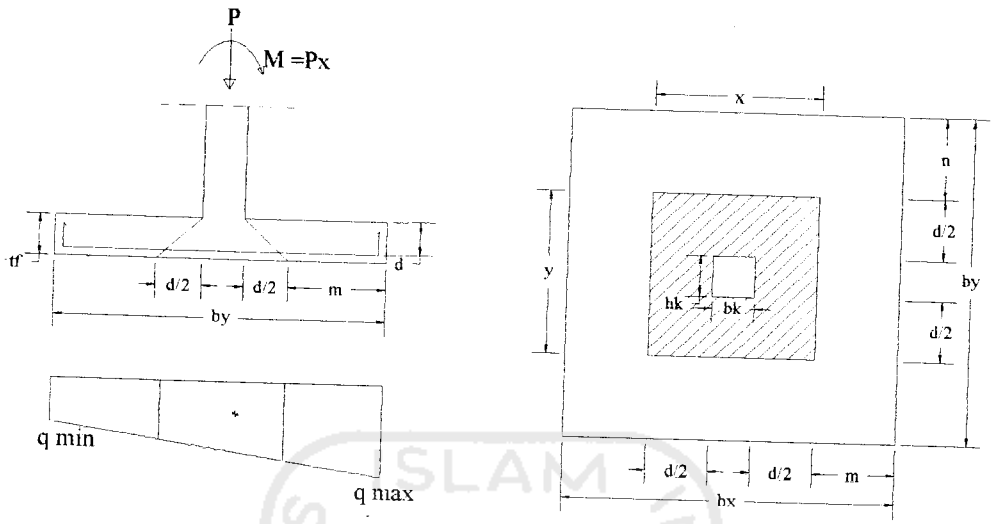
• Kekuatan beton menahan geser:

$$Vc = 1/6 . \sqrt{f'c} . Bx . d = 1/6 . \sqrt{22,5}\ 1,2 . 0,300 . 10^3 = 284\ KN$$

• Kontrol gaya geser :

$$Vc = 284\ KN \geq Vu/\phi = 60\ KN$$ **.....Ok !**

C. Perencananaan Geser Dua Arah



Gambar 4.31 Pondasi dengan geser dua arah

x = bk + d

= 300 + 300

= 600 mm = 0,600 m

y = hk + d

= 300 + 300

= 600 mm = 0,600 m

- Tegangan kontak yang terjadi :

$$q_{terjadi} = \frac{P}{A} \pm \frac{6.My}{Bx^2.By}$$

$$q_{terjadi\ max} = \frac{177,84}{1,44} + \frac{6.19,08}{1,2^2.1,2}$$

$q_{tjd\ mak} = 189{,}75\ KN/m^2$

$q_{tjd\ min} = 57{,}25\ KN/m^2$

$q_{tjd\ pakai} = \frac{1}{2}\,(q_{tjd\ max} + q_{tjd\ min}) = \frac{1}{2}\,(189{,}75 + 57{,}25) = 123{,}5\ KN/m^2$

- Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

$Vu = q_{tjd\ pakai}.\ ((Bx.By) - (x.y))$

$= 123{,}5\ .\ ((1{,}2\ .\ 1{,}2) - (0{,}600.0.600)) = 177{,}48\ KN$

$Vu/\phi = 177{,}48/0{,}6 = 295{,}8\ KN$

- Kekuatan beton menahan geser :

$\beta c = \frac{sisipanjang}{sisipendek} = \frac{y}{x} = \frac{0{,}60}{0{,}60} = 1$

$bo = 2\ .\ (x + y) = 2\ .\ (600 + 600) = 2400\ mm$

$Vc_1 = (1 + 2/\beta c).\ \frac{\sqrt{f'c}}{6}.\ bo.\ d$

$= (1 + 2/1).\ \frac{\sqrt{22{,}5}}{6}.\ 2400.\ 300.\ 10^{-3} = 1707{,}63\ KN$

$Vc_2 = 1/3\ .\ \sqrt{f'c}\ .\ bo.\ d$

$= 1/3.\ \sqrt{22{,}5}\ .\ 2400.\ 300.\ 10^{-3} = 1138{,}42\ KN$

$Vc = 1138{,}42\ KN \geq Vu/\phi = 295{,}8\ KN$**.....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 = 0,30. 0,30 = 0,09 m^2

Luas pelat pondasi (A_2) = Bx . By = 1,2 . 1,2 = 1,44 m^2

$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{1{,}44}{0{,}09}} = 4 > 2$ (jika lebih besar dari 2, dipakai nilai 2)

$\phi.Pn = \phi.(0{,}85.\ f'c.\ A_1.2)$

$= 0{,}7.\ (0{,}85\ .\ 22{,}5\ .\ 0{,}09\ .\ 2).10^3 = 2409{,}75$ KN

• Kuat tumpuan kolom :

$\phi.Pn = \phi.(0{,}85.\ f'c.\ A_1)$

$= 0{,}7.\ (0{,}85.\ 22{,}5.\ 0{,}09\ ).10^3 = 1204{,}87$ KN

• Kontrol kuat tumpuan :

$\phi.Pn_{pondasi} = 2409{,}75$ KN > $\phi.Pn_{kolom} = 1204{,}87$ 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.

$$L = \frac{Bx - bk}{2} = \frac{1{,}2 - 0{,}30}{2} = 0{,}45 \text{ m}$$

$q_{terjadi} = 189{,}75$ KN/m^2

$Mu = 0{,}5.\ q_{terjadi}.\ L^2 = 0{,}5\ .\ 189{,}75\ .\ 0{,}45^2 = 19{,}2$ KNm

$$Mu / \phi = \frac{19,2}{0,8} = 24 \text{ KNm}$$

- Digunakan tulangan pokok $\varnothing_{16}$ mm, maka : $A_{1\varnothing} = 201 mm^2$
- Tebal pelat pondasi : tf = 400 mm, selimut beton (Pb) = 70 mm

d = tf – Pb – 0,5. $\varnothing_{tul.\ pokok}$ = 400 – 70 – 0,5. 16 = 322 mm

$$m = \frac{fy}{0,85.f'c} = \frac{360}{0,85.22,5} = 18,82$$

Koefisien ketahanan (Rn), diambil nilai b tiap 1000 mm :

$$Rn = \frac{Mu/\phi}{b.d^2} = \frac{24.10^6}{1000.322^2} = 0,23 \text{ MPa}$$

Rasio Tulangan :

$$\rho_{min} = \frac{1,4}{fy} = \frac{1,4}{360} = 0,0038$$

$$\rho b = \frac{0,85.f'c.\beta_1}{fy}\left(\frac{600}{600+fy}\right) = \frac{0,85.22,5.0,85}{360}\left(\frac{600}{600+360}\right) = 0,0282$$

$$\rho_{mak} = 0,75.\ \rho b = 0,75.\ 0,0244 = 0,0211$$

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

$$= \frac{1}{18,82}\left(1-\sqrt{1-\frac{2.18,82.0,23}{360}}\right) = 0,0005 < \rho_{mak} = 0,0211$$

$$< \rho_{min} = 0,0038$$

$1,33\ .\ 0,0005 = 0,00066 < \rho_{min} = 0,0038$

$\rho_{pakai} = 0,00066$

$As_{perlu} = \rho_{pakai}\ .\ b\ .\ d = 0,00066\ .\ 1000\ .\ 322 = 214\ mm^2$

$0{,}002.b.h = 0{,}002.1000.400 = 800 mm^2 > As_{perlu}$, maka, $As_{perlu} = 800\ mm^2$

Jarak antar tulangan :

$$s \leq \frac{A_{\theta 1}.b}{As_{perlu}} = \frac{201.1000}{800} = 251\ mm$$

$$s \leq 2.h = 2.400 = 800\ mm$$

$$s \leq 250\ mm$$

→ Dipakai Tulangan Pokok : $D_{16} - 250$ mm

$$As_{ada} = \frac{A_{10}.1000}{s} = \frac{201.1000}{250} = 804\ mm^2$$

- Kontrol Kapasitas Lentur Pelat pondasi :

$$a = \frac{As_{ada}.fy}{0{,}85.f'c.b} = \frac{804.360}{0{,}85.22{,}5.1000} = 15{,}13\ mm$$

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

$$= 804 . 360 \left(322 - \frac{15{,}13}{2}\right) . 10^{-6}$$

$$= 91{,}01\ KNm \geq \frac{Mu}{\phi} = 24\ KNm \ldots\ldots\ldots Ok !$$

Perencanaan Tulangan Bagi Pondasi

$As_{bagi} = 0{,}002.\ b.\ h = 0{,}002.\ 1000.\ 400 = 800\ mm^2$

- Digunakan tulangan bagi ∅16 mm, maka: $A_{1\varnothing} = 201\ mm^2$

Jarak antar tulangan bagi :

$$s \leq \frac{A_{01}.b}{As_{susut}} = \frac{201.1000}{800} = 251\ mm$$

→ Dipakai Tulangan bagi : $P_{16} - 251$ mm