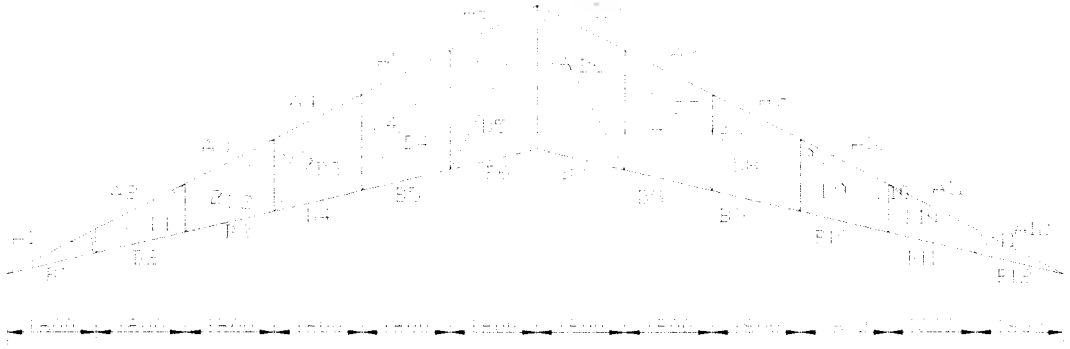
BAB IV

PERHITUNGAN KONSTRUKSI

4.1 Perhitungan Perencanaan Atap

Hasil perencanaan atap Gedung Administrasi Universitas Sanata Dharma Yogyakarta sebagai berikut :

1. Data :
   a. Gording = 6,13 kg/m
   b. Eternit dan plafond = 18 kg/m^2
   c. Penutup atap = 50 kg/m^2
   d. Beban hidup = 20 kg/m^2
   e. Fy = 2400 kg/cm^2
   f. E = 2,1x10^6 kg/cm^2
   g. Fu = 3600 kg/cm^2



Gambar 4.1 Rencana kuda-kuda

2. Hasil perhitungan perencanaan atap

Hitungan sambungan pada setengah bentang

Data :

Tebal pelat sambung(tp) = 1 cm

Fy = 2400 kg/cm^2

Fu = 3600 kg/cm^2

Ø baut (5/8 inchi) = 1,59 cm

Perhitungan :

Kekuatan untuk satu baut (n) = 1 baut

$P_{tumpuan}$ = tp. D_{baut}. 1,2. Fu. n

= 1. 1,59. 1,2. 3600. 1

= 6868,8 kg

P_{geser} = A_{baut}. 0,17. Fu. 2.n

= 1/4.π. D^2_{baut}. 0,17. Fu. 2.n

= 1/4.π. $(1,59)^2$. 0,17. 3600. 2.1

= 2430,332 kg

Dipakai nilai P yang terkecil = 2430,332 kg

Untuk batang A1(tekan): gaya batang(P) = 14754,61 Kg

n = P/P_{pakai}

= 14754,61 / 2430,332

= 6,071

n pakai = 7 baut

Tabel 4.1 Hasil perhitungan perencanaan atap

Batang	Panjang (m)	Jenis batang	Profil	n baut tiap join
A1	2,078	Tekan	2 L 70 x 70 x7	7 Ø 5/8"
A2	2,078	Tekan	2 L 70 x 70 x7	7 Ø 5/8"
A3	2,078	Tekan	2 L 70 x 70 x7	6 Ø 5/8"
A4	2,078	Tekan	2 L 70 x 70 x7	5 Ø 5/8"
A5	2,078	Tekan	2 L 70 x 70 x7	5 Ø 5/8"
A6	2,078	Tekan	2 L 70 x 70 x7	4 Ø 5/8"
A7	2,078	Tekan	2 L 70 x 70 x7	4 Ø 5/8"
A8	2,078	Tekan	2 L 70 x 70 x7	5 Ø 5/8"
A9	2,078	Tekan	2 L 70 x 70 x7	5 Ø 5/8"
A10	2,078	Tekan	2 L 70 x 70 x7	6 Ø 5/8"
A11	2,078	Tekan	2 L 70 x 70 x7	7 Ø 5/8"
A12	2,078	Tekan	2 L 70 x 70 x7	7 Ø 5/8"
B1	1,863	Tarik	2 L 70 x 70 x7	6 Ø 5/8"
B2	1,863	Tarik	2 L 70 x 70 x7	5 Ø 5/8"
B3	1,863	Tarik	2 L 70 x 70 x7	5 Ø 5/8"
B4	1,863	Tarik	2 L 70 x 70 x7	4 Ø 5/8"
B5	1,863	Tarik	2 L 70 x 70 x7	4 Ø 5/8"
B6	1,863	Tarik	2 L 70 x 70 x7	3 Ø 5/8"
B7	1,863	Tarik	2 L 70 x 70 x7	3 Ø 5/8"
B8	1,863	Tarik	2 L 70 x 70 x7	4 Ø 5/8"

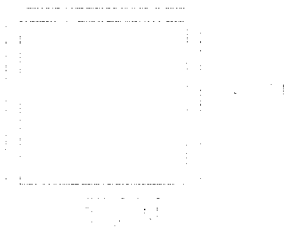
B9	1,863	Tarik	2 L 70 x 70 x7	7 Ø ½ "
B10	1,863	Tarik	2 L 70 x 70 x7	7 Ø ½ "
B11	1,863	Tarik	2 L 70 x 70 x7	8 Ø ½ "
B12	1,863	Tarik	2 L 70 x 70 x7	9 Ø ½ "
D1	2,406	Tarik	2 L 50 x 50 x5	2 Ø ½ "
D2	2,807	Tarik	2 L 50 x 50 x5	2 Ø ½ "
D3	3,208	Tarik	2 L 50 x 50 x5	2 Ø ½ "
D4	3,609	Tarik	2 L 50 x 50 x5	2 Ø ½ "
D5	4,01	Tarik	2 L 50 x 50 x5	2 Ø ½ "
D6	4,01	Tarik	2 L 50 x 50 x5	2 Ø ½ "
D7	3,609	Tarik	2 L 50 x 50 x5	2 Ø ½ "
D8	3,208	Tarik	2 L 50 x 50 x5	2 Ø ½ "
D9	2,807	Tarik	2 L 50 x 50 x5	2 Ø ½ "
D10	2,406	Tarik	2 L 50 x 50 x5	2 Ø ½ "
V1	0,557	Tekan	2 L 50 x 50 x5	2 Ø ½ "
V2	1,114	Tekan	2 L 50 x 50 x5	2 Ø ½ "
V3	1,671	Tekan	2 L 50 x 50 x5	2 Ø ½ "
V4	2,228	Tekan	2 L 50 x 50 x5	2 Ø ½ "
V5	2,785	Tekan	2 L 50 x 50 x5	2 Ø ½ "
V6	3,342	Tarik	2 L 50 x 50 x5	3 Ø ½ "
V7	2,785	Tekan	2 L 50 x 50 x5	2 Ø ½ "
V8	2,228	Tekan	2 L 50 x 50 x5	2 Ø ½ "

V9	1,671	Tekan	2 L 50 x 50 x5	2 Ø ½ "
V10	1,114	Tekan	2 L 50 x 50 x5	2 Ø ½ "
V11	0,557	Tekan	2 L 50 x 50 x5	2 Ø ½ "

4.2 Perencanaan Pelat Atap

Perhitungan pelat atap Gedung Administrasi Sanata Dharma Yogyakarta sebagai berikut :

1. Diketahui :
   a. Tebal lapis kedap air : 1 cm
   b. Tebal finishing beton : 2 cm
   c. fc' (mutu beton) : 25 Mpa
   d. Fy polos (mutu baja) : 240 Mpa
   e. Bj beton : 24 KN/m^3
   f. Beban hidup : 100 Kg/m^2
   g. Di gunakan tebal pelat atap (ht) = 100 mm
   h. Penutup beton (pb) = 40 mm
   i. Digunakan tulangan berdiameter(Ø) = 8 mm.
2. Perencanaan pembebanan :



Gambar 4.2 Dimensi pelat atap

a. Beban mati (WD)

1) berat pelat = tebal pelat x bj beton = 0,1x24 = 2,4KN/m^2

2) berat finishing beton = tebal finishing beton x bj beton

= 0,02x24 = 0,48 KN/m^2

3) berat lapis kedap air = tebal lapis kedap air x bj beton

= 0,01x24 = 0,24 KN/m^2

4) berat langit^2 + penggantung = 11 + 7 = 0,18 KN/m^2

WD total = 3,3 KN/m^2

b. beban hidup (WL) = 1,0 KN/m^2

c. beban ultimit (qu)

qu = 1,2.WD + 1,6.WL = 1,2x3,3 + 1,6x1,0 = 5,56 KN/m^2

3. Perhitungan momen

Mu (lx, tx, ly, ty) = (±) 0,001. qu. L^2. x

Hitung nilai ly/lx untuk mendapatkan faktor perhitungan momen x yang dapat dilihat pada buku *Gideon seri 4 tabel 14.*

a. Momen lapangan arah x dan y

$\frac{ly}{lx} = \frac{3,6}{3,6} = 1$ lihat tabel didapat x = 25

maka Mlx = Mly = ± 0,001. qu. L^2. x

= ± 0.001.5,56. $3,6^2$. 25

= ± 1,801 KNm

b. Momen tumpuan arah x dan y

$\frac{ly}{lx} = \frac{3,6}{3,6} = 1$ lihat tabel didapat x = 51

maka Mtx = Mty = ± 0,001. qu. L^2. x

= ± 0.001.5,56. $3,6^2$. 51

= ± 3,675 KNm

c. Perencanaan tulangan

d = t.pelat – pb – 0,5.Øtul

= 100 – 40 – 0,5. 8

= 56 mm

γd = 0,9. d = 0,9. 56 = 50,4 mm

d. Menghitung luasan tulangan lapangan atau tumpuan arah x dan y

Untuk $Aslx = Asly = \dfrac{\frac{Mu}{\phi}}{\gamma d \cdot fy}$

$= \dfrac{(\frac{1,801}{0,8}) \cdot 10\text{^}6}{50,4 \cdot 240}$

= 186,115 mm^2

Untuk $Astx = Asty = \dfrac{(Mu/\phi)_{tump(lx)}}{(Mu/\phi)_{lap(lx)}} \cdot Aslx$

$= \dfrac{(3,675/0,8)}{(1,801/0,8)} \cdot 186,115$

= 379,774 mm^2

e. $As\min = \dfrac{1,4}{fy} \cdot b \cdot d$, b = 1000 mm (tinjauan per-meter)

$= \dfrac{1,4}{240} \cdot 1000 \cdot 56$

=326,667 mm^2

f. 1,33.Aslx = 1,33 Asly = 1,33. 186,115 = 247,533 mm^2

1,33.Astx = Asty = 1,33.379,774 = 505,1mm^2

g. Check nilai As > As min

Jika As < As min dan 1,33 As > As min ⟶ As min dipakai

Jika As < As min dan 1,33 As < As min ⟶ 1,33 As dipakai

Jika As > As min ⟶ Dipakai As

As pakai lx = ly : 247,533 mm^2

As pakai tx = ty : 379,774 mm^2 > Ass = 0.002.1000.100

= 200 mm^2

.....Ok

h. Menghitung $A\phi = 1/4 \cdot \pi \cdot d^2$

Dipakai $Ø_{tul}$ = 8 mm

$A\phi$ = 50,24 mm^2

i. Menghitung jarak tulangan

$$x = \frac{A\phi \cdot 1000}{As_{pakai}}$$

x arah $lx = ly = \frac{50,24 \cdot 1000}{247,533}$

= 202,91mm

Dipakai jarak tulangan = 200 mm ≤ 2.ht = 2.100 = 200 mm ..... Ok

x arah $tx = ty = \frac{50,24 \cdot 1000}{379,774}$

= 132,2913 mm

Dipakai jarak tulangan = 130 mm ≤ 2.ht = 200 mm ..... Ok

j. Dicoba tulangan :

Arah lx = arah ly : $P_{8\text{-}200}$, As = (Aθ.1000)/x

= (50,24.1000)/200

= 251,2 mm^2

Arah tx = arah ty : $P_{8\text{-}130}$, As = (Aθ.1000)/x

=(50,24.1000)/130

= 386,4615 mm^2

k. Kontrol Mn

$$a = \frac{As \cdot fy}{0{,}85 \cdot fc' \cdot b} \qquad Mn = As.fy\ (d\text{-}a/2) > Mu/Ø$$

arah lx a = arah ly a = $\frac{251{,}2.240}{0{,}85.25.1000} = 2{,}84$ mm

Mn = $251{,}2.240.(56\text{-}(2{,}84/2)).10^{-6}$

= 3,29 KNm > 1,33 Mu/Ø = 1,33.(1,801/0,8) = 2,994 KNm ..... Ok

arah tx a = arah ty a = $\frac{386{,}461.240}{0{,}85.25.1000} = 4{,}365$ mm

Mn = $386{,}461.240.(56\text{-}(4{,}365/2)).10^{-6}$

= 4,991 KNm > Mu/Ø = 4,594 KNm .....Ok

l. Menghitung tulangan susut

As susut = 0,002.b.h

= 0,002.1000.100

= 200 mm^2

Dipakai tulangan Ø = 8 mm ⟶ Aθ = 50,24 mm^2

Jarak tulangan susut (s) :

$$s = \frac{A\phi \cdot 1000}{As} = \frac{50{,}24 \cdot 1000}{200}$$

s = 251,2 mm , dipakai s = 200 mm ≤ 2.ht = 200 mm

Digunakan tulangan $P_{8\text{-}200}$

Tabel 4.2 Hasil Perencanaan Pelat Atap (Type A)

Perencanaan Pelat Atap				
	Pelat Atap			
Perhitungan	Mlx	Mly	Mtx	Mty
Mu (knm)	1.80144	1.80144	3.674938	3.674938
Mr = Mu/Ø	2.2518	2.2518	4.593672	4.593672
1,33 Mu/Ø	2.994894	2.994894	6.109584	6.109584
As (mm^2)	186.1607143	186.16071	379.7679	379.7679
1,33 As (mm^2)	247.59375	247.59375	505.0913	505.0913
As min (mm^2)	326.6666667	326.66667	326.6667	326.6667
As perlu (mm^2)	247.59375	247.59375	379.7679	379.7679
Jrk tul (mm)	202.913038	202.91304	132.2913	132.2913
Pakai Tulangan	P8-200	P8-200	P8-130	P8-130
As pakai (mm)	251.2	251.2	386.4615	386.4615
A	2.837082353	2.8370824	4.364742	4.364742
Mn	3.29060699	3.290607	4.991626	4.991626
Kontrol	Aman	Aman	Aman	Aman
Tulangan Susut	$P_{8\text{-}200}$	$P_{8\text{-}200}$	---	---

4.3 Perencanaan Pelat Lantai

Perhitungan pelat lantai Gedung Administrasi Universitas Sanata Dharma Yogyakarta sebagai berikut :

1. Diketahui :
   a. Tebal pasir : 0,05 m
   b. Tebal spesi : 0,03 m
   c. Tebal keramik : 0,01 m
   d. fc' (mutu beton) : 25 Mpa

e. Fy polos (mutu baja) : 240 Mpa

f. Bj pasir : 16 KN/m^3

g. Bj spesi : 24 KN/m^3

h. Bj keramik : 20 KN/m^3

i. Bj beton : 24 KN/m^3

j. Beban hidup : 400 Kg/m^2

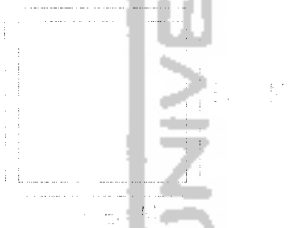
k. Digunakan tebal pelat lantai (ht) = 120 mm

l. Penutup beton (pb) = 20 mm

m. Digunakan tulangan berdiameter(Ø) = 8 dan 10 mm.

n. Θ = 0,8 (faktor reduksi kekuatan untuk pembebanan lentur tanpa aksial)

2. Perencanaan pembebanan :



Gambar 4.3 Dimensi pelat lantai

a. Beban mati (WD)

1) berat pelat = t pelat x bj beton = 0,12 x 24 = 2,88 KN/m^2

2) berat pasir = t pasir x bj pasir = 0,05 x 16 = 0,8 KN/m^2

3) berat spesi = t spesi x bj spesi = 0,03 x 24 = 0,72 KN/m^2

4) berat keramik = t keramik x bj keramik = 0,01 x 20 = 0,2 KN/m^2

WD total = 4,6 KN/m^2

b. beban hidup (WL) = 4,0 KN/m^2

c. beban ultimit (qu)

qu = 1,2.WD + 1,6.WL = 1,2 x 4,6 + 1,6 x 4,0 = 11,92 KN/m^2

3. Perhitungan momen

Mu (lx, tx, ly, ty) = (±) 0,001. qu. L^2. x

Hitung nilai ly/lx untuk mendapatkan faktor perhitungan momen x yang dapat dilihat pada buku *Gideon seri 4 tabel 14.*

a. Momen lapangan arah x dan y

$\frac{ly}{lx} = \frac{3,6}{3,6} = 1$ lihat tabel didapat x = 25

maka Mlx = Mly = ± 0,001. qu. L^2. x

= ± 0.001.11,92 .$3,6^2$. 25

= ± 3,862 KNm

b. Momen tumpuan arah x dan y

$\frac{ly}{lx} = \frac{3,6}{3,6} = 1$ lihat tabel didapat x = 51

maka Mtx = Mty = ± 0,001. qu. L^2. x

= ± 0.001.11,92. $3,6^2$. 51

= ± 7,879 KNm

c. Perencanaan tulangan

d = t.pelat – pb – 0,5.Øtul

= 120 – 20 – 0,5. 8

= 96 mm

γd = 0,9. d = 0,9. 96 = 86,4 mm

d. Menghitung luasan tulangan lapangan atau tumpuan arah x dan y

Untuk $Aslx = Asly = \dfrac{\dfrac{Mu}{\phi}}{\gamma d \cdot fy}$

$$= \frac{(^{3,862}_{0,8}) \cdot 10\text{^}6}{86,4 \cdot 240}$$

$$= 232,808 \text{ mm}^2$$

Untuk $Astx = Asty = \dfrac{(Mu/\phi)_{tump(tx)}}{(Mu/\phi)_{lap(lx)}} \cdot Aslx$

$$= \frac{(7,879/0,8)}{(3,862/0,8)} \cdot 232,808$$

$$= 474,96 \text{ mm}^2$$

e. $As\min = \dfrac{1,4}{fy} \cdot b \cdot d$, b = 1000 mm (tinjauan per-meter)

$$= \frac{1,4}{240} \cdot 1000 \cdot 96$$

$$= 560 \text{ mm}^2$$

f. 1,33.Aslx = 1,33 Asly = 1,33. 232,808 = 309,635 mm^2

1,33.Astx = Asty = 1,33. 474,96 = 631,697 mm^2

g. Check nilai As > As min

Jika As < As min dan 1,33 As > As min ⟶ As min dipakai

Jika As < As min dan 1,33 As < As min ⟶ 1,33 As dipakai

Jika As > As min ⟶ Dipakai As

As pakai lx = ly : 309,635 mm^2

As pakai tx = ty : 560 mm^2 > Ass = 0.002.1000.120 = 240 mm^2 ..... Ok

h. Menghitung $A\phi = 1/4 \cdot \pi \cdot d^2$

Dipakai $\emptyset_{tul}$ = 8 mm $\longrightarrow A\phi = 50{,}24$ mm^2

$\emptyset_{tul}$ = 10 mm $\longrightarrow A\phi = 78{,}54$ mm^2

i. Menghitung jarak tulangan

$$x = \frac{A\phi \cdot 1000}{As_{pakai}}$$

x arah $lx = ly = \dfrac{50{,}24 \cdot 1000}{309{,}635}$

= 162,253 mm

Dipakai jarak tulangan = 160 mm ≤ 2.ht = 2.100 = 200 mm ..... Ok

x arah $tx = ty = \dfrac{78{,}54 \cdot 1000}{560}$

= 140,25 mm

Dipakai jarak tulangan = 140 mm ≤ 2.ht = 200 mm ..... Ok

j. Dicoba tulangan :

Arah lx = arah ly : $P_{8\text{-}160}$, As = (Aθ.1000)/x

= (50,27.1000)/160

= 314,188 mm^2

Arah tx = arah ty : $P_{8\text{-}140}$, As = (Aθ.1000)/x

= (78,54.1000)/140

= 561 mm^2

k. Kontrol Mn

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

Mn = As.fy (d-a/2) > Mu/Ø

arah lx a = arah ly a = $\frac{314,188.240}{0,85.25.1000}$ = 3,55 mm

Mn =314,188.240.(96-(3,55/2)).10^{-6}

= 7,105 KNm > 1,33 Mu/Ø = 6,42 KNm ..... Ok

arah tx a = arah ty a = $\frac{561.240}{0,85.25.1000}$ = 6,336 mm

Mn = 561.240.(96-(6,336/2)).10^{-6}

= 12,499 KNm > Mu/Ø = 9,849 KNm .....Ok

l. Menghitung tulangan susut/bagi

As susut = 0,002.b.h

= 0,002.1000.120

= 240 mm^2

Dipakai tulangan Ø = 8 mm ⟶ Aθ = 50,24 mm^2

Jarak tulangan susut (s) :

$$s = \frac{A\phi \cdot 1000}{Ass} = \frac{50,24 \cdot 1000}{240}$$

s = 209,333 mm , dipakai s = 200 mm ≤ 2.ht = 200 mm .....Ok

Digunakan tulangan $P_{8\text{-}200}$

Selanjutnya hasil perhitungan pelat lantai dan pelat atap terdapat pada tabel dengan perhitungan lengkapnya terdapat pada lampiran.

Tabel 4.3 Hasil Perencanaan Pelat Lantai Type A

	Pelat Lantai Hall			
Perhitungan	Mlx	Mly	Mtx	Mty
Mu (knm)	3.86208	3.86208	7.878643	7.878643
Mr = Mu/Ø	4.8276	4.8276	9.848304	9.848304
As (mm^2)	232.8125	232.8125	474.9375	474.9375
1,33 As (mm^2)	309.640625	309.64063	631.6669	631.6669
As min (mm^2)	560	560	560	560
As perlu (mm^2)	309.640625	309.64063	560	560
Jrk tul (mm)	162.2526114	162.25261	140,25	140,25
Tulangn pokok	P8-160	P8-160	P10-140	P10-140
As pakai (mm)	334.9333333	334.93333	561	561
a	3.782776471	3.7827765	6,336	6,336
Mn	7.564826648	7.5648266	12,499	12,499
Kontrol	Aman	Aman	Aman	Aman
Tulangan Bagi	---	---	P8-200	P8-200

Tabel 4.4 Hasil Perencanaan Pelat Lantai Type A

	Plat lantai Bank			
Perhitungan	Mlx	Mly	Mtx	Mty
Mu (knm)	3.08448	3.08448	6.292339	6.292339
Mr = Mu/Ø	3.8556	3.8556	7.865424	7.865424
As (mm^2)	185.9375	185.9375	383.3053	383.3053
1,33 As (mm^2)	247.2969	247.2969	509.796	509.796
As min (mm^2)	560	560	554.1667	554.1667
As perlu (mm^2)	247.2969	247.2969	509.796	509.796
Jrk tul (mm)	203.1566	203.1566	153.9832	153.9832
Tulangan pokok	P8 - 200	P8 - 200	P10 - 150	P10 -150
As pakai (mm)	251.2	251.2	523.3333	523.3333
A	2.837082	2.837082	5.910588	5.910588
Mn	5.702127	5.702127	11.56082	11.56082
Kontrol	Aman	Aman	Aman	Aman
Tulangan bagi	---	---	P8 - 200	P8 - 200

Tabel 4.5 Hasil Perencanaan Pelat Atap Type B

	Pelat Atap Type B			
Perhitungan	Mlx	Mly	Mtx	Mty
Mu (knm)	6.218312	1.850688	9.753126	6.662477
Mr = Mu/Ø	7.77289	2.31336	12.19141	8.328096
As (mm^2)	654.2836	194.7273	1045.217	714
1,33 As (mm^2)	870.1972	258.9873	1390.138	949.62
As min (mm^2)	320.8333	320.8333	315	315
As perlu (mm^2)	654.2836	258.9873	1045.217	714
Jrk tul (mm)	119.9785	303.1037	108.1498	158.3193
Tulangan pokok	P10 - 110	P10 - 200	P12 -100	P12 -150
As pakai (mm)	713.6364	392.5	1130.4	753.6
a	8.059893	4.432941	12.76687	8.511247
Mn	8.72978	4.972208	12.91818	8.996967
Kontrol	Aman	Aman	Aman	Aman
Tulangan susut	P8 - 200	P8 - 200	---	---

Tabel 4.6 Hasil Perencanaan Pelat Atap Type C

	Pelat Atap Type C			
Perhitungan	Mlx	Mly	Mtx	Mty
Mu (knm)	1.17872	0.3336	1.80144	1.20096
Mr = Mu/Ø	1.4734	0.417	2.2518	1.5012
As (mm^2)	121.8089	34.47421	186.1607	124.1071
1,33 As (mm^2)	162.0058	45.85069	247.5938	165.0625
As min (mm^2)	326.6667	326.6667	326.6667	326.6667
As perlu (mm^2)	162.0058	45.85069	247.5938	165.0625
Jrk tul (mm)	310.1124	1095.73	202.913	304.3696
Tulangan pokok	P8 - 200	P8 - 200	P8 - 200	P8 - 200
As pakai (mm)	251.2	251.2	251.2	251.2
a	2.837082	2.837082	2.837082	2.837082
Mn	3.290607	3.290607	3.290607	3.290607
Kontrol	Aman	Aman	Aman	Aman
Tulangan susut	P8 - 200	P8 - 200	---	---

Tabel 4.7 Hasil Perencanaan Pelat Atap Type D

	Pelat Atap Type D			
Perhitungan	Mlx	Mly	Mtx	Mty
Mu (knm)	4.179341	1.080864	5.908723	3.819053
Mr = Mu/Ø	5.224176	1.35108	7.385904	4.773816
As (mm^2)	439.7455	113.7273	621.7091	401.8364
1,33 As (mm^2)	584.8615	151.2573	826.8731	534.4424
As min (mm^2)	320.8333	320.8333	320.8333	320.8333
As perlu (mm^2)	439.7455	151.2573	621.7091	401.8364
Jrk tul (mm)	178.5124	518.9833	126.2648	195.3532
Tulangan pokok	P10 - 170	P10 - 200	P10 - 120	P10 -190
As pakai (mm)	461.7647	392.5	654.1667	413.1579
a	5.215225	4.432941	7.388235	4.666254
Mn	5.806309	4.972208	8.055024	5.222336
Kontrol	Aman	Aman	Aman	Aman
Tulangan susut	P8 - 200	P8 - 200	---	---

Tabel 4.8 Hasil Perencanaan Pelat Atap Type E

Pelat Atap Type E	
Mu (knm)	2.2518
Mr = Mu/Ø	2.81475
As (mm^2)	232.7009
1,33 As (mm^2)	309.4922
As min (mm^2)	326.6667
As perlu (mm^2)	309.4922
Jrk tul (mm)	162.3304
Tulangan pokok	P8 - 160
As pakai (mm)	314
a	3.546353
Mn	4.086533
Kontrol	Aman
Tulangan Susut	P8 - 200

Tabel 4.9 Hasil Perencanaan Pelat Atap Type F

Pelat Atap Type F	
Mu (knm)	1.56375
Mr = Mu/Ø	1.954688
As (mm^2)	161.5978
1,33 As (mm^2)	214.9251
As min (mm^2)	326.6667
As perlu (mm^2)	214.9251
Jrk tul (mm)	233.7558
Tulangan pokok	P8 - 200
As pakai (mm)	251.2
a	2.837082
Mn	3.290607
Kontrol	Aman
Tulangan Susut	P8 - 200

Tabel 4.10 Hasil Perencanaan Pelat Lantai Type G

	Pelat Lantai Type G			
Perhitungan	Mlx	Mly	Mtx	Mty
Mu (knm)	2.01824	0.5712	3.08448	2.05632
Mr = Mu/Ø	2.5228	0.714	3.8556	2.5704
As (mm^2)	121.6628	34.43287	185.9375	123.9583
1,33 As (mm^2)	161.8115	45.79572	247.2969	164.8646
As min (mm^2)	560	560	560	560
As perlu (mm^2)	161.8115	45.79572	247.2969	164.8646
Jrk tul (mm)	310.4847	1097.046	203.1566	304.7349
Tulangan pokok	P8 - 200	P8 - 200	P8 - 200	P8 – 200
As pakai (mm)	251.2	251.2	251.2	251.2
a	2.837082	2.837082	2.837082	2.837082
Mn	5.702127	5.702127	5.702127	5.702127
Kontrol	Aman	Aman	Aman	Aman
Tulangan Bagi	---	---	P8 - 200	P8 – 200

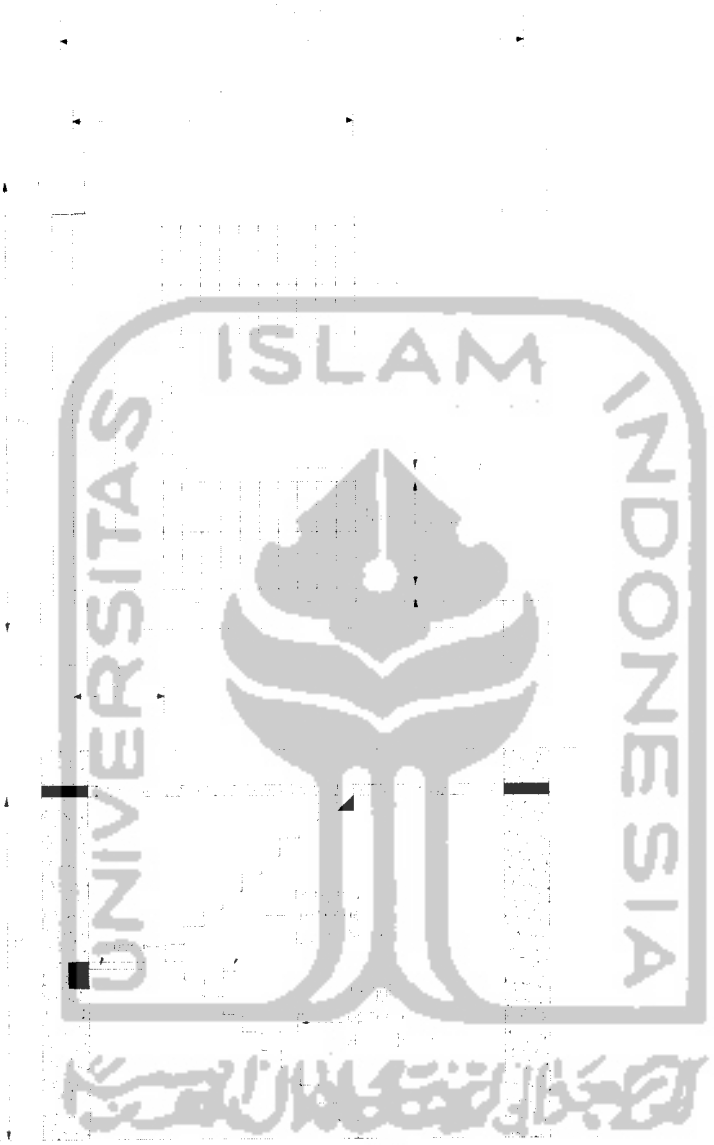
Tabel 4.11 Hasil Perencanaan Pelat Lantai Type H

Pelat Lantai Type H	
Mu (knm)	1.19
Mr = Mu/Ø	1.4875
As (mm^2)	71.73515
1,33 As (mm^2)	95.40774
As min (mm^2)	560
As perlu (mm^2)	95.40774
Jrk tul (mm)	526.582
Tulangan pokok	P8 – 200
As pakai (mm)	251.2
a	2.837082
Mn	5.702127
Kontrol	Aman
Tulangan Bagi	P8 – 200

Tabel 4.12 Hasil Perencanaan Pelat Lantai Type I

	Pelat Lantai Type I			
Perhitungan	Mlx	Mly	Mtx	Mty
Mu (knm)	2.664305	1.1777	4.585308	3.494166
Mr = Mu/Ø	3.330382	1.472125	5.731635	4.367707
As (mm^2)	160.6087	70.99369	276.4099	210.634
1,33 As (mm^2)	213.6095	94.42161	367.6251	280.1432
As min (mm^2)	560	560	560	560
As perlu (mm^2)	213.6095	94.42161	367.6251	280.1432
Jrk tul (mm)	235.1955	532.0816	136.661	179.3368
Tulangan pokok	P8 – 200	P8 – 200	P8 - 130	P8 – 170
As pakai (mm)	251.2	251.2	386.4615	295.5294
a	2.837082	2.837082	4.364742	3.337744
Mn	5.702127	5.702127	8.701657	6.690629
Kontrol	Aman	Aman	Aman	Aman
Tulangan Bagi	---	---	P8 - 200	P8 – 200

4.4 Perencanaan Tangga



Gambar 4.4 Denah tangga lantai basement

1. Data perencanaan tangga :

   a. Tinggi antar lantai = 3,5 m = 350 cm

   b. Lebar bordes = 2,0 m = 200 cm

   c. Lebar tangga = 2,0 m = 200 cm

   d. Beban sandaran tangga :

      Tinggi sandaran = 1 m

      Tebal sandaran = 0,12 m

      Berat sandaran total = 0,12. 24. 2 = 2,88 KN/m^2

   e. Tinggi optrede rencana diambil 16 cm

      Jumlah optrede = 350/16 = 22 buah

      Tinggi optrede pakai = 350/22 = 15,9 cm ≈ 16 cm

      Jumlah antrede = 22 – 2 = 20 buah

      Diambil panjang antrede = 30 cm

   f. Sudut kemiringan tangga = 16/30 = arc tg α ⟶ α = 28,072 °

   g. Dimensi tangga

      Panjang tangga = (panjang antrede x jml antrede/2) + lebar bordes

      = (30. 20/2) + 200 = 500 cm

      Lebar bersih tangga = 200 cm

   h. Tebal pelat bordes diambil 20 cm

2. Pembebanan :

   a. Pembebanan bordes :

      ❖ Beban mati (untuk panjang 1 m) :

      - berat sendiri pelat = 0,2.1. 24 = 4,80 KN/m

- berat spesi = 0,03.1. 24 = 0,72 KN/m
- berat keramik = 0,01.1. 20 = 0,20 KN/m
- berat sandaran = (0,12. 24.1. 1)/3,6 = 0,80 KN/m +

Q_D = 6,52 KN/m

❖ Beban hidup (untuk panjang 1 m) :

Q_L = 300.1 = 3 KN/m

b. Pembebanan tangga :

❖ Beban mati (untuk panjang 1m) :

- berat sendiri tangga = $\left(\frac{0,2}{\cos 28,072}+\frac{0,16}{2}\right)\cdot 1.24$ = 7,76 KN/m
- berat spesi = 0,03.1. 24 = 0,72 KN/m
- berat lantai keramik = 0,01.1. 20 = 0,20 KN/m
- berat sandaran = (0,12. 24.1.1. 2)/2 = 2,88 KN/m +

Q_D = 11,56 KN/m

❖ Beban hidup (untuk panjang 1m) :

Q_L = 300.1 = 3 KN/m

3. Perhitungan perencanaan tulangan pelat tangga

Tinggi pelat = 20 cm = 200 mm

$$\rho_b = \frac{0,85\cdot fc'\cdot\beta_1}{fy}\cdot\left(\frac{600}{600+fy}\right)=\frac{0,85\cdot 25\cdot 0,85}{240}\cdot\left(\frac{600}{600+240}\right)$$

= 0,0537

$d = h_{pelat} - pb - 1/2\ \emptyset_{tul}$

= 200 – 20 – 1/2.12 = 174 mm

Jd = 0,9.174 = 156,6 mm

As max = 0,75.ρ_b. b.d = 0,75. 0,0537.1000.174 = 7015,346 mm^2

As min = 1,4/fy .b.d = 1,4/240.1000.174 = 1015 mm^2

a. Perencanaan tulangan lapangan

Data dari SAP 2000 seperti tercantum pada lampiran, didapat momen maksimum lapangan (M^+) = 26,36 KNm

Mn = Mu/Ø = 26,36 /0,8 = 32,95 KNm

$$As = \frac{Mu/\phi}{Jd.fy} = \frac{32,95 \cdot 10^6}{156,6.240} = 876,703\ mm^2$$

1,33 As = 1,33. 876,703 = 1166,015 mm^2

Check : 1,33 As = 1166,015 mm^2 > As min = 1015 mm^2 ,maka dipakai As = As min = 1015 mm^2 > Ass = 0,002.b.h = 0,002.1000.200 = 400 mm^2

Dipakai Ø tulangan = 12 mm ⟶ $A_1Ø = 1/4.\pi. 12^2 = 113,04\ mm^2$

Jarak tulangan (x) = A_1Ø.1000/As = 113,04.1000/1015

= 111,369 mm

Dipakai jarak tulangan (x) = 110 mm

As ada = A_1Ø.1000/x = 113,04.1000/110 = 1027,636 mm^2

Dipakai tulangan P12 - 110

$$a = \frac{As\ ada \cdot fy}{0,85 \cdot fc' \cdot b} = \frac{1027,636 \cdot 240}{0,85 \cdot 25 \cdot 1000} = 11,606mm$$

Mn_1 = As ada. fy.(d – a/2) = 1027,636.240.(174 – 11,606/2).10^{-6}

= 41,482 KNm >.Mn =.Mu/ Ø = 32,95 KNm ……Ok

b. Perencanaan tulangan tumpuan

Data dari SAP 2000 seperti tercantum pada lampiran, didapat momen maksimum tumpuan (M^-) = 25,93 KNm

Mn = Mu/Ø = 25,93/0,8 = 32,4125 KNm

$$As = \frac{Mu/\phi}{Jd.fy} = \frac{32{,}4125 \cdot 10^6}{156{,}6.240} = 862{,}402\ mm^2$$

1,33 As = 1,33. 862,402 = 1146,994 mm^2

Check : 1,33As = 1146,994 mm^2 > As min = 1015 mm^2 , maka dipakai As min = 1015 mm^2 > Ass = 0,002.b.h = 400 mm^2 .....Ok

Dipakai Ø tulangan = 12 mm ⟶ $A_1Ø = 1/4.\pi. 12^2 = 113{,}04\ mm^2$

Jarak tulangan (x) = $A_1Ø$.1000/As = 113,04.1000/1015

= 111,369 mm

Dipakai jarak tulangan (x) = 110 mm

As ada = $A_1Ø$.1000/x = 113,04.1000/110 = 1027,636 mm^2

Dipakai tulangan P12- 110

$$a = \frac{As\ ada \cdot fy}{0{,}85 \cdot fc' \cdot b} = \frac{1027{,}636 \cdot 240}{0{,}85 \cdot 25 \cdot 1000} = 11{,}606\ mm$$

Mn_1 = As ada. fy.(d – a/2) = 1027,636. 240.(174 – 11,606/2).10^{-6}

= 41,482 KNm >.Mn =.Mu/ Ø = 32,413 KNm .....Ok

c. Perencanaan tulangan tumpuan pada bordes

Data dari SAP 2000 seperti tercantum pada lampiran, didapat momen maksimum tumpuan (M^-) = 25,93 KNm

Mn = Mu/Ø = 25,93/0,8 = 32,4125 KNm

$$As = \frac{Mu/\phi}{Jd.fy} = \frac{32{,}4125 \cdot 10^6}{156{,}6.240} = 862{,}4016\ mm^2$$

1,33 As = 1,33. 862,4016 = 1146,994 mm^2

Check : 1,33As = 1146,994 mm^2 > As min = 1015 mm^2 , maka dipakai As min = 1015 mm^2 > Ass = 0,002.b.h = 400 mm^2 .....Ok

Dipakai Ø tulangan = 12 mm ⟶ $A_1Ø = 1/4.\pi. 12^2 = 113,04\ mm^2$

Jarak tulangan (x) = $A_1Ø.1000/As$ = 113,04.1000/1015

= 111,369 mm

Dipakai jarak tulangan (x) = 110 mm

As ada = $A_1Ø.1000/x$ = 113,04.1000/110 = 1027,636 mm^2

Dipakai tulangan P12 - 110

$$a = \frac{As\ ada \cdot fy}{0,85 \cdot fc' \cdot b} = \frac{1027,636 \cdot 240}{0,85 \cdot 25 \cdot 1000} = 11,606\ mm$$

Mn_1 = As ada. fy.(d – a/2) = $1027,636.240.(174 – 11,606/2).10^{-6}$

= 41,483 KNm >.Mn =.Mu/ Ø = 32,413 KNm .....Ok

4. Perhitungan pondasi tangga

Gambar 4.5 Perencanaan pondasi tangga

a. Data : - σ tanah = 250 KN/m^2

- γ batu = 22 KN/m^3

- γ tanah = 18 KN/m^3

- balok diatas pondasi 40/60

Tinjauan untuk lebar tangga = 2 m

Tinggi pondsi tangga = 1m

b. Pembebanan :

- Akibat beban tangga = 111,85 KN

- Berat balok diatas pondasi = 0,4. 0,6. 2. 24 = 11,52 KN

Diperoleh beban P = 111,89 + 11,52 = 123,41 KN

Tegangan ijin tanah pakai :

σ = σ tanah – berat pondasi

= 250 - 1. 22 = 228 KN/m^2

Diketahui pada kondisi kritis ⟶ σ = P/A

A = P/ σ = 123,41 /228 = 0,541 m^2

B =A/L = 0,541 /2 = 0,271 m ⟶ diambil lebar (B) = 30 cm

Kontrol tegangan tanah :

σ = P/A = 123,41/(0,3. 2) = 205,683 KN/m^2 < σ tanah = 228 KN/m^2 .....Ok

4.5 Perencanaan Balok Induk

1. Data perencanaan balok :

   a. fy = 400 MPa

   b. fc' = 25 MPa

   c. penutup beton (Pb) = 40 mm

   d. diameter tul.pokok = 22 mm

   e. diameter tul.sengkang = 10 mm

   f. lebar balok (b) = 400 mm

   g. tinggi balok (h) = 600 mm

   h. $\theta = 0,8$

   i. $\beta = 0,85$

   j. d' = 40+10+(22/2) = 61 mm

   k. d = 600 – 61 =539 mm

   l. $\rho b = \frac{0,85 \cdot fc' \cdot \beta_1}{fy} \cdot \left(\frac{600}{600 + fy} \right) = \frac{0,85 \cdot 25 \cdot 0,85}{400} \cdot \left(\frac{600}{600 + 400} \right) = 0,0271$

   m. $\rho \max = 0,75 \cdot \rho b = 0,75 \cdot 0,0271 = 0,0203$

   n. $\rho \min = \frac{1,4}{fy} = \frac{1,4}{400} = 0,0035$

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

   p. $\rho_{pakai} = 0,5.\ \rho_{maks} = 0,5.0,0203 = 0,0102$

   q. $Rn = \rho_{pakai}.fy.(1-0,5.\ \rho_{pakai}.\ m)$

   $= 0,0102.\ 400.(1-0,5.\ 0,0102.\ 18,824) = 3,624$ Mpa

2. Perencanaan dimensi balok portal A lantai dasar (GF)

   a. Perencanaan tulangan tumpuan

      Data dari SAP 2000 seperti tercantum pada lampiran, didapat momen maksimum tumpuan (M') = 254,577 KNm

      Mu/O = 254,577 /0,8 = 318,221 KNm

$$bd^2 = \frac{Mn}{Rn} = \frac{318{,}221 \cdot 10^6}{3{,}624} = 87{,}809.10^6$$

      Dicoba ukuran b = 400 mm dan h = 600 mm

$$d_{perlu} = \sqrt{\frac{bd^2}{b}} = \sqrt{\frac{87{,}809 \cdot 10^6}{400}} = 468{,}532\ mm$$

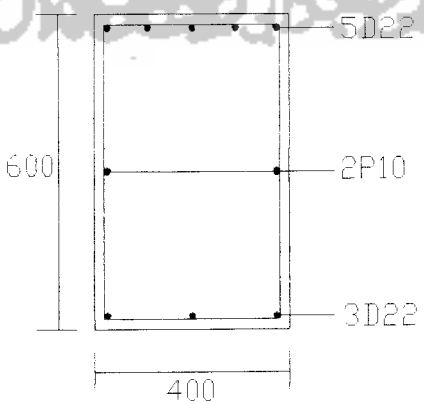
      Check : d_{perlu} = 468,532 mm < d = 539 mm, dipakai tulangan sebelah

$$Rn_{baru} = \frac{Mn}{b \cdot d^2} = \frac{318{,}221 \cdot 10^6}{400 \cdot 539^2} = 2{,}738\ MPa < Rn_{lama} = 3{,}624\ \text{MPa}$$

$$\rho_{baru} = \rho_{lama} \cdot \frac{Rn_{baru}}{Rn_{lama}} = 0{,}0102 \cdot \frac{2{,}738}{3{,}624} = 0{,}008$$

      $As_{perlu} = \rho_{baru}.b.d = 0{,}008.400.\ 539 = 1724{,}8\ mm^2$

      Dipakai 5D22 ⟶ $As_{ada} = 5.\ (1/4).\ \pi.\ 22^2 = 1900\ mm^2$



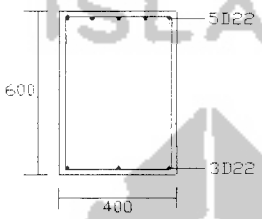
Gambar 4.6 Detail potongan balok induk pada tumpuan

Periksa penempatan tulangan :

$$Ybd = \frac{b - 2\cdot(pb + \theta_{sengkang}) - (n\cdot\theta_{pokok})}{n-1} = \frac{400 - 2\cdot(40+10) - (5\cdot 22)}{5-1}$$

= 47,5 mm > D22 = 22 mm .....Ok

❖ Momen Nominal Aktual Balok Negatif (Mnak,b$^{(-)}$) :



Tulangan atas = 5D22 dengan As_{ada} = 1900 mm^2

Tulangan bawah = 3D22 dengan As'_{ada} = 1140 mm^2

$$\rho = \frac{As_{ada}}{b\cdot d_{pakai}} = \frac{1900}{400\cdot 539} = 0{,}0088 \;;\; \rho' = \frac{As'_{ada}}{b\cdot d_{pakai}} = \frac{1140}{400\cdot 539} = 0{,}00528$$

$$fs' = 600\cdot\left\{1 - \frac{0{,}85\cdot fc'\cdot\beta_1\cdot d'}{(\rho - \rho')\cdot fy\cdot d}\right\} = 600\cdot\left\{1 - \frac{0{,}85\cdot 25\cdot 0{,}85\cdot 61}{(0{,}0088 - 0{,}00528)\cdot 400\cdot 539}\right\}$$

$= 271{,}099\ MPa$

fs' < fy dipakai fs' = 271,099 MPa

$$a = \frac{(As_{ada}\cdot fy) - (As'_{ada}\cdot fs')}{0{,}85\cdot fc'\cdot b} = \frac{(1900\cdot 400) - (1140\cdot 271{,}099)}{0{,}85\cdot 25\cdot 400} = 53{,}053\ mm$$

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

$= (1900 \,.\, 400 - 1140 \,.\, 271{,}099) \,.\, (539 - 53{,}053/2) \,.\, 10^{-6}$

= 231,098 KNm

$Mn_2 = (As'_{ada} \,.\, fs') \,.(d\text{-}d')$

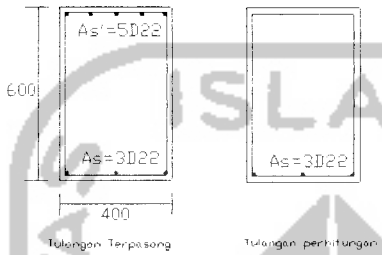
$= (1140 \,.\, 271{,}099) \,.\, (539\text{-}61) \,.10^{-6}$ = 147,727 KNm

$Mnak,b^{(-)} = Mn_1 + Mn_2$

$= 231,098 + 147,727 = 378,825$ KNm > Mu/O = 318,221 KNm …Ok

$Mkap,b^{(-)} = 1,25 \cdot 378,825 = 473,531$ KNm

❖ Momen Nominal Aktual Balok Positif ($Mnak,b^{(+)}$) :



As = As' = 3D22 =1140 mm^2

$$\rho_{aktual} = \frac{As'_{ada}}{b \cdot d} = \frac{1140}{400 \cdot 539} = 0,00528$$

Rn = ρ .fy .(1-0,5 .m .ρ)

$= 0,00528 .400 .(1-0,5 .18,824 .0,00528) = 2$ MPa

$Mnak,b^{(+)} = Rn .b .d^2$

$= 2 .400 .539^2 .10^{-6} = 233,55$ KNm

$Mkap,b^{(+)} = 1,25 .233,55 = 291,9375$ KNm

b. Perencanaan tulangan lapangan

Data dari SAP 2000 seperti tercantum pada lampiran, didapat momen maksimum lapangan (M^+) = 236,115 KNm

Mn = Mu/O = 236,115/0,8 = 295,144 KNm

$$bd^2 = \frac{Mn}{Rn} = \frac{295,144 \cdot 10^6}{3,624} = 81,442 \cdot 10^6$$

Dicoba ukuran b = 400 mm dan h = 600 mm

$$d_{perlu} = \sqrt{\frac{bd^2}{b}} = \sqrt{\frac{81{,}442 \cdot 10^6}{400}} = 451{,}226\ mm$$

Check : d_{perlu} = 451,226 mm < d = 539 mm, dipakai tulangan sebelah

$$Rn_{baru} = \frac{Mn}{b \cdot d^2} = \frac{295{,}144 \cdot 10^6}{400 \cdot 539^2} = 2{,}54\ MPa < Rn_{lama} = 3{,}624\ \text{MPa}$$

$$\rho_{baru} = \rho_{lama} \cdot \frac{Rn_{baru}}{Rn_{lama}} = 0{,}0102 \cdot \frac{2{,}54}{3{,}624} = 0{,}007$$

As_{perlu} = ρ.b.d = 0,007.400. 539 = 1509,2 mm^2

Dipakai 4D22 ⟶ As_{ada} = 4. (1/4). π. 22^2 = 1520,531 mm^2

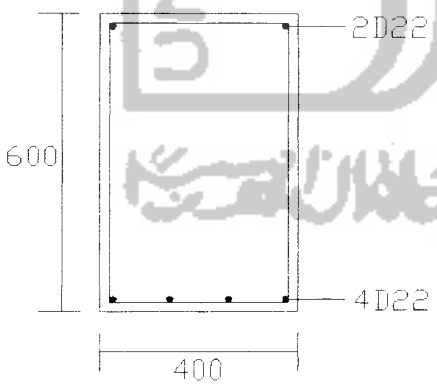
Periksa kapasitas penampang :

$$a = \frac{As \cdot fy}{0{,}85 \cdot fc' \cdot b} = \frac{1520{,}531 \cdot 400}{0{,}85 \cdot 25 \cdot 400} = 71{,}554\ \text{mm}$$

Mn = As.fy.(d-a/2)

= 1520,531. 400.(539-(71,554/2)).10^{-6}

= 306,066 KNm > Mn = Mu/Ø = 295,144 KNm .....Ok



Gambar 4.7 Detail potongan balok induk pada lapangan

Periksa penempatan tulangan :

$$Jbd = \frac{b - 2 \cdot (pb + \theta_{sengkang}) - (n \cdot \theta_{pokok})}{n - 1} = \frac{400 - 2 \cdot (40 + 10) - (4 \cdot 22)}{4 - 1}$$

= 70,667 mm > D22 = 22 mm .....Ok

$$Rn_{baru} = \frac{Mn}{b \cdot d^2} = \frac{295{,}144 \cdot 10^6}{400 \cdot 539^2} = 2{,}54\ MPa < \text{Rn}_{\text{lama}} = 3{,}624\ \text{MPa}$$

$$\rho_{baru} = \rho_{lama} \cdot \frac{Rn_{baru}}{Rn_{lama}} = 0{,}0102 \cdot \frac{2{,}54}{3{,}624} = 0{,}007$$

$\text{As}_{\text{perlu}} = \rho.b.d = 0{,}007.400.\ 539 = 1509{,}2\ \text{mm}^2$

Dipakai 4D22 ⟶ $\text{As}_{\text{ada}} = 4.\ (1/4).\ \pi.\ 22^2 = 1520{,}531\ \text{mm}^2$

Periksa kapasitas penampang :

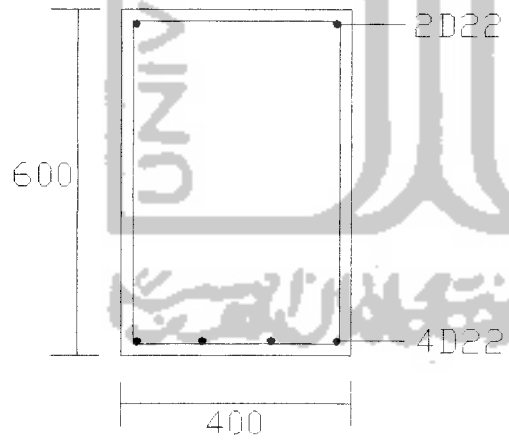
$$a = \frac{As \cdot fy}{0{,}85 \cdot fc' \cdot b} = \frac{1520{,}531 \cdot 400}{0{,}85 \cdot 25 \cdot 400}$$

$= 71{,}554$ mm

$Mn = As.fy.(d\text{-}a/2)$

$= 1520{,}531.\ 400.(539\text{-}(71{,}554/2)).10^{-6}$

$= 306{,}066$ KNm > $Mn = Mu/\emptyset = 295{,}144$ KNm ……..Ok



Gambar 4.7 Detail potongan balok induk pada lapangan

Periksa penempatan tulangan :

$$Jbd = \frac{b - 2 \cdot (pb + \theta_{sengkang}) - (n \cdot \theta_{pokok})}{n - 1} = \frac{400 - 2 \cdot (40 + 10) - (4 \cdot 22)}{4 - 1}$$

$= 70{,}667$ mm > D22 = 22 mm ……Ok

c. Perencanaan Gaya Geser

Data dari SAP 2000 seperti tercantum pada lampiran, didapat gaya geser maksimum sebagai berikut :

VD = 141,431 KN

VL = 34,031 KN

VE = 21,912 KN

$$Vu_1b = 0,7 \cdot \left(\frac{Mkap,b^{(+)} + Mkap,b^{(-)}}{Ln} \right) + 1,05 \cdot Vg \quad ; Vg = VD + VL$$

$$= 0,7 \cdot \left(\frac{291,9375 + 473,531}{6,5} \right) + 1,05 \cdot (141,431 + 34,031)$$

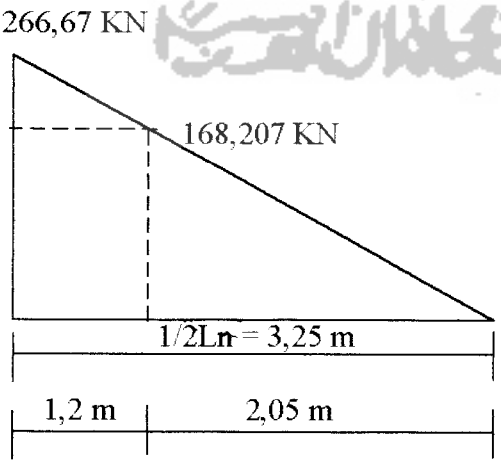
$$= 266,67\ KN$$

$$Vu_2b = 1,05 \cdot \left(VD + VL + \left({}^{4}/_{K} \right) \cdot VE \right)$$

$$= 1,05 \cdot \left(141,431 + 34,031 + \left({}^{4}/_{1} \right) \cdot 21,912 \right)$$

$$= 276,266\ KN$$

Vu pakai adalah nilai terkecil antara Vu_1b dan Vu_2b, sehingga didapat Vu pakai = 266,67 KN



$$\frac{266,67}{X} = \frac{3,25}{2,05}$$

X = 168,207 KN

Gambar 4.8 Diagram segitiga tegangan geser

Daerah sendi plastis (sepanjang 2h) :

Vc = 0 (beton dianggap tidak menerima geser)

$$Vs = \frac{Vub\ pakai}{\phi} = \frac{266,67}{0,6} = 444,45\ KN$$

Tersedia tul.sengkang P_{10} mm ⟶ fy = 240 Mpa

$$S = \frac{4 \cdot A_V \cdot fy \cdot d}{Vs} = \frac{4 \cdot 0,25 \cdot \pi \cdot 10^2 \cdot 240 \cdot 539}{444,45 \cdot 10^3}$$

= 91,43 mm < d/4 = 539/4 = 134,75 mm, maka S pakai = 90 mm

Dipakai tulangan $P_{10\text{-}90}$

Periksa :

Vub/Ø < Vc + Vs

266,67 /0,8 < 0 + 444,45

333,33 KN < 444,45 KN .....Ok

Daerah luar sendi plastis :

Vub diluar daerah sendi plastis diperoleh = 174,107 KN

$$Vc = \frac{1}{6} \cdot \sqrt{fc'} \cdot b \cdot d = \frac{1}{6} \cdot \sqrt{25} \cdot 400 \cdot 539 \cdot 10^{-3} = 179,667\ KN$$

$$Vs = \frac{Vub\ pakai}{0,6} \quad Vc = \frac{168,207}{0,6} - 179,667 = 100,678\ KN$$

$$S = \frac{4 \cdot Av \cdot fy \cdot d}{Vs} = \frac{4 \cdot 0,25 \cdot \pi \cdot 10^2 \cdot 240 \cdot 539}{100,678 \cdot 10^3}$$

= 403,659 mm > d/2 = 539/2 = 269,5 mm, maka S pakai = 265 mm

Dipakai tulangan $P_{10\text{-}265}$

Periksa :

Vub/O < Vc + Vs

168,207 /0,8 < 179,667 + 100,678

210,258 KN < 280,345 KN .....Ok

4.6 Perencanaan Kolom

1. Data :

   a. Ukuran kolom :

      b = 700 mm

      h = 700 mm

   b. Ukuran balok :

      b = 400 mm

      h = 600 mm

   c. r = 0,3.h = 0,3.0,7 = 0,21 m

   d. fc' = 25 MPa

   e. Fy deform = 400 MPa

   f. Fy polos = 240 MPa

   g. $Ec = Eg = 4700 \cdot \sqrt{fc'} = 4700 \cdot \sqrt{25} = 23500\ MPa = 23500000\ KN/m^2$

   h. Ic (Inersia kolom) = $1/12.b_k.h_k^3 = 1/12.\ 0,7.\ 0,7^3 = 0,02\ m^4$

   i. Ig (Inersia balok) = $1/12.b_b.h_b^3 = 1/12.\ 0,4.\ 0,6^3 = 0,0072\ m^4$

   j. Tinggi kolom = 4,5 m

   k. Tinggi (h) pondasi = 1,8 m

   l. Panjang balok (Lb_1) = 7,2 m

   m. Lb_1' (bentang bersih balok) = Lb_1-b kolom = 7,2 – 0,7 = 6,5 m

   n. Lb_2 = 7,2 m

   o. Lb_2' = 6,5 m

   p. Cm = 1 (unbraced frame)

   q. Ø = 0,65

   r. Øo = 1,25

s. d' = 61 mm

t. d = h-d' = 700-61 = 639 mm

2. Perencanaan grafik Mn – Pn

a. Batang desak aksial

Ag = b.h

= 700. 700 = 490000 mm^2

Ast = 1%.Ag

= 1%. 490000 = 4900 mm^2

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

= (0,85. 25. (490000-4900) + 4900. 400).10^{-3}

= 12268,375 KN = 1226,8375 T

Pno = 0,8.Po

= 0,8. 12268,375

= 9814,7 KN = 981,47 T

b. Batang desak dan momen

1) Keadaan seimbang

$$xb = \frac{600}{600 + fy} \cdot d = \frac{600}{600 + 400} \cdot 639 = 383{,}4\ mm$$

$$fs' = \frac{xb - d'}{xb} \cdot 600 = \frac{383{,}4 - 61}{383{,}4} \cdot 600$$

= 504,5383 MPa ≥ fy = 400 MPa

fs' pakai = fy = 400 MPa

a = β_1. xb = 0,85. 383,4 = 325,89 mm

Cc = 0,85. fc'. b. a. 10^{-3} = 0,85. 25. 700. 325,89. 10^{-3}

= 4847,6138 KN = 484,7614 T

As = As' = 1/2. Ast = 1/2. 4900 = 2450 mm^2

Cs = As'. (fs'- 0,85. fc'). 10^{-3} = 2450. (400 – 0,85. 25). 10^{-3}

= 927,9375 KN = 92,7938 T

Ts = As. fy. 10^{-3} = 2450. 400. 10^{-3}

= 980 KN = 98 T

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

= (4847,6138.(700/2 –325,89 /2) + 927,9375. (700/2 –61) + 980.

(639 – 700/2)). 10^{-3}

= 1458,1643 KNm = 145,8164 Tm

Pnb = Cc + Cs – Ts = 4847,6138 + 927,9375 – 980

= 4795,5513 KN = 479,5551 T

$$eb = \frac{Mnb}{Pnb} = \frac{1458,1643}{4795,5513} = 0,3041\ m$$

2) Patah desak

- x = 190 %. xb = 1,9. 383,4

= 728,46 mm > xb = 383,4 mm

$$fs' = \frac{x - d'}{x} \cdot 600 = \frac{728,46 - 61}{728,46} \cdot 600$$

= 549,757 MPa ≥ fy = 400 MPa

fs' pakai = fy = 400 MPa

$$fs = \frac{d - x}{x} \cdot 600 = \frac{639 - 728,46}{728,46} \cdot 600$$

= -73,6842 MPa < fy = 400 MPa

fs pakai = -73,6842 MPa

$a = \beta_1 . x = 0,85 . 728,46 = 619,191$ mm

$Cc = 0,85 . fc' . b . a . 10^{-3} = 0,85 . 25 . 700 . 619,191 . 10^{-3}$

$= 9210,4661$ KN $= 921,0466$ T

$Cs = As'.(fs' - 0,85.fc').10^{-3} = 2450.(400 - 0,85 . 25).10^{-3}$

$= 927,9375$ KN $= 92,7938$ T

$Ts = As . fs . 10^{-3} = 2450.(-73,6842).10^{-3} = -180,5263$ KN

$Pn = Cc + Cs - Ts$

$= 9210,4661 + 927,9375 + 180,5263 = 10318,9299$ KN

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

$= (9210,4661.(700/2 - 619,191/2) + 927,9375.(700/2 - 61) -$

$180,5263 .(639 - 700/2)).10^{-3}$

$= 588,15$ KNm $= 58,815$ Tm

$$e = \frac{Mn}{Pn} = \frac{588,15}{10318,9299} = 0,057\ m$$

- x = 150 %. xb = 1,5. 383,4

$= 575,1$ mm > xb = 383,4 mm

$$fs' = \frac{x - d'}{x} \cdot 600 = \frac{575,1 - 61}{575,1} \cdot 600$$

$= 536,3589$ MPa > fy = 400 MPa

fs' pakai = fy = 400 MPa

$$fs = \frac{d - x}{x} \cdot 600 = \frac{639 - 575,1}{575,1} \cdot 600$$

$= 66,6667$ MPa $\leq$ fy = 400 MPa

fs pakai = 66,6667 MPa

$a = \beta_1 . x = 0,85 . 575,1 = 488,835$ mm

$Cc = 0,85 . fc' . b . a . 10^{-3} = 0,85 . 25 . 700 . 488,835 . 10^{-3}$

$= 7271,4206$ KN = 727,1421 T

$Cs = As'.(fs' - 0,85.fc').10^{-3} = 2450.(400 - 0,85 . 25).10^{-3}$

$= 927,9375$ KN = 92,7938 T

$Ts = As . fs . 10^{-3} = 2450.(66,6667).10^{-3} = 163,3334$ KN

Pn = Cc + Cs – Ts

$= 7271,4206 + 927,9375 - 163,3334 = 8036,0247$ KN

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

$= 7271,4206 . (700/2 - 488,835/2) + 927,9375.(700/2 - 61) + 163,3334.(639 - 700/2).10^{-3}$

$= 1083,1121$ KNm = 108,3112 Tm

$$e = \frac{Mn}{Pn} = \frac{1083,1121}{8036,0247} = 0,1345\ m$$

- x = 125 %. xb = 1,25. 383,4

  = 479,25 mm > xb = 383,4 mm

$$fs' = \frac{x - d'}{x} \cdot 600 = \frac{479,25 - 61}{479,25} \cdot 600$$

= 523,6307 MPa > fy = 400 MPa

fs' pakai = fy = 400 MPa

$$fs = \frac{d - x}{x} \cdot 600 = \frac{639 - 479,25}{479,25} \cdot 600$$

= 200 MPa ≤ fy = 400 MPa

fs pakai = 200 MPa

$a = \beta_1 . x = 0,85 . 479,25 = 407,3625$ mm

$Cc = 0,85 . fc' . b . a . 10^{-3} = 0,85 . 25 . 700 . 407,3625 . 10^{-3}$

$= 6059,5172$ KN $= 605,9517$ T

$Cs = As'.(fs' - 0,85.fc').10^{-3} = 2450.(400 - 0,85 . 25).10^{-3}$

$= 927,9375$ KN $= 92,7938$ T

$Ts = As . fs . 10^{-3} = 2450.(200).10^{-3} = 490$ KN

$Pn = Cc + Cs - Ts$

$= 6059,5172 + 927,9375 - 490 = 6497,4547$ KN

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

$= 6059,5172.(700/2 - 407,3625/2) + 927,9375.(700/2 - 61) + 490.(639 - 700/2).10^{-3}$

$= 1296,4049$ KNm $= 129,6405$ Tm

$$e = \frac{Mn}{Pn} = \frac{1296,4049}{6497,4547} = 0,1995\ m$$

3) Patah tarik

- $x = 75\ \% . xb = 0,75 . 383,4$

$= 287,55$ mm $< xb = 383,4$ mm

$$fs' = \frac{x - d'}{x} \cdot 600 = \frac{287,55 - 61}{287,55} \cdot 600$$

$= 472,7178$ MPa $>$ fy $= 400$ MPa

fs' pakai = fy = 400 MPa

$$fs = \frac{d - x}{x} \cdot 600 = \frac{639 - 287,55}{287,55} \cdot 600$$

= 733,3333 MPa ≥ fy = 400 MPa

fs pakai = 400 MPa

$a = \beta_1 . x = 0,85 . 287,55 = 244,4175$ mm

$Cc = 0,85 . fc' . b . a . 10^{-3} = 0,85 . 25 . 700 . 244,4175 . 10^{-3}$

= 3635,7103 KN = 363,571 T

$Cs = As' . (fs' - 0,85 . fc') . 10^{-3} = 2450 . (400 - 0,85 . 25) . 10^{-3}$

= 927,9375 KN = 92,7938 T

$Ts = As . fs . 10^{-3} = 2450 . 400 . 10^{-3} = 980$ KN

Pn = Cc + Cs – Ts

= 3635,7103 + 927,9375 -980 = 3583,6478 KN

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

= 3635,7103.(700/2 - 244,4175/2) + 927,9375.(700/2 - 61) + 980.(639-700/2).10^{-3}

= 1379,5769 KNm = 137,9577 Tm

$$e = \frac{Mn}{Pn} = \frac{1379,5769}{3583,6478} = 0,385\ m$$

- x = 60 %. xb = 0,6. 383,4

= 230,04 mm < xb = 383,4 mm

$$fs' = \frac{x - d'}{x} \cdot 600 = \frac{230,04 - 61}{230,04} \cdot 600$$

= 440,8972 MPa > fy = 400 MPa

fs' pakai = fy = 400 MPa

$$fs = \frac{d - x}{x} \cdot 600 = \frac{639 - 230,04}{230,04} \cdot 600$$

= 1066,6667 MPa ≥ fy = 400 MPa

fs pakai = 400 MPa

$a = \beta_1 . x = 0,85 . 230,04 = 195,534$ mm

$Cc = 0,85 . fc' . b . a . 10^{-3} = 0,85 . 25 . 700 . 195,534 . 10^{-3}$

= 2908,5683 KN = 290,8568 T

$Cs = As' . (fs' - 0,85 . fc') . 10^{-3} = 2450 . (400 - 0,85 . 25) . 10^{-3}$

= 927,9375 KN = 92,7938 T

$Ts = As . fs . 10^{-3} = 2450 . 400 . 10^{-3} = 980$ KN

Pn = Cc + Cs – Ts

= 2908,5683 + 927,9375- 980= 2856,5058 KN

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

= 2908,5683.(700/2-195,534/2) + 927,9375.(700/2- 61) + 980.(639-700/2).10^{-3}

= 1285,0308 KNm = 128,5031 Tm

$$e = \frac{Mn}{Pn} = \frac{1285,0308}{2856,5058} = 0,4499\ m$$

- x = 50 %. xb = 0,5. 383,4

= 191,7 mm < xb = 383,4 mm

$$fs' = \frac{x - d'}{x} \cdot 600 = \frac{191,7 - 61}{191,7} \cdot 600$$

= 409,0767 MPa > fy = 400 MPa

fs' pakai = fy = 400 MPa

$$fs = \frac{d - x}{x} \cdot 600 = \frac{639 - 191,7}{191,7} \cdot 600$$

= 1400 MPa ≥ fy = 400 MPa

fs pakai = 400 MPa

$a = \beta_1 . x = 0,85 . 191,7 = 162,945$ mm

$Cc = 0,85 . fc'.b . a . 10^{-3} = 0,85 . 25 . 700 . 162,945 . 10^{-3}$

= 2423,8069 KN = 242,3807 T

$Cs = As'.(fs' - 0,85.fc').10^{-3} = 2450.(400 - 0,85 . 25).10^{-3}$

= 927,9375 KN = 92,7938 T

$Ts = As . fs . 10^{-3} = 2450 . 400 . 10^{-3} = 980$ KN

Pn = Cc + Cs – Ts

= 2423,8069 + 927,9375 -980 = 2371,7444 KN

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

= 2423,8069.(700/2 – 162,945/2) + 927,9375.(700/2 - 61) + 980.(639 -700/2).10^{-3}

= 1202,2527 KNm = 120,2253 Tm

$$e = \frac{Mn}{Pn} = \frac{1202,2527}{2371,7444} = 0,5069\ m$$

c. Lentur murni

$$a = \frac{As \cdot fy}{0,85 \cdot fc \cdot b} = \frac{2450 \cdot 400}{0,85 \cdot 25 \cdot 700} = 65,8824\ mm$$

$Mn = As . fy.(d – a/2).10^{-6} = 2450 . 400.(639 – 65,8824/2).10^{-6}$

= 593,9376 KNm = 59,3938 Tm

3. Perencanaan kolom tengah portal B-A elemen KA28

   a. Data momen diperoleh dari SAP 2000 seperti tercantum pada lampiran :

   MD_A = 56,242 KNm MD_B = 53,906 KNm

   ML_A = 20,43 KNm ML_B = 16,08 KNm

   ME_A = 119,514 KNm ME_B = 139,24 KNm

   Mu_A = 198,284 KNm Mu_B = 214,438 KNm

   b. Data gaya aksial diperoleh dari SAP 2000 seperti tercantum pada lampiran :

   PD_A = 1968,199 KN PD_B = 2021,119 KN

   PL_A = 381,926 KN PL_B = 381,926 KN

   PE_A = 103,155 KN PE_B =103,155 KN

   Pu_A = 2972,92 KN Pu_B = 3036,424 KN

   c. Data gaya geser diperoleh dari SAP 2000 seperti tercantum pada lampiran :

   VD_A = 8,578 KN VD_B = 9,785 KN

   VL_A = 15,272 KN VL_B = 15,272 KN

   VE_A = 35,378 KN VE_B = 35,378 KN

d. Perhitungan momen rencana (Mc)

Gambar 4.9 Elemen kolom

e = Mu/Pu

= 198.284 /2972.92 = 0,0667 m

e_{min} = (1,5 + 0,03.h) cm = (1,5 + 0,03.70) cm = 3,6 cm = 0,036 m

= 0,036 m < 0,0667 m, maka dipakai e = 0,0667 m

$$\psi_A = \frac{\sum\left(\frac{Ec \cdot Ic}{Lc}\right)}{\sum\left(\frac{Eg \cdot Ig}{Lg}\right)} = \frac{\sum\left(\frac{23500000 \cdot 0,02}{4,5}\right) + \left(\frac{23500000 \cdot 0,02}{4,5}\right)}{\sum\left(\frac{23500000 \cdot 0,0072}{6,5}\right) + \left(\frac{23500000 \cdot 0,0072}{6,5}\right)}$$

= 4,014

$$\psi_B = \frac{\sum\left(\frac{Ec \cdot Ic}{Lc}\right)}{\sum\left(\frac{Eg \cdot Ig}{Lg}\right)} = \frac{\sum\left(\frac{23500000 \cdot 0,02}{4,5}\right) + \left(\frac{23500000 \cdot 0,02}{5}\right)}{\sum\left(\frac{23500000 \cdot 0,0072}{6,5}\right) + \left(\frac{23500000 \cdot 0,0072}{6,5}\right)}$$

= 3,813

Lihat nomogram unbranced frames, didapat k = 2

$$\frac{k \cdot l}{r} = \frac{2 \cdot (4{,}5 - 0{,}6)}{0{,}21}$$

= 37,143 > 22 dan ≤ 100, maka termasuk kolom panjang, sehingga dipakai konsep perbesaran momen :

$$EI_1 = \frac{\frac{1}{5} \cdot (Ec \cdot Ig) + Es \cdot Ise}{1 + \beta d}$$

$$= \frac{\frac{1}{5} \cdot (23500000 \cdot 0{,}0072) + (2.10^8 \cdot 0{,}025 \cdot 0{,}7 \cdot 0{,}7 \cdot 0{,}289^2)}{\left(1 + \frac{2361{,}8388}{2972{,}9204}\right)}$$

$$= 132891{,}054\ KNm^2$$

$$EI_2 = \frac{0{,}4 \cdot Ec \cdot Ig}{1 + \beta d} = \frac{0{,}4 \cdot 23500000 \cdot 0{,}0072}{\left(1 + \frac{2361{,}8388}{2972{,}9204}\right)}$$

$$= 37716{,}276\ KNm^2$$

dipakai EI = 132891,054 KNm^2

$$Pc = \frac{\pi^2 \cdot EI}{(k \cdot l)^2} = \frac{\pi^2 \cdot 132891{,}054}{(2 \cdot 3{,}9)^2} = 21557{,}993\ KN$$

$$\delta b = \frac{Cm}{1 - \left(\frac{Pu}{\phi Pc}\right)} \geq 1$$

$$= \frac{1}{1 - \left(\frac{2972{,}9204}{0{,}65 \cdot 21557{,}993}\right)} \geq 1$$

$$= 1{,}269$$

ΣPu = 2407,796+2848,042+2791,534+2972,92+2919,828+2298,98

= 16239,1 KN

ΣPc =126427,604+22002,37+21743,841+21557,993+21977,94+127512,32

= 341222,065 KN

$$\delta s = \frac{1}{1-\left(\frac{\sum Pu}{\phi \cdot \sum Pc}\right)} \geq 1$$

$$= \frac{1}{1-\left(\frac{16239,1}{0,65 \cdot 341222,065}\right)} \geq 1$$

= 1,079

Mu_1 = 1,2. MD + 1,6.ML = 1,2. 56,242 + 1,6. 20,43 = 100,178 KNm

Mu_2 = ME = 119,514 KNm

Mc = δb.Mu_1 + δs.Mu_2

= 1,269. 100,178 + 1,097. 119,514 = 258,233 KNm

e. Perencanaan tulangan kolom

Kolom tengah portal B-A elemen KA28

hk = 4,5 m

hk' = 4,5 – h balok

= 4,5 – 0,6 = 3,9 m

Rv = 1

$Mkap_{kiri}$ = 292,038 KNm

$Mkap_{kanan}$ = 473,711 KNm

Nuk_1 = 0,7. Rv. ($Mkap_{kiri}$ + $Mkap_{kanan}$)/hk + 1,05. Ng

= 0,7. 1.((-292,038) + 473,711)/4,5 + 1,05. (1968,199 + 381,926)

= 2495,891 KN

Nuk_2 = 1,05. (P_D + P_L + (4/K). P_E)

= 1,05. (1968,199 + 381,926 + (4/1). 103,155)

= 2900,882 KN

dipakai Nuk minimum, yaitu Nuk_1 = 2495,891 KN

$Mmaks_{atas}$ = 198,284 KNm

$Mmaks_{bawah}$ = 214,438 KNm

$$\alpha_k = \frac{Mmaks_{atas}}{Mmaks_{atas} + Mmaks_{bawah}} = \frac{198,284}{198,284 + 214,438}$$

= 0,48

ω_d = 1,3

$$Muk_1 = \frac{hk}{hk'} \cdot 0,7 \cdot \omega_d \cdot \alpha_k \cdot \left(\frac{l_{ki}}{l_{nki}} \cdot (\Sigma Mkap\ bx + 0,3 \cdot \Sigma Mkap\ by) \right)$$

$$= \frac{4,5}{3,9} \cdot 0,7 \cdot 1,3 \cdot 0,48 \cdot \left(\frac{7,2}{6,5} \cdot ((473,711 + 292,038) + 0,3 \cdot (564,33 + 292,038)) \right)$$

$$= 570,927\ KNm$$

$$Muk_2 = \frac{hk}{hk'} \cdot 0,7 \cdot \omega_d \cdot \alpha_k \cdot \left(\frac{l_{ki}}{l_{nki}} \cdot (0,3 \cdot \Sigma Mkap\ bx + \Sigma Mkap\ by) \right)$$

$$= \frac{4,5}{3,9} \cdot 0,7 \cdot 1,3 \cdot 0,48 \cdot \left(\frac{7,2}{6,5} \cdot (0,3 \cdot (473,711 + 292,038) + (564,33 + 292,038)) \right)$$

$$= 606,34\ KNm$$

$Muk_3 = 1,05.(M_{Dk} + M_{Lk} + (4/K).M_{Ek})$

= 1,05. (56,242 +20,43 + (4/1).119,514)

= 582,464 KNm

Dipakai Muk minimum, yaitu Muk_3 = 570,927 KNm

Bandingkan Mc = 258,233 KNm dan Muk pakai = 570,927 KNm, sehingga dipakai yang terbesar, yaitu Mu terpakai = 570,927 KNm

Mu/Ø = 570,927 /0,65 = 878,349 KNm = 87,8349 Tm

Nu/Ø = 2495,891 /0,65 = 3839,832 KN = 383,983 T

$$e = \frac{Mu/\phi}{Nu/\phi} = \frac{87,8349}{383,983} = 0,2287\ m$$

Dari grafik Pn-Mn, didapat luas tulangan total (Ast) :

Ast ≤ 1%. b. h = 1%. 700. 700 = 4900 mm^2 ⟶ diambil Ast = 1 %. b. h

As perlu = Ast/2

= 4900/2 = 2450 mm^2

dipakai 7D22 ⟶ As ada = 2660,929 mm^2 > As perlu .....Ok

As = As' = 2660,929 mm^2

Keadaan seimbang :

$$cb = \frac{600}{600 + fy} \cdot d = \frac{600}{600 + 400} \cdot 639 = 383,4\ mm$$

$$fs' = \frac{cb - d'}{cb} \cdot 600 = \frac{383,4 - 61}{383,4} \cdot 600 = 504,538\ MPa > \text{fy} = 400\ \text{MPa}$$

dipakai fs' = fy = 400 MPa

Cc = 0,85. fc'.b. β. cb = (0,85. 25. 700. 0,85. 383,4). 10^{-3} = 4847,614 KN

Cs = As'. (fs'-0,85. fc') = 2660,929.(400-0,85. 25).10^{-3} = 1007,827 KN

Ts = As. fy = (2660,929. 400).10^{-3} = 1064,372 KN

Pnb = Cc + Cs – Ts

= 4847,614 + 1007,827 - 1064,372 = 4791,069 KN = 479,107 T

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

= (4847,614.((700/2)-((0,85.383,4)/2)) + 1007,827((700/2)-61) +

1064,372. (639-(700/2))).10^{-3}

= 1505,636 KNm = 150,564 Tm

Nu / Ø = 3839,832 KN < Pnb = 4791,107 KN

Terjadi patah tarik :

Kontrol patah tarik dengan rumus Whitney :

$$Pn = 0{,}85 \cdot fc' \cdot b \cdot d \cdot \left[\frac{h - 2 \cdot e}{2 \cdot d} + \sqrt{\left(\left(\frac{h - 2 \cdot e}{2 \cdot d} \right)^2 + 2 \cdot m \cdot \rho_b \cdot \left(1 - \frac{d}{d'} \right) \right)} \right]$$

$$= 0{,}85 \cdot 25 \cdot 10^3 \cdot 700 \cdot 10^{-3} \cdot 639 \cdot 10^{-3} \cdot \{ \frac{700 \cdot 10^{-3} - 2 \cdot 0{,}2314 \cdot 10^{-3}}{2 \cdot 639 \cdot 10^{-3}} +$$

$$\sqrt{\left(\frac{700 \cdot 10^{-3} - 2 \cdot 0{,}2314 \cdot 10^{-3}}{2 \cdot 639 \cdot 10^{-3}} \right)^2 + \frac{2 \cdot 400}{0{,}85 \cdot 25} \cdot \frac{2660{,}929}{700 \cdot 639} \cdot \left(1 - \frac{61}{639} \right)} \}$$

$$= 11938{,}675\ KN \quad > \quad Nu / \phi = 4023{,}089\ KN$$

.....Ok

f. Perencanaan tulangan geser kolom

$Vuk_1 = (Muk_{atas} + Muk_{bawah})/hk'$

$= (571{,}439 + 617{,}993)/3{,}9$

$= 304{,}982$ KN

$Vuk_2 = 1{,}05.(V_{Dk} + V_{Lk} + (4/K).\ V_{Ek})$

$= 1{,}05.\ (8{,}578 + 15{,}272 + (4/1).35{,}378)$

$= 173{,}63$ KN

Vu pakai adalah nilai terkecil antara Vuk_1 dan Vuk_2, sehingga didapat Vuk pakai = 173,63 KN

$$Vs = \frac{Vuk\ \ pakai}{\phi}$$

$$= \frac{173{,}63}{0{,}6} = 289{,}383\ KN$$

Tersedia tul.sengkang P_{10} mm ⟶ fy = 240 Mpa

$$S = \frac{2 \cdot A_V \cdot fy \cdot d}{Vs}$$

$$= \frac{2 \cdot 0{,}25 \cdot \pi \cdot 10^2 \cdot 240 \cdot 639}{289{,}383 \cdot 10^3}$$

= 83,245 mm ≤ 8D22 = 176 mm, maka S pakai = 80 mm

Periksa :

Vuk/Ø < Vs

173,63 /0,8 < 289,383 KN

217,0375 KN < 289,383 KN .....Ok

4. Pertemuan balok kolom (JG41)

➢ Pertemuan balok kolom luar

1. Perhitungan gaya-gaya dalam

Mnak,ki = 233,63 KNm

Mnak,ka = 378,968 KNm

Mkap,ki = 1,25. Mnak,b

= 1,25. 233,63 = 292,038 KNm

Mkap,ka = 1,25. Mnak,b

= 1,25. 378,968 = 473,710 KNm

$$V_{kol} = \frac{0,7 \cdot \left(Lki/Lnki \cdot Mkap,ki + Lka/Lnka \cdot Mkap,ka \right)}{1/2 \cdot (hk,a + Hk,b)}$$

$$= \frac{0,7 \cdot \left(7,2/6,5 \cdot 292,038 + 1/0,65 \cdot 473,710 \right)}{1/2 \cdot (5 + 3,5)} = 173,315\ KN$$

T_{ka} = 0,7. Mkap,ka/zka

C_{ki} = 0,7. Mkap,ki/zki

d = 639 mm = 0,639 m

zka = zki = 0,85. d

= 0,85. 639 = 543,15 mm = 0,5432 m

T_{ka} = 0,7. 473,710 /0,5432 = 610,507 KN

C_{ki} = 0,7. 292,038 /0,5432 = 376,372 KN

Vj,h = T_{ka} + C_{ki} – Vkol

= 610,507 + 376,372 – 173,315 = 813,563 KN

2. Kontrol tegangan geser horizontal minimal

$$vj,h = \frac{Vj,h}{bj \cdot hc} \le 1{,}5 \cdot \sqrt{fc'} \quad ; bj = h = 700\ mm$$

$$vj,h = \frac{813{,}563}{0{,}7 \cdot 0{,}7} = 1660\ KN/m^2 = 1{,}660\ \text{N/mm}^2 < 1{,}5.\sqrt{25} = 7{,}5\ \text{N/m}^2$$

.....Ok

3. Penulangan geser horizontal

Nu = 2556,448 KN

$$\frac{Nu}{Ag} = \frac{2556{,}448}{0{,}7 \cdot 0{,}7} = 5217\ KN/m^2 = 5{,}217\ \text{N/mm}^2$$

$$Vc,h = \tfrac{2}{3} \cdot \sqrt{\left\{\left(Nuk/Ag\right) - 0{,}1 \cdot fc'\right\}} \cdot bj \cdot hc$$

$$Vc,h = \tfrac{2}{3} \cdot \sqrt{\{5{,}217 - 0{,}1 \cdot 25\}} \cdot 700 \cdot 700$$

$$= 538479\ \text{N} = 538{,}479\ \text{KN}$$

Vs,h + Vc,h = Vj,h

Vs,h = 813,563 - 538,479 = 275,084 KN

$$Aj,h = \frac{Vs,h}{fy} = \frac{275084}{240} = 1146{,}184\ mm^2$$

Digunakan sengkang rangkap P10 dengan Av = 314,159 mm^2

$$Jumlah\ lapis\ sengkang = \frac{1146{,}184}{314{,}159} = 3{,}648 = 4\ lapis$$

4. Penulangan geser vertikal

$$Vc,v = \frac{As'}{As} \cdot Vj,h \cdot \left(0{,}6 + Nuk/Ag \cdot fc'\right)$$

$$= 1 \cdot 813{,}563 \cdot 10^3 \cdot (0{,}6 + 5{,}217 / 25)$$

$$= 657920N = 657{,}920KN$$

Vj,v = bc/hc. Vj,h = (0,7/0,7). 813,563= 813,563 KN

Vs,v = Vj,v – Vc,v = 813,563 – 657,920 = 155,643 KN

$$Aj,v = \frac{Vs,v}{fy} = \frac{155643}{240} = 648{,}513\ mm^2$$

pakai 9D10 dengan As = 706,86 mm^2 > Aj,v = 648,513 mm^2

5. Kontrol jarak tulangan vertikal

s = (hc – 2. pb – 2.$O_{sengkang}$ – n. $O_{tulangan}$)/(n -1)

= (700 – 2. 40 – 2. 10 – 2. 10)/9 = 63,75 mm ≥ 20 mm .....Ok

➢ Pertemuan balok kolom dalam

1. Perhitungan gaya-gaya dalam

Mnak,ki = 158,474 KNm

Mnak,ka = 305,76 KNm

Mkap,ki = 1,25. Mnak,b = 1,25. 158,474 = 198,093 KNm

Mkap,ka = 1,25. Mnak,b = 1,25. 305,76 = 382,200 KNm

$$V_{kol} = \frac{0{,}7 \cdot \left(\frac{Lki}{Lnki} \cdot Mkap,ki + \frac{Lka}{Lnka} \cdot Mkap,ka\right)}{\frac{1}{2} \cdot (hk,a + Hk,b)}$$

$$= \frac{0{,}7 \cdot \left(\frac{3{,}6}{3{,}05} \cdot 198{,}093 + \frac{7{,}2}{6{,}5} \cdot 382{,}200\right)}{\frac{1}{2} \cdot (5 + 3{,}5)} = 108{,}240\ KN$$

T_{ka} = 0,7. Mkap,ka/zka

C_{ki} = 0,7. Mkap,ki/zki

d = 639 mm = 0,639 m

zka = zki = 0,85. d = 0,85. 639 = 543,15 mm = 0,5432 m

T_{ka} = 0,7. 382,200 /0,5432 = 492,571 KN

C_{ki} = 0,7. 198,093 /0,5432 = 255,297 KN

$Vj,h = T_{ka} + C_{ki} - Vkol = 492,571 + 255,297 - 108,24 = 639,628$ KN

2. Kontrol tegangan geser horizontal minimal

$$vj,h = \frac{Vj,h}{bj \cdot hc} \leq 1,5 \cdot \sqrt{fc'} \quad ; bj = h = 700\ mm$$

$$vj,h = \frac{639,628}{0,7 \cdot 0,7} = 1305\ KN/m^2 = 1,305\ \text{N/mm}^2 < 1,5.\sqrt{25} = 7,5\ \text{N/m}^2$$

.....Ok

3. Penulangan geser horizontal

Nu = 2556,448 KN

$$\frac{Nu}{Ag} = \frac{2556,448}{0,7 \cdot 0,7} = 5217\ KN/m^2 = 5,217\ \text{N/mm}^2$$

$$Vc,h = \frac{2}{3} \cdot \sqrt{\left\{\left(\frac{Nuk}{Ag}\right) - 0,1 \cdot fc'\right\}} \cdot bj \cdot hc$$

$$Vc,h = \frac{2}{3} \cdot \sqrt{\{5,217 - 0,1 \cdot 25\}} \cdot 700 \cdot 700$$

$= 538,479$ N $= 538,479$ KN

$Vs,h + Vc,h = Vj,h$

$Vs,h = 639,628 - 538,479 = 101,149$ KN

$$Aj,h = \frac{Vs,h}{fy} = \frac{101149}{240} = 421,454\ mm^2$$

Digunakan sengkang rangkap P10 dengan Av = 314,159 mm^2

$$Jumlah\ lapis\ sengkang = \frac{421,454}{314,159} = 1,342 = 2\ lapis$$

4. Penulangan geser vertikal

$$Vc,v = \frac{As'}{As} \cdot Vj,h \cdot \left(0{,}6 + {Nuk}/{Ag \cdot fc'}\right)$$
$$= 1 \cdot 639{,}628 \cdot 10^3 \cdot (0{,}6 + 5{,}217/25)$$
$$= 517261\ N = 517{,}261\ KN$$

Vj,v = bc/hc. Vj,h = (0,7/0,7). 639,628 = 639,628 KN

Vs,v = Vj,v – Vc,v = 639,628 – 517,261 = 122,367 KN

$$Aj,v = \frac{Vs,v}{fy} = \frac{122367}{240} = 509{,}864\ mm^2$$

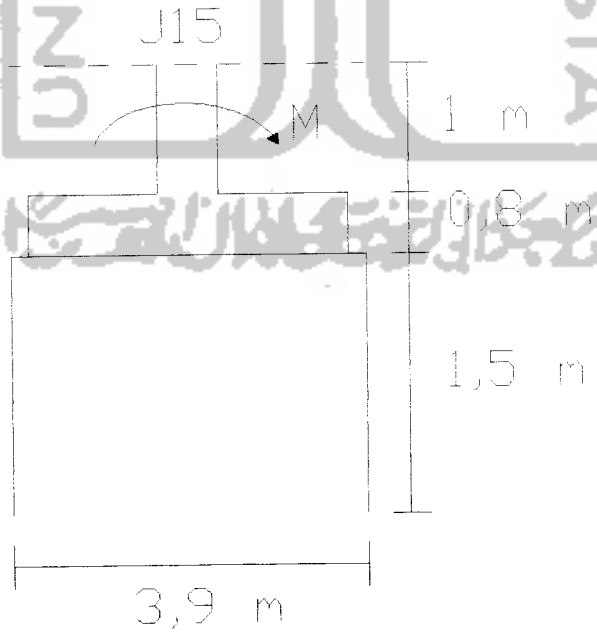
pakai 7P10 dengan As = 549,78 mm^2 > Aj,v = 509,864 mm^2

5. Kontrol jarak tulangan vertikal

s = (hc – 2. pb – 2.$O_{sengkang}$ – n. $O_{tulangan}$)/(n -1)

= (700 – 2. 40 – 2. 10 – 2. 10)/6 = 88,33 mm ≥ 20 mm .....Ok

4.7 Perencanaan Pondasi



Gambar 4.11 Perencanaan pondasi J15

1. Data : J15

   a. σ tanah = 250 KN/m^2

   b. h tanah keras = 1,8 m

   c. fc' = 25 MPa

   d. fy = 400 MPa

   e. γ tanah = 18 KN/m^3

   f. γ beton = 24 KN/m^3

   g. P = 4475,201 KN

   h. Mx = 24,311 KNm

   i. My = 19,613 KNm

2. Perencanaan pondasi sumuran

   Bj beton siklop = 23 KN/m^3

   Coba $Ø_{sumuran}$ = 3 m

   h pondasi sumuran = 1,5 m

   P pondasi sumuran = 0,25. π. $Ø^2$. h. γ

   = 0,25. π. 3^2. 1,5. 23

   = 243,866 KN

   P total = P + P pondasi sumuran

   = 4475,201 + 243,866

   = 4719,067 KN

   σ ijin di bawah sumuran = 450 KN/m^2

   Daya dukung tanah netto :

   $q = h_1 . \gamma_1 + h_2 . \gamma_2$

$= 1.18 + 0{,}8.\ 24 = 37{,}2\ \text{KN/m}^2$

q netto $= \sigma_{\text{sumuran}} - q$

$= 450 - 37{,}2 = 412{,}8\ \text{KN/m}^2$

$$A\ perlu = \frac{P_{total}}{\sigma_{\text{tanah}}} = \frac{4719{,}067}{412{,}8} = 11{,}43\ m^2$$

$$Diameter\ sumuran\ perlu = \sqrt{\frac{A\ perlu}{0{,}25 \cdot \pi}} = \sqrt{\frac{11{,}43}{0.25 \cdot \pi}} = 3{,}815\ m$$

Dipakai diameter sumuran = 3,9 m

3. Perencanaan pondasi telapak

Asumsi tebal pelat pondasi = 0,8 m

Ukuran kolom : $b_k = 700$ mm

$h_k = 700$ mm

σ ijin di bawah pondasi telapak :

σ netto $= 0{,}5.\ 10\%.\ 25\ \text{MPa} = 1250\ \text{KN/m}^2$

Dimensi pondasi bujur sangkar (terdapat momen yang bekerja pada arah x dan y) :

$$\sigma_{netto} = \frac{P}{A_{perlu}} + \frac{Mx}{1/6 \cdot Bx \cdot By^2} + \frac{My}{1/6 \cdot By \cdot Bx^2}$$

dicoba dengan nilai B = 2 m

$$A_{perlu} = \frac{P}{\sigma_{netto} - \left(\frac{Mx}{1/6 \cdot Bx \cdot By^2}\right) - \left(\frac{My}{1/6 \cdot By \cdot Bx^2}\right)}$$

$$= \frac{4719{,}067}{1250 - \left(\frac{24{,}311}{1/6 \cdot 2 \cdot 2^2}\right) - \left(\frac{19{,}613}{1/6 \cdot 2 \cdot 2^2}\right)}$$

$= 3{,}877\ m^2$

$B_p = \sqrt{A} = \sqrt{3{,}877} = 1{,}97\ m$

dicoba $B_p = 2{,}3$ m , $L_p = 2{,}3$ m

Luas penampang pelat pondasi : $A_{ada} = B_p \times L_p = 2{,}3 \times 2{,}3 = 5{,}29\ m^2$

Kontrol luas pelat pondasi dan tegangan yang terjadi :

$A_{ada} = 5{,}29\ m^2 > A_{perlu} = 3{,}877\ m^2$ ..... Ok

Tegangan kontak yang terjadi di dasar pondasi :

$$\sigma = \frac{P}{A_{ada}} + \frac{Mx}{1/6 \cdot Bx \cdot By^2} + \frac{My}{1/6 \cdot By \cdot Bx^2}$$

$$\sigma = \frac{4719{,}067}{5{,}29} + \frac{24{,}311}{1/6 \cdot 2{,}3 \cdot 2{,}3^2} + \frac{19{,}613}{1/6 \cdot 2{,}3 \cdot 2{,}3^2}$$

$= 913{,}734\ KN/m^2 < \sigma_{netto} = 1250\ KN/m^2$ ..... Ok

Perencanaan tebal pondasi telapak (syarat kuat geser) :

Asumsi tebal pondasi (h_p) = 800 mm

Tebal selimut beton (pb) = 70 mm

Ø tulangan pokok = 22 mm

d = t pondasi – t penutup beton – 0,5. Ø tulangan

= 800 – 70 – 0,5.22 = 719 mm

4. Tinjauan terhadap beban sementara

Eksentrisitas yang terjadi :

$$ex = \frac{Mx}{P} = \frac{24{,}311}{4719{,}067} = 0{,}0052\ m$$

$$ey = \frac{My}{P} = \frac{19{,}613}{4719{,}067} = 0{,}0042\ m$$

Kontrol tegangan yang terjadi :

$$\sigma = \frac{P}{B_p \cdot (L_p - 2 \cdot ex) + L_p \cdot (B_p - 2 \cdot ey)}$$

$$= \frac{4719,067}{2,3 \cdot (2,3 - 2 \cdot 0,0052) + 2,3 \cdot (2,3 - 2 \cdot 0,0042)}$$

$= 447,867 \text{ KN/m}^2 < 1,5.\sigma_{netto} = 1,5.\ 1250 = 1875 \text{ KN/m}^2$ .....Ok

5. Perhitungan geser beton untuk 2 arah

➢ Ditinjau pada arah momen terbesar :

Pu = 4541,95 KN

Mux = 70,101 KN/m^2

Muy = 72,908 KN/m^2

$B_1 = b_{kolom} + d$

= 700 + 719 = 1419 mm

$B2 = h_{kolom} + d$

= 700 + 719 = 1419 mm

- Tegangan kontak yang terjadi :

$$qu = \frac{Pu}{A_{ada}} \pm \frac{Mux}{1/6 \cdot Bx \cdot By^2} \pm \frac{Muy}{1/6 \cdot By \cdot Bx^2}$$

$$= \frac{4541,95}{5,29} \pm \frac{70,101}{1/6 \cdot 2,3 \cdot 2,3^2} \pm \frac{72,908}{1/6 \cdot 2,3 \cdot 2,3^2}$$

$qu_{max} = 929,115 \text{ KN/m}^2$

$qu_{min} = 788,069 \text{ KN/m}^2$

$qu_{terjadi} = ½..(\ qu_{max} + qu_{min}) = ½.(929,115 + 788,069) = 858,592 \text{ KN/m}^2$

Gambar 4.12 Penampang kritis dan permukaan geser beton 2 arah

- Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

  $Vu = qu_{terjadi}.((B_p.\ L_p) - (B1.\ B2))$

  $= 858{,}592.((2{,}3.\ 2{,}3) - (1{,}419.\ 1{,}419)) = 2813{,}124\ KN$

  $Vu/\phi = 2813{,}124/0{,}6 = 4688{,}54\ KN$

- Kekuatan beton menahan geser :

  $$\beta c = \frac{sisi\ panjang}{sisi\ pendek} = \frac{B_p}{L_p} = \frac{2{,}3}{2{,}3} = 1$$

  $bo = 2.(B1 + B2) = 2.(1419 + 1419) = 5676\ mm$

$$Vc_1 = \left(1 + \frac{2}{\beta c}\right) \cdot \left(2 \cdot \sqrt{fc'}\right) \cdot bo \cdot d$$

$$= \left(1 + \frac{2}{1}\right) \cdot \left(2 \cdot \sqrt{25}\right) \cdot 5676 \cdot 719 \cdot 10^{-3} = 122431{,}32KN$$

Tetapi nilai tersebut tidak boleh lebih besar dari nilai :

$Vc_2 = (4.\ \sqrt{fc'}).bo.\ d$

$= (4.\ \sqrt{25}).\ 5676.\ 719.\ 10^{-3} = 81620{,}88$ KN

Digunakan nilai terkecil dari Vc_1 dan Vc_2, yaitu $Vc_2 = 81620{,}88$ KN

- Kontrol gaya geser :

$Vc_2 = 81620{,}88$ KN $\geq Vu/\phi = 2813{,}124/0{,}6 = 4688{,}54\ KN$ ..... Ok

6. Perhitungan geser beton untuk 1 arah

➢ Ditinjau pada arah momen terbesar

$G = (B_p - 2.d - b_{kolom})/2$

$= (2300 - 2.\ 719 - 700)/2 = 81$ mm

Arah X :

- Tegangan kontak yang terjadi :

$$qux = \frac{Pu}{A_{ada}} \pm \frac{Mux}{1/6 \cdot Bx \cdot By^2}$$

$$= \frac{4541{,}95}{5{,}29} \pm \frac{70{,}101}{1/6 \cdot 2{,}3 \cdot 2{,}3^2}$$

$qux_{max} = 893{,}161$ KN/m^2

$qux_{min} = 824{,}022$ KN/m^2

$$qu_m = \frac{(L_p - G) \cdot qux_{max} + G \cdot qux_{min}}{L_p}$$

$$= \frac{(2{,}3 - 0{,}081) \cdot 893{,}161 + 0{,}081 \cdot 824{,}022}{2{,}3} = 890{,}726\ KN/m^2$$

$qux_{terjadi} = ½.(qux_{max} + qux_{min}) = ½.(893,161 + 824,022) = 858,592\ KN/m^2$

- Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

  $Vu = qu_{terjadi}.\ L_p.\ G$

  $= 858,592.\ 2,3.\ 0,081 = 159,956\ KN$

  $Vu/\phi = 159,956/0,6 = 266,593\ KN$

- Kekuatan beton menahan geser :

  $Vc = 1/6.\ \sqrt{fc'}.\ L_p.\ d$

  $= 1/6.\ \sqrt{25}.\ 2300.\ 719.\ 10^{-3} = 1378,083\ KN$

- Kontrol gaya geser :

  $Vc = 1378,083\ KN \geq Vu/\phi = 159,956/0,6 = 266,593\ KN$ .....Ok

Arah Y :

- Tegangan kontak yang terjadi :

$$quy = \frac{Pu}{A_{ada}} \pm \frac{Muy}{1/6 \cdot By \cdot Bx^2}$$

$$= \frac{4541,95}{5,29} \pm \frac{72,908}{1/6 \cdot 2,3 \cdot 2,3^2}$$

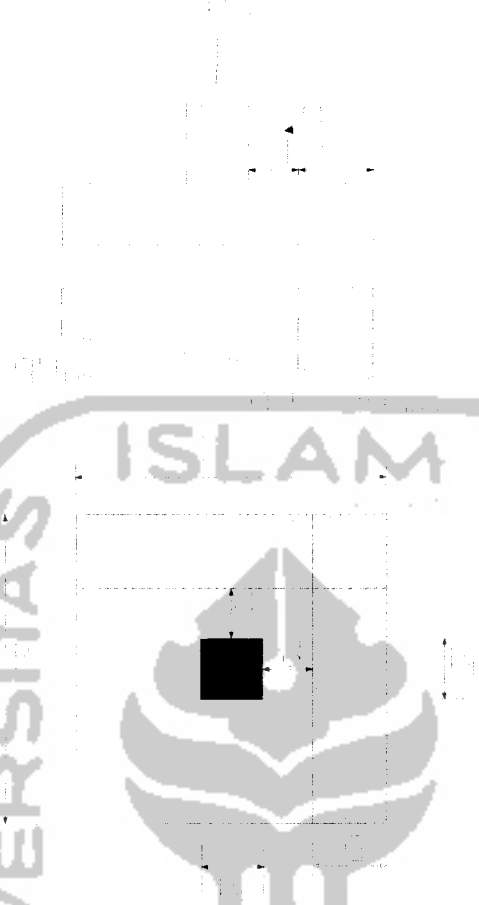
$quy_{max} = 894,545\ KN/m^2$

$quy_{min} = 822,638\ KN/m^2$

$$qu_m = \frac{(L_p - G) \cdot quy_{max} + G \cdot quy_{min}}{L_p}$$

$$= \frac{(2,3 - 0,081) \cdot 894,545 + 0,081 \cdot 822,638}{2,3} = 892,013\ KN/m^2$$

$qux_{terjadi} = ½.(qux_{max} + qux_{min}) = ½.(894,545 + 822,638) = 858,592\ KN/m^2$



Gambar 4.13 Penampang kritis dan permukaan geser beton 1 arah

- Gaya geser akibat beban luar yang bekerja pada penampang kritis pondasi :

  $Vu = qu_{terjadi}. B_p. G$

  $= 858{,}592. 2{,}3. 0{,}081 = 159{,}956$ KN

  $Vu/\phi = 159{,}956/0{,}6 = 266{,}593\ KN$

- Kekuatan beton menahan geser :

  $Vc = 1/6. \sqrt{fc'}. B_p. d$

  $= 1/6. \sqrt{25}. 2300. 719. 10^{-3} = 1378{,}083$ KN

- Kontrol gaya geser :

$$Vc = 1378{,}083 \text{ KN} \geq Vu/\phi = 159{,}956/0{,}6 = 266{,}593 \text{ KN} \quad \ldots\ldots\ldots\text{Ok}$$

7. Kuat tumpuan pondasi

- Kuat tumpuan pondasi :

$$\phi.Pn = \phi.(0{,}85.\ fc'.\ A_1.\ \sqrt{A_2/A_1}\ )$$

Luas pelat pondasi (A_2) $= B_p.\ L_p = 2{,}3.\ 2{,}3 = 5{,}29\ m^2$

Luas penampang kolom (A_1) $= b_k.\ h_k = 0{,}7.\ 0{,}7 = 0{,}49\ m^2$

$$\sqrt{A_2/A_1} = \sqrt{5{,}29/0{,}49} = 3{,}286 > 2 \text{ (jika lebih besar dari 2, dipakai nilai 2)}$$

$$\phi.Pn = \phi.(0{,}85.\ fc'.\ A_1.\ 2)$$

$$= 0{,}7.\ (0{,}85.\ 25.\ 490000.\ 2).\ 10^{-3} = 14577{,}5 \text{ KN}$$

- Kuat tumpuan kolom :

$$\phi.Pn = \phi.(0{,}85.\ fc'.\ A_1)$$

$$= 0{,}7.\ (0{,}85.\ 25.\ 490000).\ 10^{-3} = 7288{,}75 \text{ KN}$$

- Kontrol kuat tumpuan :

$\phi.Pn_{pondasi} = 14577{,}5 \text{ KN} > \phi.Pn_{kolom} = 7288{,}75 \text{ KN}$ .....Ok

8. Perencanaan tulangan lentur telapak pondasi

Momen yang terjadi :

$$l = \frac{L_p - h_k}{2} = \frac{2{,}3 - 0{,}7}{2} = 0{,}8\ m$$

$qu_{maks} = 929{,}115\ KN/m^2$

$Mu = 0{,}5.\ qu_{maks}.\ l^2 = 0{,}5.\ 929{,}115.\ 0{,}8^2 = 297{,}317$ KNm

$$Mu/\phi = 297{,}317/0{,}8 = 371{,}646\ KNm$$

- Digunakan tulangan pokok $Ø_{22\ mm}$ ⟶ $A_1Ø = ¼ . \pi . 22^2 = 380{,}133\ mm^2$
- Tebal pelat pondasi : h = 800 mm, selimut beton (pb) = 70 mm

  $d = h - pb - 0{,}5 . Ø_{tul\ pokok} = 800 - 70 - 0{,}5.22 = 719$ mm

$$As_{perlu} = \frac{Mu}{\gamma d \cdot fy} \theta = \frac{371{,}646 \cdot 10^6}{0{,}9 \cdot 719 \cdot 400} = 1435{,}814\ mm^2$$

$As_{min} = (1{,}4/fy) . b . d = (1{,}4/400) . 1000 . 719 = 2516{,}5\ mm^2$

$1{,}33\ As_{perlu} = 1{,}33 . 1435{,}814 = 1909{,}633\ mm^2$

Check :

$1{,}33\ As_{perlu} < As_{min}$, maka dipakai $As = 1{,}33.As_{perlu} = 1909{,}633\ mm^2$

Jarak tulangan :

$$x = \frac{A\phi \cdot 1000}{As_{pakai}} = \frac{0{,}25 \cdot \pi \cdot 22^2 \cdot 1000}{1909{,}633} = 199{,}061\ mm$$

Dipakai jarak tulangan (x) = 195 mm ⟶ dipakai tul pokok : $D_{22\ -\ 195}$

$$As_{ada} = \frac{A\phi \cdot 1000}{jarak\ tulangan} = \frac{0{,}25 \cdot \pi \cdot 22^2 \cdot 1000}{195} = 1949{,}399\ mm^2$$

- Kontrol kapasitas lentur pelat pondasi :

$$a = \frac{As \cdot fy}{0{,}85 \cdot fc' \cdot b} = \frac{1949{,}399 \cdot 400}{0{,}85 \cdot 25 \cdot 1000} = 36{,}695\ mm$$

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

$= 1949{,}399 . 400 . (719 - 36{,}695/2) . 10^{-6}$

$= 546{,}341$ KNm > $1{,}33.Mu/Ø = 1{,}33 . 371{,}646 = 494{,}289$ KNm .....Ok

- Perencanaan tulangan susut pondasi :

  $As_{tul\ susut} = 0{,}002.\ b.\ h = 0{,}002.\ 1000.\ 800 = 1600\ mm^2$

- Digunakan tulangan bagi Ø12 mm, sehingga luas tampang 1 tul. susut :

  $A_1Ø = ¼.\ \pi.12^2 = 113{,}097\ mm^2$

  Jarak antar tulangan susut :

$$s \le \frac{A_1\phi}{As_{susut}} = \frac{113{,}097}{1600} = 70{,}685\ mm$$

  Dipakai jarak tulangan (x) = 70 mm ⟶ dipakai tul susut : $P_{12\ -\ 70}$