BAB V
PERHITUNGAN SALURAN PINTU AIR

5.1 Data-Data Perhitungan

5.1.1 Data Teknis Material Beton

- Baja Tulangan

  Diameter ≥ Ø 12 : $\sigma_a = \sigma'_a$ = tegangan tarik/tekan ijin = 2400 kg/cm^2

  Diameter < Ø 12 : $\sigma_a = \sigma'_a$ = tegangan tarik/tekan ijin = 1400 kg/cm^2

  Modulus elastisitas baja = $E_a = 2 \times 10^6$ kg/cm^2
- Beton

  Kuat tekan karakteristik beton = σ'_{bk}= 225 kg/cm^2

  Modulus elastisitas beton =$E_b = 6400\sqrt{\sigma'_{bk}} = 6400\sqrt{225} = 9{,}6\times10^4$kg/cm^2
- Lentur tanpa/dengan gaya normal

  σ'_b (tekan) = 0,33 σ'_{bk} = 0,33 x 225 = 74,25 kg/cm^2

  σ_b (tarik) = 0,48 $\sqrt{\sigma'_{bk}}$ = 0,48 x $\sqrt{225}$ = 7,2 kg/cm^2
- Geser oleh lentur/puntir

  τ_b (tanpa tulangan geser) = 0,43 $\sqrt{\sigma'_{bk}}$ = 0,43 x $\sqrt{225}$ = 6,45 kg/cm^2

  τ_{bm} (dengan tulangan geser) = 1,08 $\sqrt{\sigma'_{bk}}$ = 1,08 x $\sqrt{225}$ = 16,2 kg/cm^2
- Geser dengan lentur oleh puntir

  τ_b (tanpa tulangan geser) = 0,54 $\sqrt{\sigma'_{bk}}$ = 0,54 x $\sqrt{225}$ = 8,1 kg/cm^2

  τ_{bm} (dengan tulangan geser) = 1,35 $\sqrt{\sigma'_{bk}}$ = 1,35 x $\sqrt{225}$ = 20,25 kg/cm^2
- Angka ekivalensi

  $n = E_a / E_b = (2 \times 10^6)/(9{,}6\times10^4) = 20{,}833$

5.1.2 Data Elevasi Muka Air

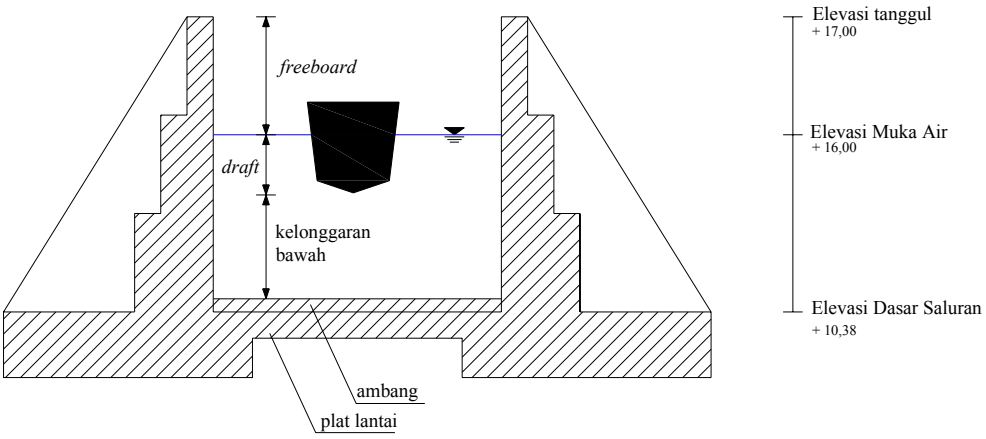
Elevasi muka air saluran pintu air mengikuti elevasi muka air sungai di sekitar bendung (*Up Stream* dan *Down Stream*), yaitu :

- Elevasi saluran bagian atas (Saluran A) : + 16,00 m

- Elevasi saluran bagian bawah (Saluran B) : + 13,50 m

5.2 Perhitungan Elevasi Dasar Saluran

Setelah data elevasi muka air sungai diketahui, maka perhitungan elevasi dasar saluran adalah sebagai berikut :



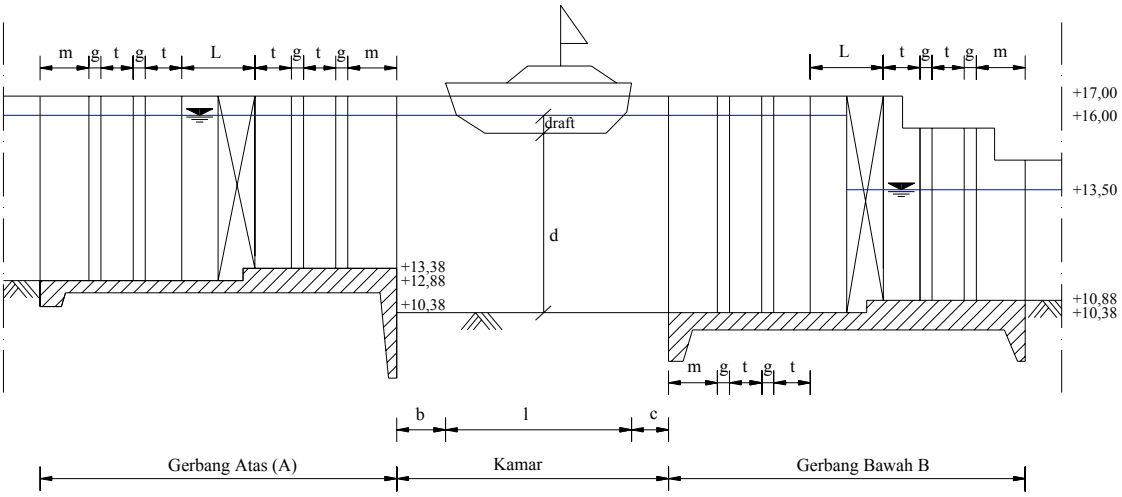
Gambar 5.1 Elevasi Dasar Saluran

Tabel 5.1 Elevasi Dasar Saluran

	Sal. A (Hulu)	**Sal. B (Hilir)**	**Kamar**
Muka Air (m)	+ 16,00	+13,50	+16,00 (diambil yang tertinggi)
Draft (m)	1,62	1,62	
Kelonggaran Bawah (m)	1,00	1,00	
Ambang (m)	0,50	0,50	
Elevasi Dasar Saluran (m)	+ 12,88	+ 10,38	+10,38 (diambil yang terendah)
Freeboard (m)	1,00	1,00	
Elevasi Tanggul (m)	+ 17,00	+ 14,50	+17,00 (diambil yang tertinggi)

5.3 Perhitungan Dimensi Gerbang dan Kamar

Dimensi gerbang dan kamar direncanakan seperti gambar di bawah ini.



Gambar 5.2 Dimensi Gerbang dan Kamar

Tabel 5.2 Standar Ukuran Perencanaan

	Ukuran (m)
Panjang Kapal (l_1)	13,85
Lebar kapal (w)	4,02
Draft	1,62
Kelonggaran Samping (a)	1,00
Kelonggaran Depan (b)	1,50
Kelonggaran Belakang (c)	1,50
Jarak Antar Kapal (e)	2,00
Kelonggaran Bawah (d)	1,00
Jarak Celah *Schotbalk* ke Tepi (m)	2,50
Celah *Schotbalk* (g)	
Jarak Antara Celah *Schotbalk* (t)	1,00
Lebar Pintu Gerbang (L)	
Kelonggaran Depan Pintu (n)	0,25
Kelonggaran Belakang Pintu (s)	0,03

Dimana untuk ukuran celah *schotbalk* (g) dan lebar pintu gerbang (L) belum bisa ditentukan, karena belum dilakukan perhitungan. Perhitungan ukuran celah *schotbalk* (g) dan lebar pintu gerbang (L) dilakukan pada Sub Bab 5.4 dan 5.5.

Tabel 5.3 Dimensi Gerbang

	Rumus	**Nilai**
Lebar Gerbang (W_1)	w + (2 x a)	4,02 + (2 x 1) = **6,02 m**
Panjang Gerbang (Lg)	(2 x m) + (4 x g) + (4 x t) + L	(2 x 2,5) + (4 x g) + (4 x 1) + L = **9 + 4g + L**

Tabel 5.4 Dimensi Kamar

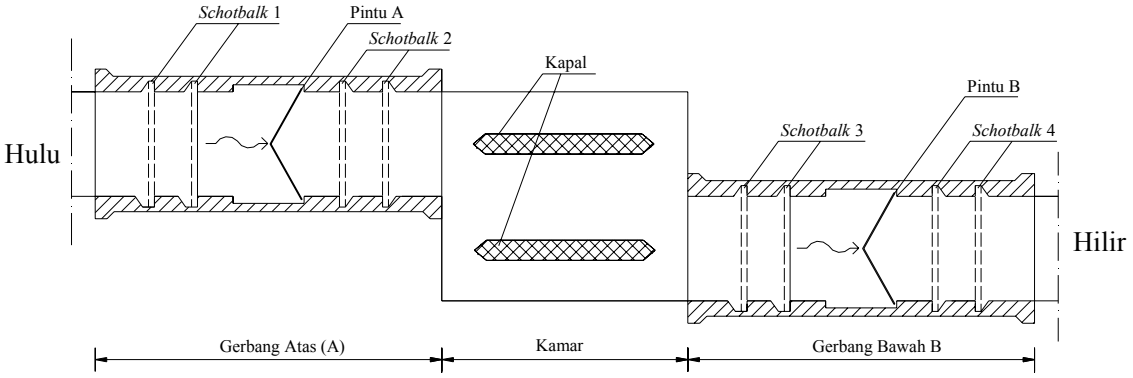
	Rumus	**Nilai**
Lebar Kamar (W_2)	(2 x a) + (n x w) + ((n – 1) x e)	(2 x 1) + (2 x 4,02) + ((2 – 1) x 2) = **12,04 m**
Panjang Kamar (Lk)	b + c + (n x l_1) + ((n – 1) x e)	1,5 + 1,5 + (1 x 13,85) + ((1 – 1) x 2) = **16,85 m**
Luas Kamar (Fk)	W_2 x Lk	12,04 x 16,85 = **202,874 m^2**

Keterangan :

n = jumlah kapal (2 buah secara paralel) dilihat secara melintang dan memanjang kamar. Sehingga nilai n untuk perhitungan W_2 adalah 2, sedangkan nilai n untuk perhitungan Lk adalah 1.

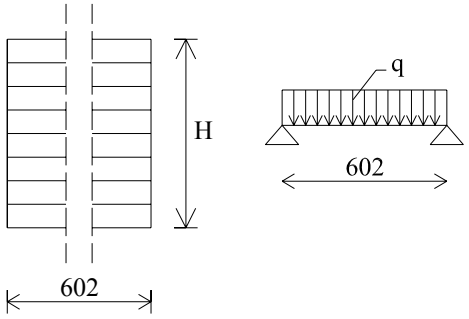
5.4 Perhitungan *Schotbalk*

Setiap pintu gerbang direncanakan memiliki masing-masing dua buah *schotbalk* (sepasang) yang dipasang di depan dan di belakang pintu. Sehingga jumlah total *schotbalk* adalah delapan buah (empat pasang).



Gambar 5.3 Lay Out Letak Pintu Gerbang dan *Schotbalk*

5.4.1 Perhitungan *Schotbalk* 1



Gambar 5.4 Pembebanan pada *Schotbalk* 1

<u>Data Perhitungan</u>

Tinggi tekanan hidrostatis :

H = *draft* kapal + kelonggaran bawah + tinggi ambang

= 1,62 + 1,00 + 0,50

= 3,12 m

Lebar gerbang (W_1) = lebar kapal + 2 x kelonggaran samping

= 4,02 + (2 x 1,00)

= 6,02 m

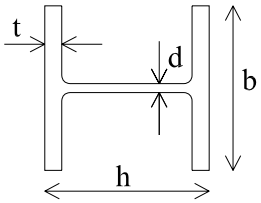
Direncanakan menggunakan profil : IWF 175 x 175

h = 17,5 cm Ix = 2880 cm^4

b = 17,5 cm Wx = 330 cm^3

d = 0,75 cm

t = 1,1 cm



1. Perhitungan pembebanan

   $P_{aw} = \gamma_w.H_w = 1 \times 3{,}12 = 3{,}12$ t/m^2

   q = ½ .P_{aw}.b profil = ½ x 3,12 x 0,175 = 0,273 t/m

   - Perhitungan momen

     q = 0,273 t/m = 2,73 kg/cm

     M = $^1/_8$ q.L^2 dimana L = lebar gerbang

     M = $^1/_8$ x 2,73 x 602^2 = 123670,365 kg cm

   - Cek terhadap kekuatan bahan

     - Terhadap lentur

       $$\sigma = \frac{M}{Wx}$$

       $$= \frac{123670{,}365}{330} = 374{,}759 \text{ kg/cm}^2 < \sigma_{\text{ijin}} = 1400 \text{ kg/cm}^2 \text{ (AMAN)}$$

     - Terhadap geser

       D = ½ .q.L = ½ x 2,73 x 602 = 821,73 kg

       Sx = d ½ h ¼ h + (b – d) t ½ (h – t)

       = 0,75 x ½ 17,5 x ¼ 17,5 + (((17,5 – 0,75) x 1,1) x ½ (17,5 – 1,1))

       = 179,796 cm^3

       $$\tau = \frac{D.S_x}{t.I_x} = \frac{821{,}73x179{,}796}{0{,}75x2880} = 68{,}4 \text{ kg/cm}^2 < 0{,}58.\ \sigma_{\text{ijin}} = 0{,}58 \times 1400$$

       $$= 68{,}4 \text{ kg/cm}^2 < 812 \text{ kg/cm}^2 \text{ (AMAN)}$$

     - Terhadap lendutan

       $$f = \frac{5.L^2.M_{maks}}{48.E.I_x} = \frac{5x602^2x123670{,}365}{48x2x10^6x2880} = 0{,}811 \text{ cm} \leq 1/500.\text{L}$$

       $$= 0{,}811 \leq 1/500 \times 602$$

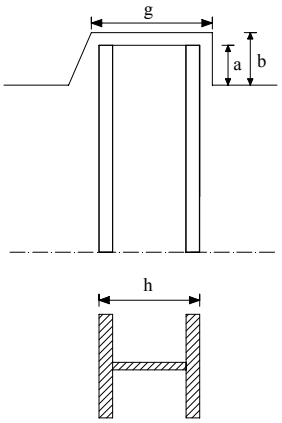
       $$= 0{,}811 \text{ cm} \leq 1{,}204 \text{ cm (AMAN)}$$

2. Jumlah profil yang diperlukan

   H = 3,12 + 1 = 4,12 m

$$n = \frac{H}{b_{profil}} = \frac{412}{17,5} = 23,543 \text{ ; dipakai 24 buah}$$

3. Ukuran celah atau lubang *shcotbalk*



Lebar gerbang (W_1) = 602 cm

h = tinggi profil = 17,5 cm

a = (0,5.h + 5) = 13,75 cm, diambil min 30 cm

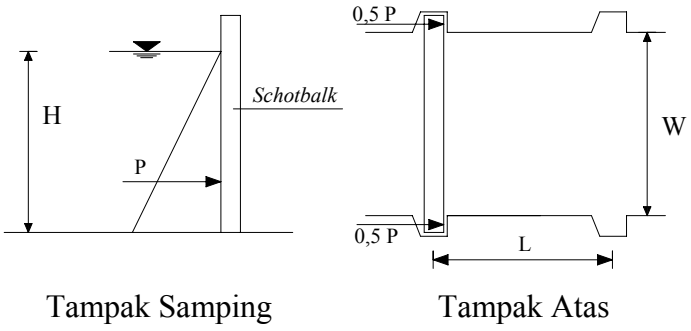
b = a + (3 + 0,1.h) = 34,75 cm

g = h + (3 + 0,1.h) + 1 = 23,25 cm

tinggi *schotbalk* = H + fb = 312 + 100 = 412 cm

lebar *schotbalk* = W + 2a = 602 +(2 x 30) = 662 cm

4. Perhitungan lebar bidang geser



Gambar 5.5 Lebar Bidang Geser *Schotbalk* 1

A = L x tinggi dinding = L x (3,12 + 1) = 4,12 L m^2

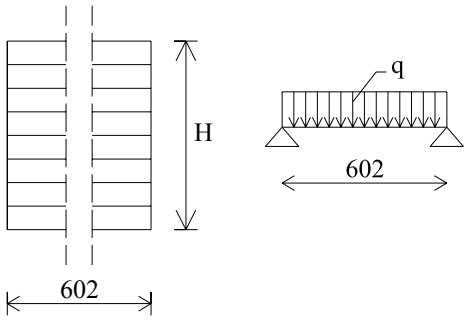
P = ½ γ $(H+Fb)^2$ = ½ x 1 x $4,12^2$ = 8,49 t/m

D = P.W = 8,49 x 6,02 = 51,11 t

$$\tau = \frac{0,5D}{A} \longrightarrow 80 = \frac{0,5x51,11}{4,12L} \longrightarrow L = 0,08 \text{ m}$$

Lebar bidang geser diambil = 1 m

5.4.2 Perhitungan *Schotbalk* 2 & 4



Gambar 5.6 Pembebanan pada *Schotbalk* 2 & 4

<u>Data Perhitungan</u>

Tinggi tekanan hidrostatis :

H = *draft* kapal + kelonggaran bawah

= 1,62 + 1,00

= 2,62 m

Lebar gerbang (W_1) = lebar kapal + 2 x kelonggaran samping

= 4,02 + (2 x 1,00)

= 6,02 m

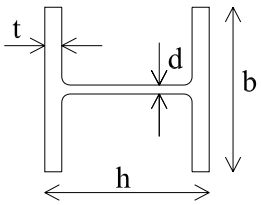
Direncanakan menggunakan profil : IWF 150 x 150

h = 15,0 cm Ix = 1640 cm^4

b = 15,0 cm Wx = 219 cm^3

d = 0,70 cm

t = 1,00 cm



1. Perhitungan pembebanan

   $P_{aw} = \gamma_w.H_w = 1 \times 2,62 = 2,62$ t/m^2

   q = ½ .P_{aw}.b profil = ½ x 2,62 x 0,15 = 0,1965 t/m

   - Perhitungan momen

     q = 0,1965 t/m = 1,965 kg/cm

     M = $^1/_8$ q.L^2 dimana L = lebar gerbang

     M = $^1/_8$ x 1,965 x 602^2 = 89015,48 kg cm

- Cek terhadap kekuatan bahan
  - Terhadap lentur

$$\sigma = \frac{M}{Wx}$$

$$= \frac{89015,48}{219} = 406,463 \text{ kg/cm}^2 < \sigma_{\text{ijin}} = 1400 \text{ kg/cm}^2 \text{ (AMAN)}$$

  - Terhadap geser

D = ½ .q.L = ½ x 1,965 x 602 = 591,465 kg

Sx = d ½ h ¼ h + (b – d) t ½ (h – t)

= 0,7 x ½ 15 x ¼ 15 + (((15 – 0,7) x 1) x ½ (15 – 1))

= 119,788 cm^3

$$\tau = \frac{D.S_x}{t.I_x} = \frac{591,465x119,788}{0,7x1640} = 61,7 \text{ kg/cm}^2 < 0,58.\sigma_{\text{ijin}} = 0,58\text{x}1400$$

$$= 61,7 \text{ kg/cm}^2 < 812 \text{ kg/cm}^2 \text{ (AMAN)}$$

  - Terhadap lendutan

$$f = \frac{5.L^2.M_{maks}}{48.E.I_x} = \frac{5x602^2x89015,48}{48x2x10^6x1640} = 1,02 \text{ cm} \leq 1/500.\text{L}$$

$$= 1,02 \leq 1/500 \text{ x } 602$$

$$= 1,02 \text{ cm} \leq 1,204 \text{ cm (AMAN)}$$

2. Jumlah profil yang diperlukan

H = 2,62 + 1 = 3,62 m

$$n = \frac{H}{b_{profil}} = \frac{362}{15} = 24,133 \text{ ; dipakai 24 buah}$$

3. Ukuran celah atau lubang *shcotbalk*

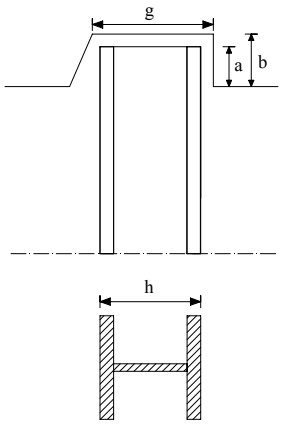
Lebar gerbang (W_1) = 602 cm

h = tinggi profil = 15 cm

a = (0,5.h + 5) = 12,5 cm, diambil min 30 cm

b = a + (3 + 0,1.h) = 34,5 cm

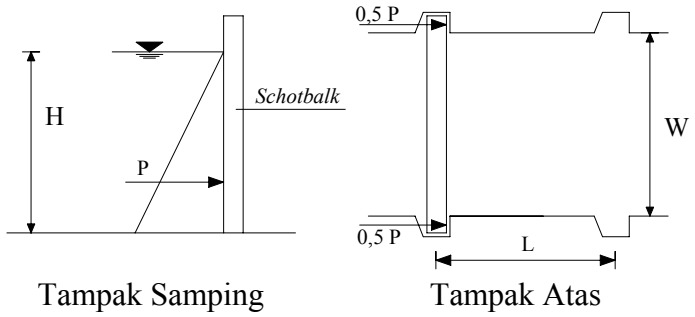
g = h + (3 + 0,1.h) + 1 = 20,5 cm



tinggi *schotbalk* = H + fb = 262 + 100 = 362 cm

lebar *schotbalk* = W + 2a = 602 +(2 x 30) = 662 cm

4. Perhitungan lebar bidang geser



Gambar 5.7 Lebar Bidang Geser *Schotbalk* 2 &4

A = L x tinggi dinding = L x (2,62 + 1) = 3,62 L m^2

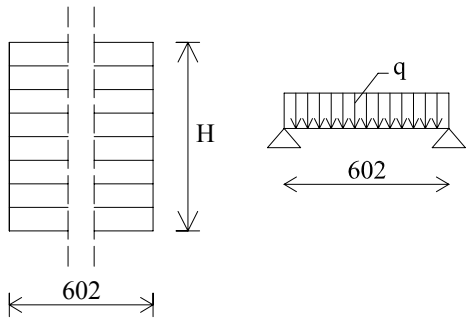
P = ½ γ $(H+Fb)^2$ = ½ x 1 x $3,62^2$ = 6,55 t/m

D = P.W = 6,55 x 6,02 = 39,431 t

$$\tau = \frac{0,5D}{A} \longrightarrow 80 = \frac{0,5x39,431}{3,62L} \longrightarrow L = 0,07 \text{ m}$$

Lebar bidang geser diambil = 1 m

5.4.3 Perhitungan *Schotbalk* 3



Gambar 5.8 Pembebanan pada *Schotbalk* 3

Data Perhitungan

Tinggi tekanan hidrostatis :

H = Elevasi MA hulu – (elevasi MA hilir - *draft* kapal - kelonggaran bawah – tinggi ambang)

= +16 – (+13,5 – 1,62 – 1 – 0,5)

= 5,62 m

Lebar gerbang (W_1) = lebar kapal + 2 x kelonggaran samping

= 4,02 + (2 x 1,00)

= 6,02 m

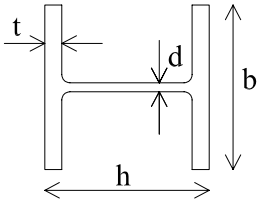
Direncanakan menggunakan profil : IWF 200 x 200

h = 20,0 cm Ix = 4720 cm^4

b = 20,0 cm Wx = 472 cm^3

d = 0,80 cm

t = 1,20 cm



1. Perhitungan pembebanan

$P_{aw} = \gamma_w.H_w$ = 1 x 5,62 = 5,62 t/m^2

q = ½ .P_{aw}.b profil = ½ x 5,62 x 0,2 = 0,562 t/m

- Perhitungan momen

q = 0,562 t/m = 5,62 kg/cm

M = $^1/_8$ q.L^2 dimana L = lebar gerbang

M = $^1/_8$ x 5,62 x 602^2 = 254588,81 kg cm

- Cek terhadap kekuatan bahan

- Terhadap lentur

$$\sigma = \frac{M}{Wx}$$

$$= \frac{254588,81}{472} = 539,383 \text{ kg/cm}^2 < \sigma_{ijin} = 1400 \text{ kg/cm}^2 \text{ (AMAN)}$$

- Terhadap geser

D = ½ .q.L = ½ x 5,62 x 602 = 1691,62 kg

Sx = d ½ h ¼ h + (b – d) t ½ (h – t)

= 0,8 x ½ 20 x ¼ 20 + (((20 – 0,8) x 1,2) x ½ (20 – 1,2))

= 256,576 cm^3

$$\tau = \frac{D.S_x}{t.I_x} = \frac{1691{,}62x256{,}576}{0{,}8x4720} = 114{,}9 \text{ kg/cm}^2 < 0{,}58.\sigma_{\text{ijin}} = 0{,}58\text{x}1400$$

$$= 114{,}9 \text{ kg/cm}^2 < 812 \text{ kg/cm}^2 \text{ (AMAN)}$$

- Terhadap lendutan

$$f = \frac{5.L^2.M_{maks}}{48.E.I_x} = \frac{5x602^2x254588{,}81}{48x2x10^6x4720} = 1{,}02 \text{ cm} \leq 1/500.\text{L}$$

$$= 1{,}02 \leq 1/500 \text{ x } 602$$

$$= 1{,}02 \text{ cm} \leq 1{,}204 \text{ cm (AMAN)}$$

2. Jumlah profil yang diperlukan

   H = 5,62 + 1 = 6,62 m

   $$n = \frac{H}{b_{profil}} = \frac{662}{20} = 33{,}1 \text{ ; dipakai 33 buah}$$

3. Ukuran celah atau lubang *shcotbalk*

   Lebar gerbang (W_1) = 602 cm

   h = tinggi profil = 20 cm

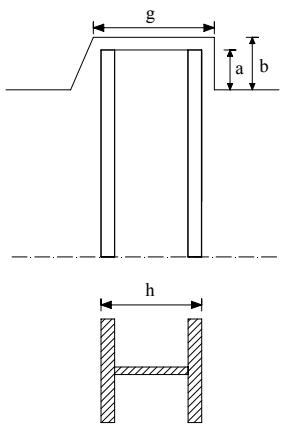
   a = (0,5.h + 5) = 15 cm, diambil min 30 cm

   b = a + (3 + 0,1.h) = 35 cm

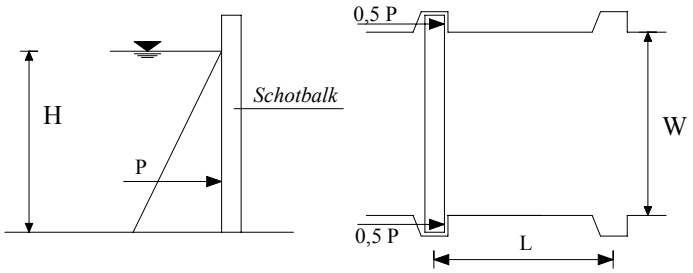
   g = h + (3 + 0,1.h) + 1 = 26 cm

   tinggi *schotbalk* = H + fb = 562 + 100 = 662 cm

   lebar *schotbalk* = W + 2a = 602 +(2 x 30) = 662 cm



4. Perhitungan lebar bidang geser



Tampak Samping Tampak Atas

Gambar 5.9 Lebar Bidang Geser *Schotbalk* 3

A = L x tinggi dinding = L x (5,62 + 1) = 6,62 L m^2

P = ½ γ H^2 = ½ x 1 x $6{,}62^2$ = 21,91 t/m

D = P.W = 21,91 x 6,02 = 131,9 t

$$\tau = \frac{0{,}5D}{A} \longrightarrow 80 = \frac{0{,}5x131{,}9}{6{,}62L} \longrightarrow L = 0{,}12 \text{ m}$$

Lebar bidang geser diambil = 1 m

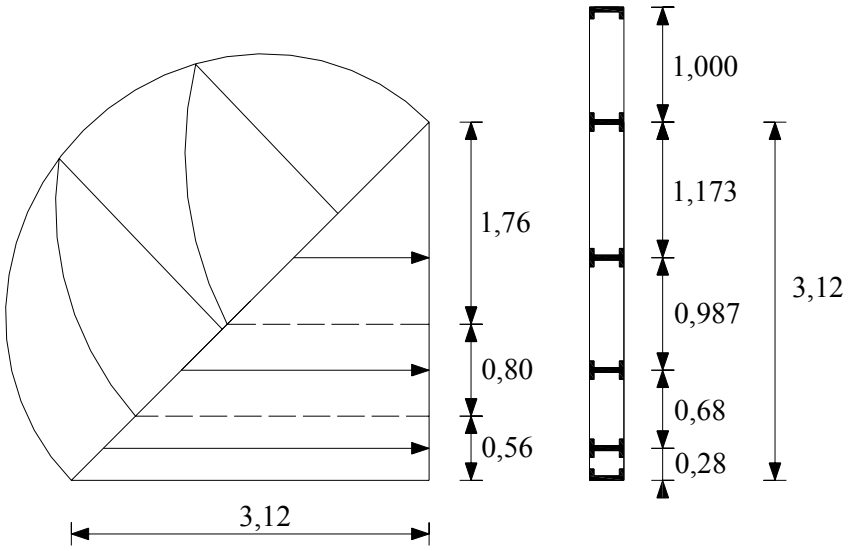
5.5 Perhitungan Pintu Gerbang

5.5.1 Perhitungan Pintu Gerbang A

1. Perhitungan Balok Vertikal dan Horizontal

Dengan tujuan agar permukaan balok sebagai tumpuan pelat menjadi sebidang, maka dimensi profil balok vertikal dan horizontal direncanakan sama dan dianggap balok tertumpu sendi - rol.

- Perhitungan pembebanan



Gambar 5.10 Pembebanan dan Penempatan Profil Pintu Gerbang A

q = ½. γ_w.(h1 + h2).b

q_1 = ½ x 1 x (3,12 + 2,56) x 0,56 = 1,59 t/m

q_2 = ½ x 1 x (2,56 + 1,76) x 0,8 = 1,73 t/m

q_3 = ½ x 1 x 1,76 x 1,76 = 1,55 t/m

Diambil beban maksimum : $q_h = q_2$ = 1,73 t/m

- Perhitungan lebar gerbang (secara praktis)

$$L = \sqrt{(1/6.W)^2 + (1/2.W)^2}$$

$$L = \sqrt{(1/6x6{,}02)^2 + (1/2x6{,}02)^2}$$

= 3,173 m

= 317,3 cm

- Momen yang terjadi

a. Momen pada balok horisontal

M = $^1/_8$ q_h.L^2

Dimana, L = lebar pintu (praktis) = 317,3 cm

q_h = 1,73 t/m = 17,3 kg/cm (beban maksimum)

M = $^1/_8$ x 17,3 x $317{,}3^2$ = 217718,96 kg.cm

b. Momen pada balok vertikal

$M = {}^{1}/_{8}\ q_v.b^2$

Dimana : b = sisi panjang (jarak antar balok horisontal)

q_v = beban masing-masing segmen

b_1 = 68 cm

b_2 = 98,7 cm

b_3 = 117,3 cm

$M_1 = {}^{1}/_{8} \times 17{,}3 \times 68^2 = 9999{,}4$ kg.cm

$M_2 = {}^{1}/_{8} \times 17{,}3 \times 98{,}7^2 = 21066{,}40$ kg.cm

$M_3 = {}^{1}/_{8} \times 17{,}3 \times 117{,}3^2 = 29754{,}46$ kg.cm

- Penentuan profil

Untuk penentuan profil diperhitungkan terhadap momen yang terbesar.

$\sigma_{ijin} = \dfrac{M}{W_{br}}$ dengan M = 217718,96 kg.cm

$W_{nt} = 0{,}8W_{br}$

$W_{br} = \dfrac{1{,}25.M}{\sigma}$

$= \dfrac{1{,}25x217718{,}96}{1400} = 194{,}39\ cm^3$

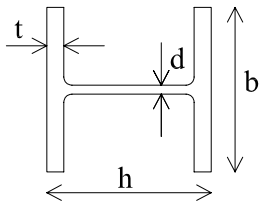
Dicoba dengan profil IWF 200 x 200 dan C 200 x 80 x 7,5 x 11

Profil IWF 200 x 200

h = 20,0 cm t = 1,20 cm

b = 20,0 cm Ix = 4720 cm^4

d = 0,80 cm Wx = 472 cm^3



- Cek terhadap kekuatan bahan

- Terhadap lentur

$\sigma = \dfrac{M}{Wx}$

$= \dfrac{217718{,}96}{472} = 461{,}27\ kg/cm^2 < \sigma_{ijin} = 1400\ kg/cm^2$ (AMAN)

- Terhadap geser

D = ½ .q.L = ½ x 17,3 x 317,3 = 2744,65 kg

Sx = d ½ h ¼ h + (b – d) t ½ (h – t)

= 0,8 x ½ 20 x ¼ 20 + (((20 – 0,8) x 1,2) x ½ (20 – 1,2))

= 256,576 cm^3

$$\tau = \frac{D.S_x}{t.I_x} = \frac{2744,65x256,576}{0,8x4720} = 186,5 \text{ kg/cm}^2 < 0,58.\sigma_{ijin} = 0,58x1400$$

$$= 186,5 \text{ kg/cm}^2 < 812 \text{ kg/cm}^2 \text{ (AMAN)}$$

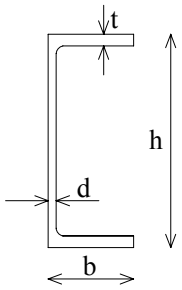
- Terhadap lendutan

$$f = \frac{5.L^2.M_{maks}}{48.E.I_x} = \frac{5x317,3^2 x217718,96}{48x2x10^6 x4720} = 0,24 \text{ cm} \leq 1/500.L$$

$$= 0,24 \leq 1/500 \text{ x } 317,3$$

$$= 0,24 \text{ cm} \leq 0,635 \text{ cm (AMAN)}$$

Profil Kanal C 200 x 80 x 7,5 x 11



h = 20,0 cm	t = 1,1 cm
b = 8,0 cm	Ix = 1950 cm^4
d = 0,75 cm	Wx = 195 cm^3

- Cek terhadap kekuatan bahan

- Terhadap lentur

$$\sigma = \frac{M}{Wx}$$

$$= \frac{217718,96}{195} = 1116,5 \text{ kg/cm}^2 < \sigma_{ijin} = 1400 \text{ kg/cm}^2 \text{ (AMAN)}$$

- Terhadap geser

D = ½ .q.L = ½ x 17,3 x 317,3 = 2744,65 kg

Sx = d ½ h ¼ h + (b – d) t ½ (h – t)

= 0,75 x ½ 20 x ¼ 20 + (((8 – 0,75) x 1,1) x ½ (20 – 1,1))

= 112,864 cm^3

$$\tau = \frac{D.S_x}{t.I_x} = \frac{2744,65x112,864}{0,8x1950}$$ =198,57 kg/cm^2< 0,58. σ_{ijin}=0,58x1400

= 198,57 kg/cm^2 < 812 kg/cm^2 (AMAN)

- Terhadap lendutan

$$f = \frac{5.L^2.M_{maks}}{48.E.I_x} = \frac{5x317,3^2 x217718,96}{48x2x10^6 x1950}$$ = 0,58 cm ≤ 1/500.L

= 0,58 ≤ 1/500 x 317,3

= 0,58 cm ≤ 0,635 cm (AMAN)

2. Perhitungan Plat Tebal Penutup

- Segmen 1

a = jarak antar balok vertikal = 1/7 . L = 1/7. 317,3 = 45,33 cm

b = jarak antar balok horizontal = 28 cm

Perhitungan :

σ_{ijin} = 1400 kg/cm^2 dan k = 0,8

P = ½ x 1 x (3,12 + 2,84) = 2,98 t/m^2 = 0,298 kg/cm^2

$$\sigma_{ijin} = {}^1/_2 .k.P.\frac{a^2.b^2}{(a^2+b^2)t^2}$$

$$1400 = {}^1/_2 x0,8x0,298x\frac{45,33^2 x28^2}{(45,33^2+28^2)xt^2}$$

t^2 = 0,048 t = 0,22 cm

- Segmen 2

a = jarak antar balok vertikal = 1/7 . L = 1/7. 317,3 = 45,33 cm

b = jarak antar balok horizontal = 68 cm

Perhitungan :

σ_{ijin} = 1400 kg/cm^2 dan k = 0,8

P = ½ x 1 x (2,84 + 2,16) = 2,5 t/m^2 = 0,25 kg/cm^2

$$\sigma_{ijin} = {}^1/_2 .k.P.\frac{a^2.b^2}{(a^2+b^2)t^2}$$

$$1400 = {}^1/_2 x0,8x0,25x\frac{45,33^2 x68^2}{(45,33^2+68^2)xt^2}$$

$t^2 = 0,1016$ $\quad$ $t = 0,32$ cm

- Segmen 3

  a = jarak antar balok vertikal = 1/7 . L = 1/7. 317,3 = 45,33 cm

  b = jarak antar balok horizontal = 98,7 cm

  Perhitungan :

  σ_{ijin} = 1400 kg/cm^2 dan k = 0,8

  P = ½ x 1 x (2,16 + 1,173) = 1,6665 t/m^2 = 0,16665 kg/cm^2

  $$\sigma_{ijin} = {}^1/_2 .k.P.\frac{a^2.b^2}{(a^2+b^2)t^2}$$

  $$1400 = {}^1/_2\ x0,8x0,16665x\frac{45,33^2\,x98,7^2}{(45,33^2+98,7^2)xt^2}$$

  $t^2 = 0,081$ $\quad$ $t = 0,28$ cm

- Segmen 4

  a = jarak antar balok vertikal = 1/7 . L = 1/7. 317,3 = 45,33 cm

  b = jarak antar balok horizontal = 117,3 cm

  Perhitungan :

  σ_{ijin} = 1400 kg/cm^2 dan k = 0,8

  P = ½ x 1 x (1,173) = 0,5865 t/m^2 = 0,05865 kg/cm^2

  $$\sigma_{ijin} = {}^1/_2 .k.P.\frac{a^2.b^2}{(a^2+b^2)t^2}$$

  $$1400 = {}^1/_2\ x0,8x0,05865x\frac{45,33^2\,x117,3^2}{(45,33^2+117,3^2)xt^2}$$

  $t^2 = 0,03$ $\quad$ $t = 0,173$ cm

Tebal plat penutup maksimum pada segmen 2 dengan tebal maksimum t = 0,32 cm diambil t = 4 mm

3. Perhitungan Tebal Pintu Gerbang

- Tebal pintu gerbang

  tp = h + 2t

  = 20 + (2 x 0,4) = 20,8 cm

Dimana :

h = tinggi balok (cm)

t = tebal plat penutup (cm)

- Perhitungan lebar pintu gerbang

tp = 20,8 cm

j = 0,2 x tp = 4,16 cm

n = 25 cm

m = 3 cm

p = 15 cm

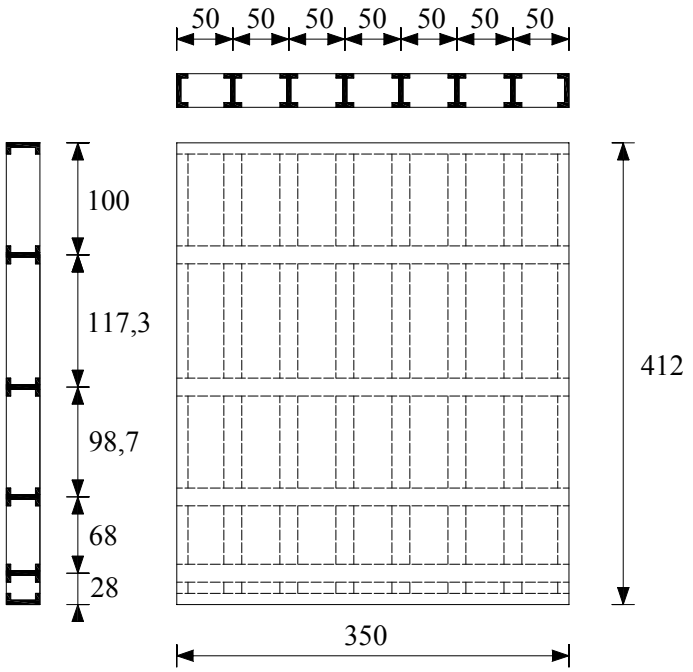
z = 12 cm

α = 15^0

$$L = \frac{w/2 + z + m + t/2}{\cos\alpha} + t/2 + (j.\tan\alpha)$$

$$L = \frac{602/2 + 12 + 3 + 20{,}8/2}{\cos 15^0} + 20{,}8/2 + (4{,}16x\tan 15^0) = 349{,}4 \text{ cm}$$

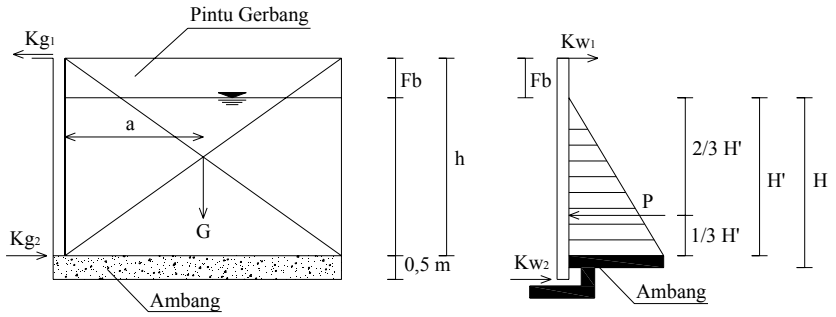
Diambil lebar pintu = 350 cm



Gambar 5.11 Dimensi Pintu Gerbang A

4. Perhitungan Engsel Pintu Gerbang

Dimensi engsel akan dihitung berdasarkan spesifikasi dan beban yang bekerja pada masing-masing pintu gerbang.



Gambar 5.12 Pembebanan pada Engsel Pintu Gerbang A

a. Dimensi Engsel Atas

Lebar gerbang = 3,5 m

Tinggi gerbang = H + Fb

= 3,12 + 1 = 4,12 m

Profil horizontal: IWF 200 x 200 dengan berat 49,9 kg/m

C 200 x 80 x 7,5 x 11 dengan berat = 24,6 kg/m

Profil vertikal : IWF 200 x 200 dengan berat 49,9 kg/m

C 200 x 80 x 7,5 x 11 dengan berat = 24,6 kg/m

Tebal plat penutup : 0,004 m, dengan γ baja = 7850 kg/cm^3

- Berat pintu gerbang

Balok vertikal	= [200x80x7,5x11 = 2 x 4,12 x 24,6	=	202,704 kg
	= IWF 200 x 200 = 6 x 4,12 x 49,9	=	1233,528 kg
Balok horizontal	= [200x80x7,5x11 = 2 x 3,5 x 24,6	=	172,200 kg
	= IWF 200 x 200 = 4 x 3,5 x 49,9	=	698,600 kg
Plat	= 2 x 4,12 x 3,5 x 0,004 x 7850	=	905,576 kg +
		Q =	3212,608 kg
Safety factor = 15 % x Q		=	481,891 kg +
Berat pintu (G)		=	3694,499 kg

- Keseimbangan akibat berat pintu

  a = 0,5 x 3,5 = 1,75 m

  G = 3694,499 kg

  $$Kg = \frac{(G.a)}{h} = \frac{(3694{,}499x1{,}75)}{4{,}12} = 1569{,}265 \text{ kg}$$

  Kg1 = Kg = 1569,256 kg (→)

  Kg2 = Kg = 1569,256 kg (←)

- Keseimbangan akibat tekanan hidrostatis

  Fb = tinggi jagaan *(freeboar*d) = 1 m, L = lebar pintu = 3,5 m

  Tinggi ambang = 0,5 m, H = 3,12 m

  H' = H – tinggi ambang = 3,12 – 0,5 = 2,62 m

  P = ½.γ_w.(H')^2 x L = ½.x 1 x 2,62^2 x 3,5 = 12,013 t = 12013 kg

  $$Kw_1 = \frac{P.1/3H'}{h} = \frac{12013x1/3x2{,}62}{4{,}12} = 2546{,}445 \text{ kg } (\rightarrow)$$

  $$Kw_2 = \frac{P\left(F_b + \frac{2}{3}.H'\right)}{h} = \frac{12013\left(1 + \frac{2}{3}x2{,}62\right)}{4{,}12} = 8008{,}667 \text{ kg } (\rightarrow)$$

- Perhitungan diameter pen engsel

  $$K' = \sqrt{(Kg_1)^2 + (Kw_1)^2} = \sqrt{(1569{,}256)^2 + (2546{,}445)^2} = 2991{,}145 \text{ kg}$$

  y = 10 cm

  M = y x K' = 10 x 2991,145 = 29911,45 kg cm

  $$W = \frac{M}{\sigma_{ijin}} \text{ dan } W = \frac{\pi D^3}{32} \quad ; \quad \sigma_{ijin} = 1400 \text{ kg/cm}^2$$

  $$\frac{29911{,}45}{1400} = \frac{3{,}14xD^3}{32}$$

  D = 6,016 cm

  Diambil diameter pen engsel = 6,0 cm

- Cek terhadap geser

  $$\tau = \frac{4K'}{3\pi R^2} = \frac{4x2991{,}145}{3x3{,}14x3^2} = 141{,}125 \text{ kg/cm}^2 \leq 0{,}58\ \sigma_{ijin}$$

  $$= 141{,}125 \text{ kg/cm}^2 \leq 812 \text{ kg/cm}^2$$

- Perhitungan diameter stang angker

  K' = ½.K_1.Cos (½ α) = ½ x 2991,145 x Cos (½ 45^0) = 1381,729 kg

$F = \frac{K'}{\sigma}$; σ_{ijin}= 1400 kg/cm^2

¼ π D^2 = $\frac{1381,729}{1400}$

D = 1,121 cm

Diambil stang angker = 2 cm = 20 mm

- Perhitungan pelat angker
  - Perhitungan berdasarkan tegangan geser beton
  - Pelat angker dipasang sedalam 20 cm di dalam dinding beton (L = 20)

σ_{bs} = 0,56 √ σ'_{bk} = 0,56√ 225 = 8,4 kg/cm^2

$F = \frac{K'}{\sigma_{bs}}$

3 (a x L) = $\frac{1381,729}{8,4}$

a = 2,742 cm ; (3 adalah banyaknya bidang geser, yaitu: kiri, kanan, bawah)

Diambil ukuran pelat angker = 20 x 20 cm

- Tinjauan terhadap potongan I – I

  M = ½ σ_{bs}.a.(½.a)^2 = 1/8 σ_{bs}.a^3 = 1/8 x 8,4 x 20^3 = 8400 kg cm

  W = 1/6.a.δ^2 dan W = $\frac{M}{\sigma}$; σ_{ijin} = 1400 kg/cm^2

  1/6 x 20 x δ^2 = $\frac{8400}{1400}$

  δ = 1,342 cm

- Tinjauan terhadap potongan II – II

  P = ½.a^2. σ_{bs} = ½ x 20^2 x 8,4 = 1680 kg

  M = P.1/3.½.a.√ 2 = 1680 x 1/3 x ½ x 20 x √ 2 = 7919,59 kg cm

  W = 1/6.a.δ^2 dan W = $\frac{M}{\sigma}$; σ_{ijin} = 1400 kg/cm^2

  1/6 x 20 x δ^2 = $\frac{7919,59}{1400}$

δ = 1,303 cm

Diambil tebal pelat angker = 14 mm

b. Dimensi Engsel Bawah

$K_2 = Kw_2$ = 8008,667 kg

G = 3694,499 kg

$F = \frac{G}{\sigma}$; σ_{ijin} = 1400 kg/cm^2

$F = \frac{3694,499}{1400}$

¼ π D^2 = 2,639

D = 1,833 cm

Diambil diameter pen engsel = 2 cm = 20 mm

- Cek terhadap geser

$\tau = \frac{4K'}{3\pi R^2}$ (kg/cm^2) ; $\tau \leq \tau_{ijin}$ = 0,58 σ_{ijin} = 812 kg/cm^2

- Perhitungan ulang

$\tau = \frac{4K'}{3\pi R^2}$

$812 = \frac{4x8008,667}{3x3,14xR^2}$

R = 2,046 ≈ 2 cm

Diambil diameter pen engsel = 4 cm = 40 mm (dipakai yang terbesar)

- Perhitungan plat andas

a. Perhitungan plat 1

σ'_{bk} = 225 kg/cm^2

$F = \frac{G}{\sigma'_{bk}}$

$a^2 = \frac{3694,499}{225}$

a = 4,052 cm ; diambil ukuran plat = 20 x 20 cm

b. Perhitungan plat 2

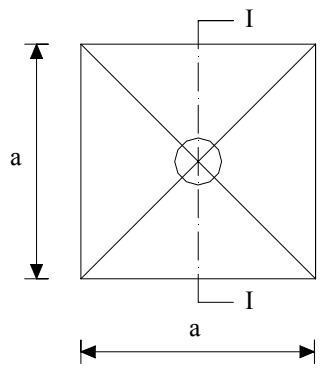
σ'_{bk} = 225 kg/cm^2

$$F = \frac{K_2}{\sigma'_{bk}}$$

$$a^2 = \frac{8008,667}{225}$$

a = 5,966 cm ; diambil ukuran plat = 20 x 20 cm

c. Tinjauan terhadap potongan I – I



Gambar 5.13 Potongan I – I Plat Andas Gerbang A

$M = ½\ \sigma_{bs}.a.(½.a)^2 = 1/8\ \sigma_{bs}.a^3 = 1/8 \times 8,4 \times 20^3 = 8400$ kg cm

$W = 1/6.a.\delta^2$ dan $W = \frac{M}{\sigma}$; $\sigma_{ijin} = 1400$ kg/cm^2

$$1/6 \times 20 \times \delta^2 = \frac{8400}{1400}$$

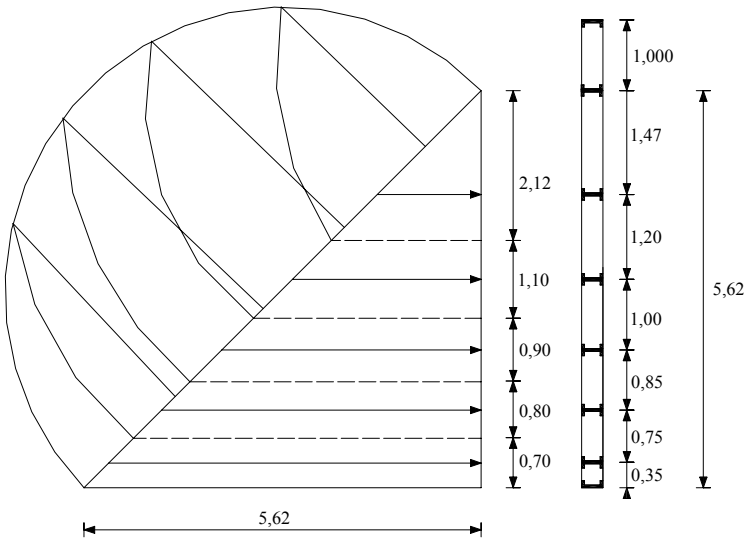
$\delta = 1,342$ cm ; diambil tebal plat angker = 14 mm

5.5.2 Perhitungan Pintu Gerbang B

1. Perhitungan Balok Vertikal dan Horizontal

Dengan tujuan agar permukaan balok sebagai tumpuan pelat menjadi sebidang, maka dimensi profil balok vertikal dan horizontal direncanakan sama dan dianggap balok tertumpu sendi - rol.

- Perhitungan pembebanan



Gambar 5.14 Pembebanan dan Penempatan Profil Pintu Gerbang B

q = ½. γ_w.(h1 + h2).b

q_1 = ½ x 1 x (5,62 + 4,92) x 0,7 = 3,689 t/m

q_2 = ½ x 1 x (4,92 + 4,12) x 0,8 = 3,616 t/m

q_3 = ½ x 1 x (4,12 + 3,22) x 0,9 = 3,303 t/m

q_4 = ½ x 1 x (3,22 + 2,12) x 1,1 = 2,937 t/m

q_5 = ½ x 1 x 2,12 x 2,12 = 2,247 t/m

Diambil beban maksimum : $q_h = q_1$ = 3,689 t/m

- Perhitungan lebar gerbang (secara praktis)

$$L = \sqrt{(1/6.W)^2 + (1/2.W)^2}$$

$$L = \sqrt{(1/6x6{,}02)^2 + (1/2x6{,}02)^2}$$

= 3,173 m

= 317,3 cm

- Momen yang terjadi

a. Momen pada balok horisontal

$M = {}^1/_8\ q_h.L^2$

Dimana, L = lebar pintu (praktis) = 317,3 cm

q_h = 3,689 t/m = 36,89 kg/cm (beban maksimum)

M = $^1/_8$ x x 36,89 x $317{,}3^2$ = 464257,376 kg.cm

b. Momen pada balok vertikal

M = $^1/_8\ q_v.b^2$

Dimana : b = sisi panjang (jarak antar balok horisontal)

q_v = beban masing-masing segmen

b_1 = 75 cm

b_2 = 85 cm

b_3 = 100 cm

b_4 = 120 cm

b_5 = 147 cm

M_1 = $^1/_8$ x 36,89 x 75^2 = 25938,28 kg.cm

M_2 = $^1/_8$ x 36,89 x 85^2 = 33316,28 kg.cm

M_3 = $^1/_8$ x 36,89 x 100^2 = 46112,50 kg.cm

M_4 = $^1/_8$ x 36,89 x 120^2 = 66402,00 kg.cm

M_5 = $^1/_8$ x 36,89 x 147^2 = 99644,50 kg.cm

- Penentuan profil

Untuk penentuan profil diperhitungkan terhadap momen yang terbesar.

$\sigma_{ijin} = \dfrac{M}{W_{br}}$ dengan M = 464257,38 kg.cm

$W_{nt} = 0{,}8W_{br}$

$W_{br} = \dfrac{1{,}25.M}{\sigma}$

$= \dfrac{1{,}25x464257{,}38}{1400} = 414{,}52\ cm^3$

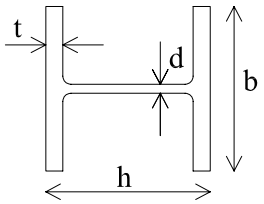
Dicoba dengan profil IWF 300 x 300 dan C 300 x 90 x 10 x 14,5

Profil IWF 300 x 300

h = 30,0 cm t = 1,50 cm

b = 30,0 cm Ix = 20400 cm^4

d = 1,00 cm Wx = 1360 cm^3



- Cek terhadap kekuatan bahan

  - Terhadap lentur

$$\sigma = \frac{M}{Wx}$$

$$= \frac{464257,38}{1360} = 341,37 \text{ kg/cm}^2 < \sigma_{\text{ijin}} = 1400 \text{ kg/cm}^2 \text{ (AMAN)}$$

  - Terhadap geser

D = ½ .q.L = ½ x 36,89 x 317,3 = 5852,59 kg

Sx = d ½ h ¼ h + (b – d) t ½ (h – t)

= 1 x ½ 30 x ¼ 30 + (((30 – 1) x 1,5) x ½ (30 – 1,5))

= 732,38 cm^3

$$\tau = \frac{D.S_x}{t.I_x} = \frac{5852,59x732,38}{0,8x20400} = 262,64 \text{ kg/cm}^2 < 0,58.\sigma_{\text{ijin}} = 0,58\text{x}1400$$

$$= 262,64 \text{ kg/cm}^2 < 812 \text{ kg/cm}^2 \text{ (AMAN)}$$

  - Terhadap lendutan

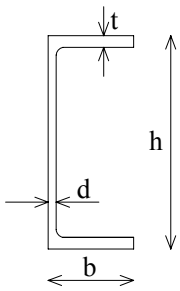
$$f = \frac{5.L^2.M_{maks}}{48.E.I_x} = \frac{5x317,3^2x464257,38}{48x2x10^6x20400} = 0,12 \text{ cm} \leq 1/500.\text{L}$$

$$= 0,12 \leq 1/500 \text{ x } 317,3$$

$$= 0,12 \text{ cm} \leq 0,635 \text{ cm (AMAN)}$$

Profil Kanal C 300 x 90 x 10 x 15,5

h = 30,0 cm	t = 1,55 cm
b = 9,0 cm	Ix = 7400 cm^4
d = 1,00 cm	Wx = 494 cm^3



- Cek terhadap kekuatan bahan

  - Terhadap lentur

$$\sigma = \frac{M}{Wx}$$

$$= \frac{464257,38}{494} = 939,79 \text{ kg/cm}^2 < \sigma_{\text{ijin}} = 1400 \text{ kg/cm}^2 \text{ (AMAN)}$$

- Terhadap geser

D = ½ .q.L = ½ x 36,89 x 317,3 = 5852,59 kg

Sx = d ½ h ¼ h + (b – d) t ½ (h – t)

= 1 x ½ 30 x ¼ 30 + (((9 – 1) x 1,55) x ½ (30 – 1,55))

= 288,89 cm^3

$$\tau = \frac{D.S_x}{t.I_x} = \frac{5852,59x288,89}{0,8x7400} = 285,6 \text{ kg/cm}^2 < 0,58.\sigma_{ijin} = 0,58x1400$$

$$= 285,6 \text{ kg/cm}^2 < 812 \text{ kg/cm}^2 \text{ (AMAN)}$$

- Terhadap lendutan

$$f = \frac{5.L^2.M_{maks}}{48.E.I_x} = \frac{5x317,3^2x464257,38}{48x2x10^6x7400} = 0,33 \text{ cm} \leq 1/500.L$$

$$= 0,33 \leq 1/500 \text{ x } 317,3$$

$$= 0,33 \text{ cm} \leq 0,635 \text{ cm (AMAN)}$$

2. Perhitungan Plat Tebal Penutup

- Segmen 1

a = jarak antar balok vertikal = 1/7 . L = 1/7. 317,3 = 45,33 cm

b = jarak antar balok horizontal = 35 cm

Perhitungan :

σ_{ijin} = 1400 kg/cm^2 dan k = 0,8

P = ½ x 1 x (5,62 + 5,27) = 5,45 t/m^2 = 0,545 kg/cm^2

$$\sigma_{ijin} = {}^1/_2 .k.P.\frac{a^2.b^2}{(a^2+b^2)t^2}$$

$$1400 = {}^1/_2 x0,8x0,545x\frac{45,33^2x35^2}{(45,33^2+35^2)xt^2}$$

t^2 = 0,119 t = 0,345 cm

- Segmen 2

a = jarak antar balok vertikal = 1/7 . L = 1/7. 317,3 = 45,33 cm

b = jarak antar balok horizontal = 75 cm

Perhitungan :

σ_{ijin} = 1400 kg/cm^2 dan k = 0,8

P = ½ x 1 x (5,27 + 4,52) = 4,89 t/m^2 = 0,489 kg/cm^2

$$\sigma_{ijin} = {}^1/_2 .k.P.\frac{a^2.b^2}{(a^2+b^2).t^2}$$

$$1400 = {}^1/_2 x0{,}8x0{,}489x\frac{45{,}33^2 x75^2}{(45{,}33^2+75^2)xt^2}$$

t^2 = 0,21 t = 0,459 cm

- Segmen 3

  a = jarak antar balok vertikal = 1/7 . L = 1/7. 317,3 = 45,33 cm

  b = jarak antar balok horizontal = 85 cm

  Perhitungan :

  σ_{ijin} = 1400 kg/cm^2 dan k = 0,8

  P = ½ x 1 x (4,52 + 3,67) = 4,09 t/m^2 = 0,409 kg/cm^2

$$\sigma_{ijin} = {}^1/_2 .k.P.\frac{a^2.b^2}{(a^2+b^2).t^2}$$

$$1400 = {}^1/_2 x0{,}8x0{,}409x\frac{45{,}33^2 x85^2}{(45{,}33^2+85^2)xt^2}$$

t^2 = 0,187 t = 0,432 cm

- Segmen 4

  a = jarak antar balok vertikal = 1/7 . L = 1/7. 317,3 = 45,33 cm

  b = jarak antar balok horizontal = 100 cm

  Perhitungan :

  σ_{ijin} = 1400 kg/cm^2 dan k = 0,8

  P = ½ x 1 x (3,67 + 2,67) = 3,17 t/m^2 = 0,317 kg/cm^2

$$\sigma_{ijin} = {}^1/_2 .k.P.\frac{a^2.b^2}{(a^2+b^2).t^2}$$

$$1400 = {}^1/_2 x0{,}8x0{,}317x\frac{45{,}33^2 x100^2}{(45{,}33^2+100^2)xt^2}$$

t^2 = 0,154 t = 0,393 cm

- Segmen 5

  a = jarak antar balok vertikal = 1/7 . L = 1/7. 317,3 = 45,33 cm

  b = jarak antar balok horizontal = 120 cm

Perhitungan :

σ_{ijin} = 1400 kg/cm^2 dan k = 0,8

P = ½ x 1 x (2,67 + 1,47) = 2,07 t/m^2 = 0,207 kg/cm^2

$$\sigma_{ijin} = {}^1/_2 .k.P.\frac{a^2.b^2}{(a^2+b^2)t^2}$$

$$1400 = {}^1/_2 x0{,}8x0{,}207x\frac{45{,}33^2 x120^2}{(45{,}33^2+120^2)xt^2}$$

t^2 = 0,106 t = 0,326 cm

- Segmen 6

a = jarak antar balok vertikal = 1/7 . L = 1/7. 317,3 = 45,33 cm

b = jarak antar balok horizontal = 147 cm

Perhitungan :

σ_{ijin} = 1400 kg/cm^2 dan k = 0,8

P = ½ x 1 x (1,47) = 0,735 t/m^2 = 0,074 kg/cm^2

$$\sigma_{ijin} = {}^1/_2 .k.P.\frac{a^2.b^2}{(a^2+b^2)t^2}$$

$$1400 = {}^1/_2 x0{,}8x0{,}074x\frac{45{,}33^2 x147^2}{(45{,}33^2+147^2)xt^2}$$

t^2 = 0,039 t = 0,199 cm

Tebal plat penutup maksimum pada segmen 2 dengan tebal maksimum t = 0,459 cm diambil t = 5 mm

3. Perhitungan Tebal Pintu Gerbang

- Tebal pintu gerbang

tp = h + 2t

= 30 + (2 x 0,5) = 31 cm

Dimana :

h = tinggi balok (cm)

t = tebal plat penutup (cm)

- Perhitungan lebar pintu gerbang

tp = 31 cm

j = 0,2 x tp = 6,2 cm

n = 25 cm

m = 2 cm

p = 12 cm

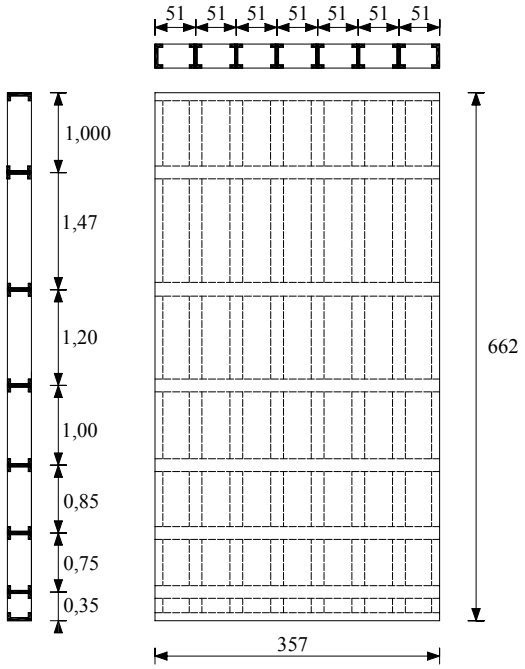
z = 10 cm

α = 15^0

$$L = \frac{w/2 + z + m + t/2}{\cos\alpha} + t/2 + (j.\tan\alpha)$$

$$L = \frac{602/2 + 10 + 2 + 31/2}{\cos 15^0} + 31/2 + (6{,}2x\tan 15^0) = 357{,}2 \text{ cm}$$

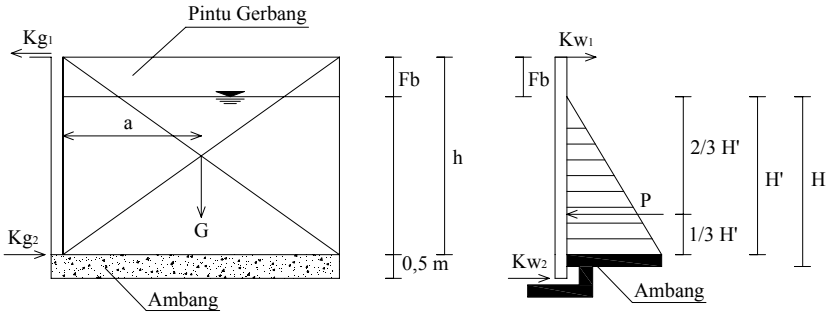
Diambil lebar pintu = 357 cm



Gambar 5.15 Dimensi Pintu Gerbang B

4. Perhitungan Engsel Pintu Gerbang

Dimensi engsel akan dihitung berdasarkan spesifikasi dan beban yang bekerja pada masing-masing pintu gerbang.



Gambar 5.16 Pembebanan pada Engsel Pintu Gerbang B

a. Dimensi Engsel Atas

Lebar gerbang = 3,57 m

Tinggi gerbang = H + Fb

= 5,62 + 1 = 6,62 m

Profil horizontal: IWF 300 x 300 dengan berat 94 kg/m

C 300 x 90 x 10 x 15,5 dengan berat = 43,8 kg/m

Profil vertikal : IWF 300 x 300 dengan berat 94 kg/m

C 300 x 90 x 10 x 15,5 dengan berat = 43,8 kg/m

Tebal plat penutup : 0,005 m, dengan γ baja = 7850 kg/cm^3

- Berat pintu gerbang

Balok vertikal	= [300x90x10x15,5 = 2 x 6,62 x 43,8=	579,912 kg
	= IWF 300 x 300 = 6 x 6,62 x 94 =	3733,680kg
Balok horizontal	= [300x90x10x15,5 = 2 x 3,57 x 43,8=	312,732 kg
	= IWF 300 x 300 = 6 x 3,57 x 94 =	2013,480 kg
Plat	= 2 x 6,62 x 3,57 x 0,005 x 7850 =	1855,222 kg +
	Q =	8495,026 kg
Safety factor = 15 % x Q	=	1274,254 kg +
Berat pintu (G)	=	9769,580 kg

- Keseimbangan akibat berat pintu

a = 0,5 x 3,57 = 1,79 m

G = 9769,580 kg

$$Kg = \frac{(G.a)}{h} = \frac{(9769{,}58x1{,}79)}{6{,}62} = 2641{,}623 \text{ kg}$$

Kg1 = Kg = 2641,623 kg (→)

Kg2 = Kg = 2641,623 kg (←)

- Keseimbangan akibat tekanan hidrostatis

  Fb = tinggi jagaan *(freeboar*d) = 1 m, L = lebar pintu = 3,57 m

  Tinggi ambang = 0,5 m, H = 5,62 m

  H' = H – tinggi ambang = 5,62 – 0,5 = 5,12 m

  P = ½.γ_w.(H')^2 x L = ½.x 1 x 5,12^2 x 3,57 = 46,792 t = 46792 kg

  $$Kw_1 = \frac{P.1/3H'}{h} = \frac{46792x1/3x5{,}12}{6{,}62} = 12063{,}19 \text{ kg } (\rightarrow)$$

  $$Kw_2 = \frac{P\left(F_b + \frac{2}{3}.H'\right)}{h} = \frac{46792\left(1 + \frac{2}{3}x5{,}12\right)}{6{,}62} = 31194{,}66 \text{ kg } (\rightarrow)$$

- Perhitungan diameter pen engsel

  K' $= \sqrt{(Kg_1)^2 + (Kw_1)^2} = \sqrt{(2641{,}623)^2 + (12063{,}19)^2}$ = 12349,03 kg

  y = 10 cm

  M = y x K' = 10 x 12349,03 = 123490,3 kg cm

  $W = \frac{M}{\sigma_{ijin}}$ dan $W = \frac{\pi D^3}{32}$; σ_{ijin} = 1400 kg/cm^2

  $$\frac{123490{,}3}{1400} = \frac{3{,}14xD^3}{32}$$

  D = 9,651 cm

  Diambil diameter pen engsel = 10 cm

- Cek terhadap geser

  $\tau = \frac{4K'}{3\pi R^2} = \frac{4x12349{,}03}{3x3{,}14x5^2}$ = 209,749 kg/cm^2 ≤ 0,58 σ_{ijin}

  = 209,749 kg/cm^2 ≤ 812 kg/cm^2

- Perhitungan diameter stang angker

  K' = ½.K_1.Cos (½ α) = ½ x 12349,03 x Cos (½ 45^0) = 5704,5 kg

  $F = \frac{K'}{\sigma}$; σ_{ijin}= 1400 kg/cm^2

  ¼ π D^2 = $\frac{5704{,}5}{1400}$

  D = 2,2 cm

Diambil stang angker = 2,5 cm = 25 mm

- Perhitungan pelat angker
  - Perhitungan berdasarkan tegangan geser beton
  - Pelat angker dipasang sedalam 20 cm di dalam dinding beton (L = 20)

σ_{bs} = 0,56 √ σ'_{bk} = 0,56√ 225 = 8,4 kg/cm^2

$$F = \frac{K'}{\sigma_{bs}}$$

$$3\ (a \times L) = \frac{5704,5}{8,4}$$

a = 11,32 cm ; (3 adalah banyaknya bidang geser, yaitu: kiri, kanan, bawah)

Diambil ukuran pelat angker = 20 x 20 cm

- Tinjauan terhadap potongan I – I

  M = ½ σ_{bs}.a.(½.a)^2 = 1/8 σ_{bs}.a^3 = 1/8 x 8,4 x 20^3 = 8400 kg cm

  W = 1/6.a.δ^2 dan W = $\frac{M}{\sigma}$; σ_{ijin} = 1400 kg/cm^2

  $$1/6 \times 20 \times \delta^2 = \frac{8400}{1400}$$

  δ = 1,34 cm

- Tinjauan terhadap potongan II – II

  P = ½.a^2. σ_{bs} = ½ x 20^2 x 8,4 = 1680 kg

  M = P.1/3.½.a.√ 2 = 1680 x 1/3 x ½ x 20 x √ 2 = 7919,59 kg cm

  W = 1/6.a.δ^2 dan W = $\frac{M}{\sigma}$; σ_{ijin} = 1400 kg/cm^2

  $$1/6 \times 20 \times \delta^2 = \frac{7919,59}{1400}$$

  δ = 1,303 cm

Diambil tebal pelat angker = 14 mm

b. Dimensi Engsel Bawah

$K_2 = Kw_2 = 31194,66$ kg

G = 9769,580 kg

$F = \frac{G}{\sigma}$; $\sigma_{ijin} = 1400$ kg/cm^2

$F = \frac{9769,58}{1400}$

$\frac{1}{4} \pi D^2 = 2,639$

D = 2,9 cm

Diambil diameter pen engsel = 3 cm = 30 mm

- Cek terhadap geser

  $\tau = \frac{4K'}{3\pi R^2}$ (kg/cm^2) ; $\tau \leq \tau_{ijin} = 0,58\ \sigma_{ijin} = 812$ kg/cm^2

- Perhitungan ulang

  $\tau = \frac{4K'}{3\pi R^2}$

  $812 = \frac{4x31194,66}{3x3,14xR^2}$

  R = 4,03 ≈ 4 cm

Diambil diameter pen engsel = 8 cm = 80 mm (dipakai yang terbesar)

- Perhitungan plat andas

  a. Perhitungan plat 1

     $\sigma'_{bk} = 225$ kg/cm^2

     $F = \frac{G}{\sigma'_{bk}}$

     $a^2 = \frac{9769,580}{225}$

     a = 6,59 cm ; diambil ukuran plat = 20 x 20 cm

  b. Perhitungan plat 2

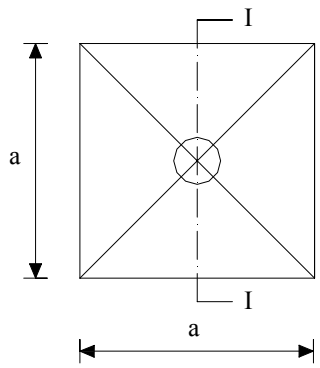
     $\sigma'_{bk} = 225$ kg/cm^2

$$F = \frac{K_2}{\sigma'_{bk}}$$

$$a^2 = \frac{31194,66}{225}$$

a = 11,77 cm ; diambil ukuran plat = 20 x 20 cm

c. Tinjauan terhadap potongan I – I



Gambar 5.17 Potongan I – I Plat Andas Gerbang B

$M = ½\ \sigma_{bs}.a.(½.a)^2 = 1/8\ \sigma_{bs}.a^3 = 1/8 \times 8,4 \times 20^3 = 8400$ kg cm

$W = 1/6.a.\delta^2$ dan $W = \frac{M}{\sigma}$; $\sigma_{ijin} = 1400$ kg/cm^2

$$1/6 \times 20 \times \delta^2 = \frac{8400}{1400}$$

$\delta = 1,342$ cm ; diambil tebal plat angker = 14 mm

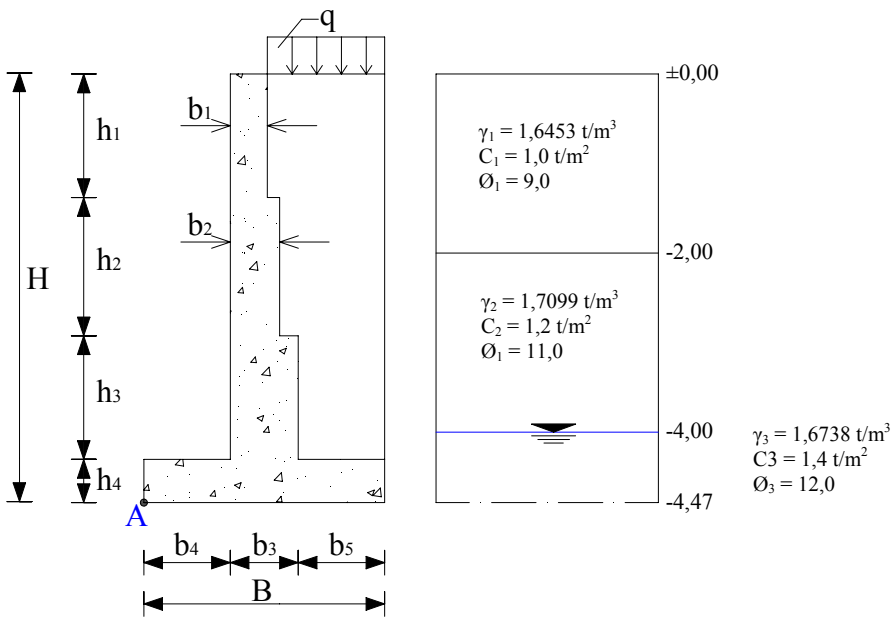
5.6 Perhitungan Dinding Gerbang dan Kamar

5.6.1 Perhitungan Konstruksi Dinding Gerbang A

5.6.1.1 Perhitungan Pembebanan

- Pendimensian Dinding Gerbang

Rencana dimensi dinding gerbang Saluran Pintu Air adalah sebagai berikut :



Gambar 5.18 Pendimensian Dinding Gerbang A

Tabel 5.5 Dimensi Dinding Gerbang A

H4 = 1/4 H – 1/12 H ; diambil 0,35 m	b1 = 0,20 – 0,30 m ; diambil 0,30 m
H = 4,12 + 0,35 = 4,47 m	b2 = 0,35 m
H1 = 1,00 m	b3 = 1/12 H – 1/10 H; diambil 0,40 m
H2 = 1,12 m	b4 = 1/3 H ; diambil 1,50 m
H3 = 2,00 m	b5 = 1,50 m
q = diambil 1,00 t/m^2	B = 0,4 – 0,7 H ; diambil 3,40 m

- Perhitungan Koefisien Tekanan Tanah Aktif

$$Ka = \frac{1-\sin\theta}{1+\sin\theta} \quad \text{atau} \quad Ka = \tan^2(45° - \varphi/2)$$

$$Ka_1 = \frac{1-\sin 9{,}0^0}{1+\sin 9{,}0^0}$$

$$Ka_1 = 0{,}729$$

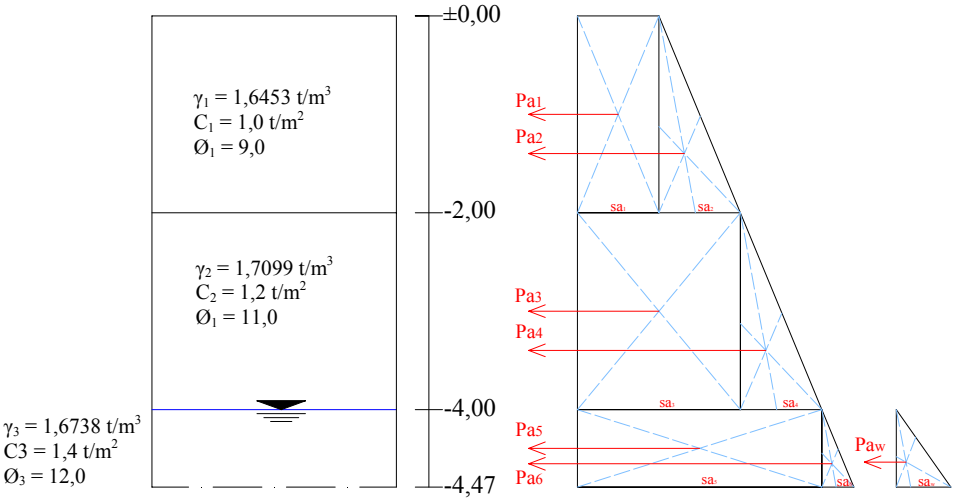
$$Ka_2 = \frac{1-\sin 11{,}0^0}{1+\sin 11{,}0^0}$$

$$Ka_2 = 0{,}679$$

$$Ka_3 = \frac{1 - \sin 12{,}0^0}{1 + \sin 12{,}0^0}$$

$$Ka_3 = 0{,}656$$

- Perhitungan Tegangan Tanah Aktif



Gambar 5.19 Diagram Tegangan Tanah Gerbang A

$\sigma a1 = Ka_1 \times q$

$= 0{,}729 \times 1$

$= 0{,}729\ t/m^2$

$\sigma a2 = (\gamma_1 \times h \times Ka_1) - (2 \times C_1 \times \sqrt{Ka_1})$

$= (1{,}6453 \times 2 \times 0{,}729) - (2 \times 1 \times \sqrt{0{,}729})$

$= 0{,}691\ t/m^2$

$\sigma a3 = \sigma a1 + \sigma a2$

$= 0{,}729 + 0{,}691$

$= 1{,}420\ t/m^2$

$\sigma a4 = (\gamma_2 \times h \times Ka_2) - (2 \times C_2 \times \sqrt{Ka_2})$

$= (1{,}7099 \times 2 \times 0{,}679) - (2 \times 1{,}2 \times \sqrt{0{,}679})$

$= 0{,}344\ t/m^2$

$\sigma a5 = (K_3 \times q) + ((\gamma_1 \times h_1) + (\gamma_2 \times h_2) \times Ka_3) - (2 \times C_3 \times \sqrt{Ka_3})$

$= (0{,}656x1)+((1{,}6453x2)+(1{,}7099x2)x0{,}656)-(2x1{,}4x\sqrt{0{,}656})$

$= 2{,}790\ t/m^2$

$\sigma a6 = \gamma_{sub3} \times h \times Ka_3$

= 0,6738 x 0,47 x 0,656

= 0,208 t/m^2

σaw = γ_w x 0,47 x K_{air}

= 1 x 0,47 x 1

= 0,47 t/m^2

- Perhitungan Tekanan Tanah Aktif (per 1 m lebar)

Pa1 = σa1 x h = 0,729 x 2 = 1,458 t

Pa2 = ½ x σa2 x h = ½ x 0,691 x 2 = 0,691 t

Pa3 = σa3 x h = 1,420 x 2 = 2,840 t

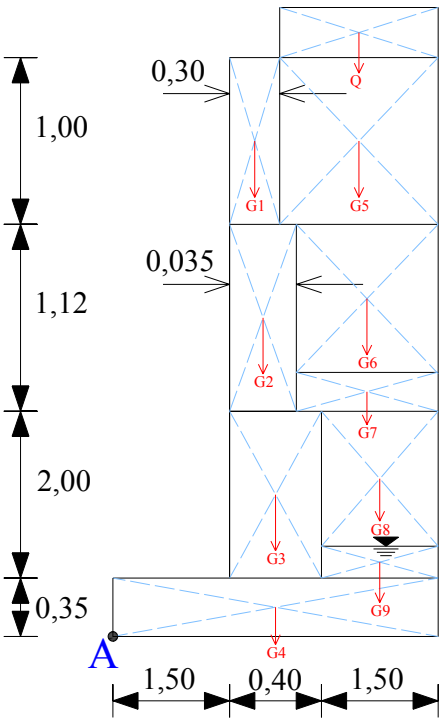
Pa4 = ½ x σa4 x h = ½ x 0,344 x 2 = 0,344 t

Pa5 = σa5 x h = 2,790 x 0,47 = 1,311 t

Pa6 = ½ x σa6 x h = ½ x 0,208 x 0,47 = 0,049 t

Paw = ½ x σaw x 0,47 = ½ x 0,47 x 0,47 = 0,110 t

- Perhitungan Gaya – Gaya Vertikal (per 1 m lebar)



Gambar 5.20 Gaya-Gaya Vertikal pada Dinding Gerbang A

Akibat beban merata :

Q = q (B-b1-b4) = 1 (3,40 – 0,30 – 1,50) = 1,600 t

Akibat berat sendiri struktur :

G1 = b1 x h1 x γ_c = 0,30 x 1,00 x 2,4 = 0,720 t

G2 = b2 x h2 x γ_c = 0,35 x 1,12 x 2,4 = 0,941 t

G3 = b3 x h3 x γ_c = 0,40 x 2,00x 2,4 = 1,920 t

G4 = B x h4 x γ_c = 3,40 x 0,35 x 2,4 = 2,856 t

Akibat berat tanah diatas struktur :

G5 = (b3 + b5 – b1) x h1 x γ_{tanah} = (0,40 + 1,50 – 0,30) x 1,00 x 1,6453
= 2,632 t

G6 = (b3+b5–b2)x(h2 – 0,12)xγ_{tanah} = (0,40 + 1,50 – 0,35) x 1,00 x 1,6453
= 2,550 t

G7 = (b3+b5–b2) x 0,12 x γ_{tanah} = (0,40 + 1,50 – 0,35) x 0,12 x 1,7099
= 0,318 t

G8 = b5 x (4 – h1 – h2) x γ_{tanah} = 1,50 x (4 – 1,00 – 1,12) x 1,7099
= 4,822 t

G9 = b5 x (4,12 – 4) x γ_{sub} = 1,50 x 0,12 x 0,6738 = 0,121 t

Akibat berat air di atas struktur :

G10 = b5 x 0,12 x γ_w = 1,5 x 0,12 x 1 = 0,180 t

- Perhitungan Gaya Gempa

Karena ketinggian dinding penahan tanah < 15 m yaitu 4,47 m maka tidak diperhitungkan gaya gempa.

- Perhitungan Momen terhadap Titik A

Tabel 5.6 Momen Aktif (Horisontal)

P	Gaya (ton)	Lengan (m)	M_{aktif} (tm)
Pa1	1,458	3,470	5,059
Pa2	0,691	3,137	2,168
Pa3	2,840	1,470	4,175

Pa4	0,344	1,137	0,391
Pa5	1,311	0,235	0,308
Pa6	0,049	0,157	0,008
Paw	0,110	0,157	0,017
	Σ P = 6,803		**Σ M_{aktif} = 12,126**

Tabel 5.7 Momen Pasif (Vertikal)

G	Gaya (ton)	Lengan (m)	M_{pasif} (tm)
Q	1,600	2,600	4,160
G1	0,720	1,650	1,188
G2	0,941	1,675	1,576
G3	1,920	1,700	3,264
G4	2,856	1,700	4,855
G5	2,632	2,600	6,843
G6	2,550	2,625	6,694
G7	0,318	2,625	0,835
G8	4,822	2,650	12,778
G9	0,121	2,650	0,321
G10	0,180	2,650	0,477
	Σ G = 18,660		**Σ M_{pasif} = 42,991**

<u>Cek Stabilitas Struktur :</u>

1. Kontrol terhadap guling

$$SF = \frac{\Sigma M_{pasif}}{\Sigma M_{aktif}} \geq 2$$

$$SF = \frac{42,991}{12,126} \geq 2$$

$SF = 3,54 \geq 2$ (aman)

2. Kontrol terhadap geser

$$SF = \frac{\Sigma G \tan\phi + B * C + \Sigma P_{pasif}}{\Sigma P} \geq 1,5$$

$$SF = \frac{18{,}660 \tan 12{,}0^0 + 3{,}4 * 1{,}4 + 0}{6{,}803} \geq 1{,}5$$

$SF = 1{,}283 < 1{,}5$ (perlu tiang pancang)

3. Kontrol terhadap eksentrisitas

   e = ½.B - (ΣMp – ΣMa)/(ΣG)

$$e = 1/2B - \frac{\Sigma M_{pasif} - \Sigma M_{aktif}}{\Sigma G} \leq \frac{1}{6} B$$

$$e = 1/2x3{,}4 - \frac{42{,}991 - 12{,}126}{18{,}66} \leq \frac{1}{6} x3{,}4$$

$$e = 1{,}7 - 1{,}654 \leq 0{,}567$$

$e = 0{,}046 \leq 0{,}567$ (aman)

4. Daya dukung tanah

$$\text{Nc} = \frac{228 + 4{,}3\varphi}{40 - \varphi} = \frac{228 + 4{,}3 * 12{,}0^0}{40 - 12{,}0^0} = 9{,}986$$

$$\text{Nq} = \frac{40 + 5\varphi}{40 - \varphi} = \frac{40 + 5 * 12{,}0^0}{40 - 12{,}0^0} = 3{,}571$$

$$\text{N}\gamma = \frac{6\varphi}{40 - \varphi} = \frac{6 * 12{,}0^0}{40 - 12{,}0^0} = 2{,}571$$

q_{ult} = C.Nc + γ.D.Nq + ½.B.Nγ

= (1,4x9,986) + (0,6738 x 0,35 x 3,571) + ½ x 3,4 x 0,6738 x 2,571)

= 17,768 t/m^2

Daya dukung tanah yang diijinkan ditentukan dengan membagi q_{ult} dengan suatu faktor keamanan (SF) yaitu :

$q_{all} = q_{ult}/SF$ →diambil SF = 3

$q_{all} = 17{,}768/3$

q_{all} = 5,923 t/m^2

5. Tegangan tanah yang terjadi

Kondisi yang harus diperhitungkan adalah pada saat kamar penuh, dengan berat air yang mempengaruhi dinding (pada bagian *toe* sepanjang 1,50 m).

W = H_{air} x b4 x γ_w = ((+16,00) – (+12,88)) x 1,50 x 1 = 4,68 t/m

Tegangan yang terjadi :

$$\sigma_{maks,\min} = \frac{(\Sigma G + W)}{B * L} x(1 \pm \frac{6 * e}{B})$$

$$= \frac{(18,660 + 4,68)}{3,4 * 1} x(1 \pm \frac{6 * 0,046}{3,4})$$

σ_{maks} = 7,422 t/m^2 > q_{all} = 5,923 t/m^2 (perlu tiang pancang)

σ_{min} = 6,307 t/m^2

5.6.1.2 Perhitungan Bagian Tapak Dinding

A. Bagian Tapak Depan (*Toe*)

$$q = q_{toe} - \frac{(\sigma_{maks} - \sigma_{\min})}{B} * X$$

$$q = (\sigma_{\min} + Uplift - \gamma_c * h4) - \frac{(\sigma_{maks} - \sigma_{\min})}{B} * b4$$

Uplift = P = H_{uplift} * γ_w = 0,47 x 1 = 0,47 t/m^2

Sehingga :

$$q = (6,307 + 0,47 - (2,4 \text{ x } 0,35)) - \frac{(7,422 - 6,307)}{3,4} * X$$

= 5,937 – 0,328 (X)

= 5,937 – 0,328 (1,5)

= 5,445 t/m

$$v = \int_0^{1,5} q dx = 5,937 \text{ X} - 0,164 \text{ X}^2 = 8,536 \text{ t}$$

$$M = \int_0^{1,5} v dx = 2,968 \text{ X}^2 - 0,055 \text{ X}^3 = 6,492 \text{ tm}$$

- Perhitungan Tulangan

h4 = 350 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = h4 – d' – Ø/2 = 350 – 50 – 20/2 = 290 mm

Mu = 6,492 tm = 6,492 x 10^7 Nmm

Mn = Mu/φ = 6,492 x 10^7/0,8 = 8,115 x 10^7 Nmm

k = Mn/(b.d^2.R$_l$) → R$_l$ = β$_1$.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 8,115 x 10^7/(1000 x 290^2 x 19,125) = 0,05

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0{,}05)}$ = 0,052

Fmax = β$_1$.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R$_l$/fy = (0,052 x 1000 x 290 x 19,125)/240 = 1196,94 mm^2

ρ = As/(b.d) = 1196,94/(1000 x 290) = 4,127 x 10^{-3}

ρ$_{min}$ = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ$_{maks}$ = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ$_{maks}$ = β$_1$.450/(600 + fy).(R$_l$/fy) = 0,85x450/(600+240)x(19,125/240)
= 0,03629

ρ < ρ$_{min}$ → dipakai ρ$_{min}$

As$_{min}$ = ρ$_{min}$.b.d = 5,83 x 10^{-3} x 1000 x 290 = 1690,7 mm^2

Tulangan pokok terpasang = Ø 20 – 175 (As = 1795 mm^2)

Cek :

ρ = As$_{terpasang}$/b.d = 1795/(1000x290) = 6,189 x 10^{-3}

ρ$_{min}$ < ρ < ρ$_{maks}$

Dicoba tulangan bagi Ø 12

Tulangan bagi = 20% x tulangan pokok = 20% x 1795 = 359 mm^2

Tulangan bagi terpasang = Ø 12 – 250 (As = 452 mm^2)

B. Bagian Tapak Belakang (*Heel*)

Konstruksi pelat yang ditumpu pada ketiga sisinya, dimana beban yang bekerja di atas *heel* mencakup beberapa beban sebagai berikut :

- Beban Merata (q) = 1,000 t/m^2
- Air Tanah = 0,12 x 1 = 0,12 t/m^2
- Tanah-1 = γ_{tanah} x 2 = 1,6453 x 2 = 3,29 t/m^2
- Tanah-2 = γ_{tanah} x 2 = 1,7099 x 2 = 3,419 t/m^2
- Tanah-3 = γ_{sub} x 0,12 = 0,6738 x 0,12 = 0,08 t/m^2
- Berat Sendiri = 2,4 x 1,5 x 0,35 = 1,260 t/m^2

q_{heel} = 1+ 0,12 + 3,29 + 3,419 + 0,08 + 1,26 = 9,169 t/m^2

$q_u = \varphi$ x q_{heel} = 1,2 x 9,169 = 11,003 t/m^2

Panjang Gerbang = (2 x m) + (4 x g) + (4 x t) + L

= (2 x 250) + ((2 x 23,25) + (2 x 20,5)) + (4 x 100) + 350

= 1337,5 cm = 13,375 m

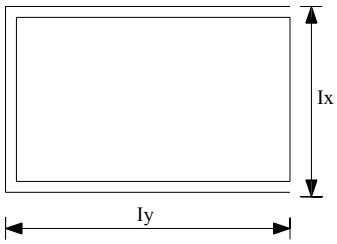
Jarak antar *counterfort* = 0,3 – 0,6 H → H = 4,47 m → diambil = 2,675 m

Iy = panjang *heel* = jarak antar *counterfort* = 2,675 m

Ix = lebar *heel* = 1,5 m

Iy/Ix = 2,675/1,5 = 1,783

Maka momen untuk pelat yang terjepit pada tiga sisi adalah :



$M_{Ix} = 0{,}001.q_u.Ix^2.x$

$M_{Iy} = 0{,}001.q_u.Ix^2.x$

$M_{tx} = -0{,}001.q_u.Ix^2.x$

$M_{ty} = -0{,}001.q_u.Ix^2.x$

$M_{tiy} = ½.M_{Ix}$

Dimana nilai x berturut-turut berdasarkan perbandingan ly/lx = 1,783 adalah 54, 17, 82, dan 53 →(Tabel Hal 26 *Grafik dan Tabel Perhitungan Beton Bertulang, Seri Beton*-4, W.C. Vis, Gideon Kusuma)

M_{Ix} = 0,001 x 11,003 x $1,5^2$x 54 = 1,337 tm

M_{Iy} = 0,001 x 11,003 x $1,5^2$x 17 = 0,421 tm

M_{tx} = -0,001 x 11,003 x $1,5^2$x 82 = -2,030 tm

M_{ty} = -0,001 x 11,003 x $1,5^2$x 53 = -1,312 tm

M_{tiy} = ½ x 1,337 = 0,669 tm

Diambil momen yang maksimum

❖ Tulangan Arah X (Tumpuan dan Lapangan)

Mu = 2,030 tm = 2,030 x 10^7 Nmm

Mn = Mu/φ = 2,030 x 10^7/0,8 = 2,538 x 10^7 Nmm

h4 = 350 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = h4 – d' – Ø/2 = 350 – 50 – 20/2 = 290 mm

k = Mn/(b.d^2.R_1) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 2,538 x 10^7 /(1000 x 290^2 x 19,125) = 0,016

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0{,}016)}$ = 0,016

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R_l/fy = (0,016 x 1000 x 290 x 19,125)/240 = 369,75 mm^2

ρ = As/(b.d) = 369,75/(1000 x 290) = 1,275 x 10^{-5}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ_{maks} = β_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)

= 0,03629

ρ < ρ_{min} → dipakai ρ_{min}

As_{min} = ρ_{min}.b.d = 5,83 x 10^{-3} x 1000 x 290 = 1690,7 mm^2

Tulangan pokok terpasang = Ø 20 – 175 (As = 1795 mm^2)

Cek :

$\rho = As_{terpasang}/b.d = 1795/(1000x290) = 6,189 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

❖ Tulangan Arah Y (Tumpuan dan Lapangan)

Mu = 1,312 tm = $1,312 \times 10^7$ Nmm

Mn = Mu/φ = $1,312 \times 10^7/0,8 = 1,64 \times 10^7$ Nmm

h4 = 350 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = h4 – d' – Ø tulangan arah X - Ø/2 = 350 – 50 – 20 - 20/2 = 270 mm

$k = Mn/(b.d^2.R_l) \rightarrow R_l = \beta_1.f'c = 0,85 \times 22,5 = 19,125$ N/mm^2

$k = 1,64 \times 10^7/(1000 \times 270^2 \times 19,125) = 0,012$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0,012)} = 0,012$

$Fmax = \beta_1.450/(600 + fy) = (0,85 \times 450)/(600 + 240) = 0,455$

F < Fmax →tulangan tunggal *under reinforced*

$As = F.b.d.R_l/fy = (0,012 \times 1000 \times 270 \times 19,125)/240 = 258,188$ mm^2

$\rho = As/(b.d) = 258,188/(1000 \times 270) = 9,562 \times 10^{-4}$

$\rho_{min} = 1,4/ fy = 1,4/240 = 5,83 \times 10^{-3}$

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)$

$= 0,03629$

$\rho < \rho_{min} \rightarrow$ dipakai ρ_{min}

$As_{min} = \rho_{min}.b.d = 5,83 \times 10^{-3} \times 1000 \times 270 = 1574,1$ mm^2

Tulangan pokok terpasang = Ø 20 – 175 (As = 1795 mm^2)

Cek :

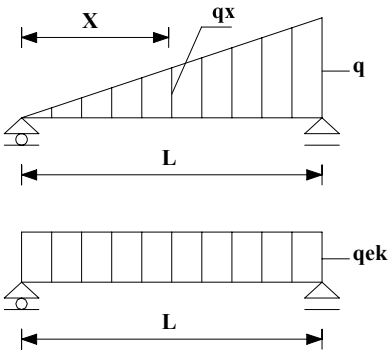
$\rho = As_{terpasang}/b.d = 1795/(1000x270) = 6,648 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

5.6.1.3 Perhitungan Konstruksi Dinding Tegak

A. Perataan beban segitiga untuk pembebanan dinding

Mmaks = q.L^2 / 9√3 untuk x = L / √3



Gambar 5.21 Perataan Beban Segitiga

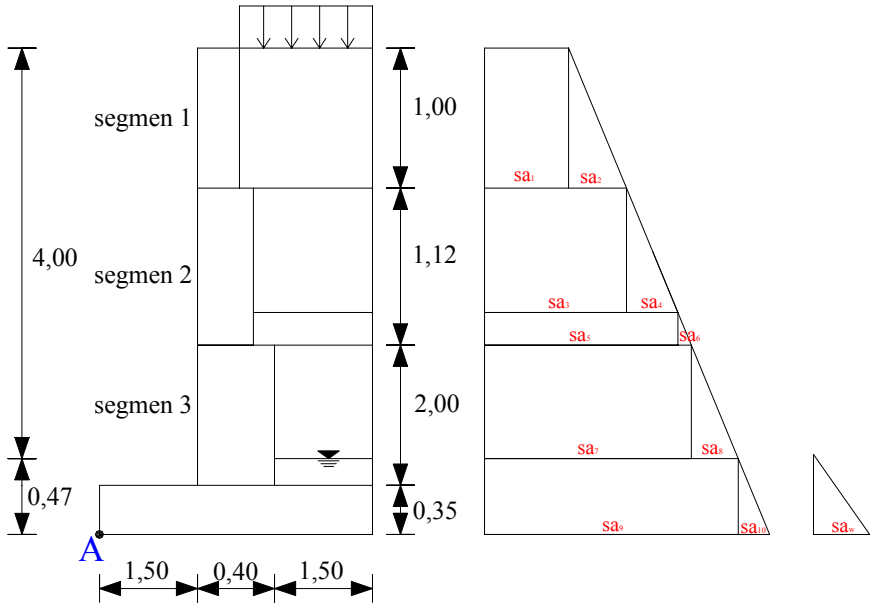
Besarnya beban ekivalen :

$M_{maks} = M_x$

$$M_x = \frac{q_{ek} * x * (L - x)}{2}$$

$$\frac{q * L^2}{9\sqrt{3}} = \frac{q_{ek} * x * (L - x)}{2} \quad \text{untuk } x = L / \sqrt{3}$$

$q_{ek} = 0{,}5246\ q$



Gambar 5.22 Diagram Tegangan Tanah Tiap Segmen Dinding Gerbang A

Segmen 1

$\sigma a1 = Ka_1 \times q$

$= 0{,}729 \times 1$

$= 0{,}729\ t/m^2$

$\sigma a2 = (\gamma_1 \times h \times Ka_1) - (2 \times C_1 \times \sqrt{Ka_1})$

$= (1{,}6453 \times 1 \times 0{,}729) - 0{,}853$

$= 0{,}346\ t/m^2$

$q_{ek} = \sigma a1 + (0{,}5246 \times \sigma a2)$

$= 0{,}729 + (0{,}5246 \times 0{,}346)$

$= 0{,}911\ t/m^2$

Segmen 2

$\sigma a3 = \sigma a1 + \sigma a2$

$= 0{,}729 + 0{,}346$

$= 1{,}075\ t/m^2$

$\sigma a4 = (\gamma_1 \times h \times Ka_1) - (2 \times C_1 \times \sqrt{Ka_1})$

$= (1{,}6453 \times 1 \times 0{,}729) - 0{,}853$

$= 0{,}346\ t/m^2$

$\sigma a5 = \sigma a3 + \sigma a4$

$= 1{,}075 + 0{,}346$

$= 1{,}421\ t/m^2$

$\sigma a6 = (\gamma_2 \times h \times Ka_2) - (2 \times C_1 \times \sqrt{Ka_1})$

$= (1{,}7099 \times 0{,}12 \times 0{,}679) - 0{,}118$

$= 0{,}021\ t/m^2$

$q_{ek} = \sigma a3 + (0{,}5246 \times \sigma a4) + \sigma a5 + (0{,}5246 \times \sigma a6)$

$= 1{,}075 + (0{,}5246 \times 0{,}346) + 1{,}421 + (0{,}5246 \times 0{,}021)$

$= 2{,}689\ t/m^2$

Segmen 3

$\sigma a7 = \sigma a5 + \sigma a6$

$= 1{,}421 + 0{,}021$

$= 1{,}442\ t/m^2$

$\sigma a8 = \gamma_2 \times h \times Ka_2$

$= 1{,}7099 \times 1{,}88 \times 0{,}679$

$= 2,183\ t/m^2$

$\sigma a9 = \sigma a5 + \sigma a6$

$= 1,442 + 2,183$

$= 3,625\ t/m^2$

$\sigma a10 = \gamma_{sub} \times h \times Ka_2$

$= 0,6738 \times 0,12 \times 0,679$

$= 0,055\ t/m^2$

$\sigma aw = \gamma_w \times h \times Kw$

$= 1 \times 0,12 \times 1$

$= 0,12\ t/m^2$

$q_{ek} = \sigma a7 + (0,5246 \times \sigma a8) + \sigma a9 + (0,5246 \times \sigma a10) + (0,5246 \times \sigma aw)$

$= 1,442 + (0,5246 \times 2,183) + 3,625 + (0,5246 \times 0,055) + (0,5246 \times 0,12)$

$= 6,304\ t/m^2$

B. Penulangan Dinding

Segmen 1

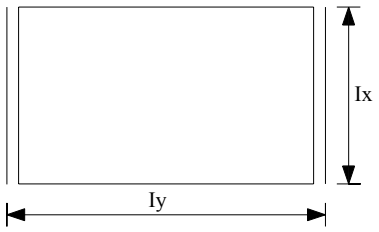
$q = 0,911\ t/m^2$

Ix = h1 = 1,00 m

Iy = jarak antar *counterfort* = 2,675 m

Iy/Ix = 2,675/1,00 = 2,675

Diasumsikan pelat terjepit di kedua sisinya



$M_{Ix} = 0,001.q_u.Ix^2.x$

$M_{Iy} = 0,001.q_u.Ix^2.x$

$M_{ty} = -0,001.q_u.Ix^2.x$

$M_{tiy} = ½.M_{Ix}$

Dimana nilai x berturut-turut berdasarkan perbandingan ly/lx = 2,675 adalah 107; 20,4; dan 112 → (Tabel Hal 26 *Grafik dan Tabel Perhitungan Beton Bertulang, Seri Beton*-4, W.C. Vis, Gideon Kusuma)

M_{Ix} = 0,001 x 0,911 x $1,0^2$x 107 = 0,097 tm

M_{Iy} = 0,001 x 0,911 x $1,0^2$x 20,4 = 0,019 tm

M_{ty} = -0,001 x 0,911 x $1,0^2$x 112 = -0,102 tm

M_{tiy} = ½ x 0,097 = 0,049 tm

Diambil momen yang maksimum

- Tulangan Arah X (Tumpuan dan Lapangan)

Mu = 0,097 tm = 9,7 x 10^5 Nmm

Mn = Mu/φ = 9,7 x 10^5/0,8 = 1,213 x 10^6 Nmm

b1 = 300 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = b1 – d' – Ø/2 = 300 – 50 – 20/2 = 240 mm

k = Mn/(b.d^2.R_l) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 1,213 x 10^6/(1000 x 240^2 x 19,125) = 1,101 x 10^{-3}

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*1,101 \text{ x } 10^{-3})}$ = 1,102 x 10^{-3}

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R_l/fy = (1,102 x 10^{-3} x 1000 x 240 x 19,125)/240 = 21,071 mm^2

ρ = As/(b.d) = 21,071/(1000 x 240) = 8,779 x 10^{-5}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ_{maks} = β_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)

= 0,03629

ρ < ρ_{min} → dipakai ρ_{min}

$As_{min} = \rho_{min}.b.d = 5,83 \times 10^{-3} \times 1000 \times 240 = 1399,2\ mm^2$

Tulangan pokok terpasang = Ø 20 – 200 (As = 1571 mm^2)

Cek :

$\rho = As_{terpasang}/b.d = 1571/(1000x240) = 6,546 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

❖ Tulangan Arah Y (Tumpuan dan Lapangan)

Mu = 0,102 tm = $1,02 \times 10^6$ Nmm

Mn = Mu/φ = $1,02 \times 10^6/0,8 = 1,275 \times 10^6$ Nmm

b1 = 300 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = b1 – d' – Ø tulangan arah X - Ø/2 = 300 – 50 – 20 - 20/2 = 220 mm

$k = Mn/(b.d^2.R_l) \rightarrow R_l = \beta_1.f'c = 0,85 \times 22,5 = 19,125\ N/mm^2$

$k = 1,275 \times 10^6/(1000 \times 220^2 \times 19,125) = 1,377 \times 10^{-3}$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*1,377 \times 10^{-3})} = 1,378 \times 10^{-3}$

$Fmax = \beta_1.450/(600 + fy) = (0,85 \times 450)/(600 + 240) = 0,455$

F < Fmax →tulangan tunggal *under reinforced*

$As = F.b.d.R_l/fy = (1,378x10^{-3} \times 1000 \times 220 \times 19,125)/240 = 24,158\ mm^2$

$\rho = As/(b.d) = 24,158/(1000 \times 220) = 1,098 \times 10^{-4}$

$\rho_{min} = 1,4/ fy = 1,4/240 = 5,83 \times 10^{-3}$

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)$

$= 0,03629$

$\rho < \rho_{min} \rightarrow$ dipakai ρ_{min}

$As_{min} = \rho_{min}.b.d = 5,83 \times 10^{-3} \times 1000 \times 220 = 1282,6\ mm^2$

Tulangan pokok terpasang = Ø 20 – 225 (As = 1396 mm^2)

Cek :

$\rho = As_{terpasang}/b.d = 1396/(1000x220) = 6,345 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

Segmen 2

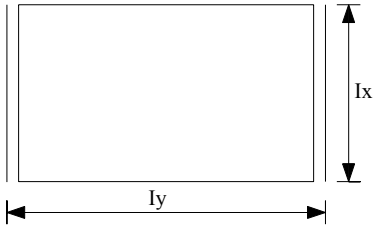
$q = 2,689\ t/m^2$

Ix = h2 = 1,12 m

Iy = jarak antar *counterfort* = 2,675 m

Iy/Ix = 2,675/1,12 = 2,388

Diasumsikan pelat terjepit di kedua sisinya



$M_{Ix} = 0,001.q_u.Ix^2.x$

$M_{Iy} = 0,001.q_u.Ix^2.x$

$M_{ty} = -0,001.q_u.Ix^2.x$

$M_{tiy} = ½.M_{Ix}$

Dimana nilai x berturut-turut berdasarkan perbandingan ly/lx = 2,388 adalah 99,4; 21,6; dan 112 →(Tabel Hal 26 *Grafik dan Tabel Perhitungan Beton Bertulang, Seri Beton*-4, W.C. Vis, Gideon Kusuma)

$M_{Ix} = 0,001 \times 2,689 \times 1,12^2 \times 99,4 = 0,335$ tm

$M_{Iy} = 0,001 \times 2,689 \times 1,12^2 \times 21,6 = 0,073$ tm

$M_{ty} = -0,001 \times 2,689 \times 1,12^2 \times 112 = -0,378$ tm

$M_{tiy} = ½ \times 0,335 = 0,168$ tm

Diambil momen yang maksimum

- Tulangan Arah X (Tumpuan dan Lapangan)

  $Mu = 0,335\ tm = 3,35 \times 10^6$ Nmm

  $Mn = Mu/\varphi = 3,35 \times 10^6/0,8 = 4,188 \times 10^6$ Nmm

  b2 = 350 mm

$f'c = 225\ kg/cm^2 = 22,5\ N/mm^2$

$fy = 2400\ kg/cm^2 = 240\ N/mm^2$

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

$d = b2 – d' – Ø/2 = 350 – 50 – 20/2 = 290$ mm

$k = Mn/(b.d^2.R_l) \rightarrow R_l = \beta_1.f'c = 0,85 \times 22,5 = 19,125\ N/mm^2$

$k = 4,188 \times 10^6/(1000 \times 290^2 \times 19,125) = 2,604 \times 10^{-3}$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2 * 2,604 \times 10^{-3})} = 2,607 \times 10^{-3}$

$Fmax = \beta_1.450/(600 + fy) = (0,85 \times 450)/(600 + 240) = 0,455$

$F < Fmax$ →tulangan tunggal *under reinforced*

$As = F.b.d.R_l/fy = (2,607 \times 10^{-3} \times 1000 \times 290 \times 19,125)/240 = 60,251\ mm^2$

$\rho = As/(b.d) = 60,251/(1000 \times 290) = 2,078 \times 10^{-4}$

$\rho_{min} = 1,4/ fy = 1,4/240 = 5,83 \times 10^{-3}$

$\rho_{maks} = 0,03635$ →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)$

$= 0,03629$

$\rho < \rho_{min} \rightarrow$ dipakai ρ_{min}

$As_{min} = \rho_{min}.b.d = 5,83 \times 10^{-3} \times 1000 \times 290 = 1690,7\ mm^2$

Tulangan pokok terpasang = Ø 20 – 175 ($As = 1795\ mm^2$)

Cek :

$\rho = As_{terpasang}/b.d = 1795/(1000x290) = 6,190 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

❖ Tulangan Arah Y (Tumpuan dan Lapangan)

$Mu = 0,378\ tm = 3,78 \times 10^6\ Nmm$

$Mn = Mu/\varphi = 3,78 \times 10^6/0,8 = 4,725 \times 10^6\ Nmm$

b2 = 350 mm

$f'c = 225\ kg/cm^2 = 22,5\ N/mm^2$

$fy = 2400\ kg/cm^2 = 240\ N/mm^2$

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = b2 – d' – Ø tulangan arah X - Ø/2 = 350 – 50 – 20 - 20/2 = 270 mm

k = Mn/(b.d^2.R_1) → $R_l = \beta_1$.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 4,725 x 10^6/(1000 x 270^2 x 19,125) = 3,389 x 10^{-3}

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*3{,}389 \text{ x } 10^{-3})}$ = 3,394 x 10^{-3}

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R_l/fy = (3,394 x 10^{-3} x 1000 x 270 x 19,125)/240 = 73,041 mm^2

ρ = As/(b.d) = 73,041/(1000 x 270) = 2,705 x 10^{-4}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ_{maks} = β_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)

= 0,03629

ρ < ρ_{min} → dipakai ρ_{min}

As_{min} = ρ_{min}.b.d = 5,83 x 10^{-3} x 1000 x 270 = 1574,1 mm^2

Tulangan pokok terpasang = Ø 20 – 175 (As = 1795 mm^2)

Cek :

ρ = $As_{terpasang}$/b.d = 1795/(1000x270) = 6,648 x 10^{-3}

ρ_{min} < ρ < ρ_{maks} → ok

Segmen 3

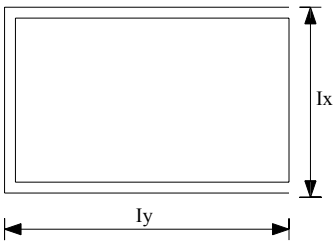
q = 6,304 t/m^2

Ix = h3 = 2,00 m

Iy = jarak antar *counterfort* = 2,675 m

Iy/Ix = 2,675/2 = 1,338

Diasumsikan pelat terjepit di ketiga sisinya :



$M_{Ix} = 0{,}001.q_u.Ix^2.x$

$M_{Iy} = 0{,}001.q_u.Ix^2.x$

$M_{tx} = -0{,}001.q_u.Ix^2.x$

$M_{ty} = -0{,}001.q_u.Ix^2.x$

$M_{tiy} = ½.M_{Ix}$

Dimana nilai x berturut-turut berdasarkan perbandingan ly/lx = 1,338 adalah 41, 20, 73, dan 55 → (Tabel Hal 26 *Grafik dan Tabel Perhitungan Beton Bertulang, Seri Beton*-4, W.C. Vis, Gideon Kusuma)

$M_{Ix} = 0{,}001 \times 6{,}304 \times 2^2 \times 41 = 1{,}034$ tm

$M_{Iy} = 0{,}001 \times 6{,}304 \times 2^2 \times 20 = 0{,}504$ tm

$M_{tx} = -0{,}001 \times 6{,}304 \times 2^2 \times 73 = -1{,}841$ tm

$M_{ty} = -0{,}001 \times 6{,}304 \times 2^2 \times 55 = -1{,}387$ tm

$M_{tiy} = ½ \times 1{,}034 = 0{,}517$ tm

Diambil momen yang maksimum

❖ Tulangan Arah X (Tumpuan dan Lapangan)

Mu = 1,841 tm = $1{,}841 \times 10^7$ Nmm

Mn = Mu/φ = $1{,}841 \times 10^7/0{,}8 = 2{,}301 \times 10^7$ Nmm

b3 = 400 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = b3 – d' – Ø/2 = 400 – 50 – 20/2 = 340 mm

$k = Mn/(b.d^2.R_1)$ → $R_l = \beta_1.f'c = 0{,}85 \times 22{,}5 = 19{,}125$ N/mm^2

$k = 2{,}301 \times 10^7/(1000 \times 340^2 \times 19{,}125) = 0{,}011$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0{,}011)} = 0{,}011$

$F_{max} = \beta_1.450/(600 + fy) = (0{,}85 \times 450)/(600 + 240) = 0{,}455$

$F < F_{max}$ →tulangan tunggal *under reinforced*

$As = F.b.d.R_l/fy = (0{,}011 \times 1000 \times 340 \times 19{,}125)/240 = 298{,}031\ mm^2$

$\rho = As/(b.d) = 298{,}031/(1000 \times 340) = 8{,}765 \times 10^{-4}$

$\rho_{min} = 1{,}4/ fy = 1{,}4/240 = 5{,}83 \times 10^{-3}$

$\rho_{maks} = 0{,}03635$ →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0{,}85x450/(600+240)x(19{,}125/240)$

$= 0{,}03629$

$\rho < \rho_{min}$ → dipakai ρ_{min}

$As_{min} = \rho_{min}.b.d = 5{,}83 \times 10^{-3} \times 1000 \times 340 = 1982{,}2\ mm^2$

Tulangan pokok terpasang = Ø 20 – 150 ($As = 2094\ mm^2$)

Cek :

$\rho = As_{terpasang}/b.d = 2094/(1000x340) = 6{,}159 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks}$ → ok

❖ Tulangan Arah Y (Tumpuan dan Lapangan)

$Mu = 1{,}387\ tm = 1{,}387 \times 10^7\ Nmm$

$Mn = Mu/\varphi = 1{,}387 \times 10^7/0{,}8 = 1{,}734 \times 10^7\ Nmm$

b3 = 400 mm

$f'c = 225\ kg/cm^2 = 22{,}5\ N/mm^2$

$fy = 2400\ kg/cm^2 = 240\ N/mm^2$

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = b3 – d' – Ø tulangan arah X - Ø/2 = 400 – 50 – 20 - 20/2 = 320 mm

$k = Mn/(b.d^2.R_1)$ → $R_l = \beta_1.f'c = 0{,}85 \times 22{,}5 = 19{,}125\ N/mm^2$

$k = 1{,}734 \times 10^7/(1000 \times 320^2 \times 19{,}125) = 8{,}854 \times 10^{-3}$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*8{,}854 \times 10^{-3})} = 8{,}894 \times 10^{-3}$

$F_{max} = \beta_1.450/(600 + fy) = (0{,}85 \times 450)/(600 + 240) = 0{,}455$

$F < F_{max}$ →tulangan tunggal *under reinforced*

$As = F.b.d.R_l/fy = (8{,}894x10^{-3} \times 1000 \times 320 \times 19{,}125)/240 = 226{,}789\ mm^2$

$\rho = As/(b.d) = 226{,}789/(1000 \times 320) = 7{,}087 \times 10^{-4}$

$\rho_{min} = 1{,}4/\ fy = 1{,}4/240 = 5{,}83 \times 10^{-3}$

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0{,}85x450/(600+240)x(\ 19{,}125/240)$

$= 0{,}03629$

$\rho < \rho_{min}$ → dipakai ρ_{min}

$As_{min} = \rho_{min}.b.d = 5{,}83 \times 10^{-3} \times 1000 \times 320 = 1865{,}6\ mm^2$

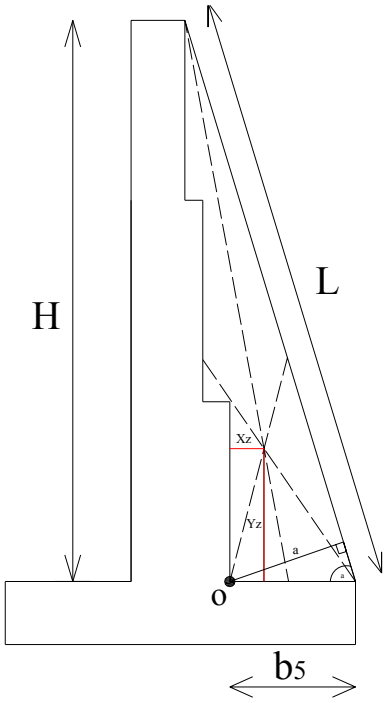
Tulangan pokok terpasang = Ø 20 – 150 (As = 2094 mm^2)

Cek :

$\rho = As_{terpasang}/b.d = 2094/(1000x320) = 6{,}544 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks}$ → ok

5.6.1.4 Perhitungan Bagian Perkuatan Dinding (*Counterfort*)



- Perhitungan titik berat *counterfort* sebagai balok

  Xz = 1/3.b5 = 1/3 x 1,50 = 0,500 m

  Yz = 1/3.H = 1/3 x 4,12 = 1,373 m

- Perhitungan tinggi balok

  Tan α = H/(b5+b3-b1) = 4,12/(1,5+0,4-0,3) = 2,575 → α = 68,776^0

  a = Sin α.b5 = Sin 68,776^0 x 1,50 = 1,398 m

  L = (b5+b3-b1)/Cos α = (1,5+0,4-0,3)/Cos 68,776^0 = 4,419 m

- Pembebanan *counterfort*

Tabel 5.8 Pembebanan *Counterfort*

P	Gaya	Titik Berat		Terhadap Titik O		Lengan	Momen
	(ton)	Yz (m)	Xz (m)	Y (m)	X (m)	(m)	(tm)
Pa1	1,458	1,373	0,500	3,120	-	1,747	2,547
Pa2	0,691	1,373	0,500	2,453	-	1,080	0,746
Pa3	2,840	1,373	0,500	1,120	-	-0,253	-0,719
Pa4	0,344	1,373	0,500	0,787	-	-0,586	-0,202
Pa5	1,311	1,373	0,500	-0,115	-	-1,488	-1,951
Pa6	0,049	1,373	0,500	-0,193	-	-1,566	-0,077
Paw	0,110	1,373	0,500	-0,193	-	-1,566	-0,172
Q	1,600	1,373	0,500	-	0,700	-0,200	-0,320
G1	0,720	1,373	0,500	-	-0,250	0,750	0,540
G2	0,941	1,373	0,500	-	-0,225	0,725	0,682
G3	1,920	1,373	0,500	-	-0,200	0,700	1,344
G4	2,856	1,373	0,500	-	-0,200	0,700	1,999
G5	2,632	1,373	0,500	-	0,700	-0,200	-0,526
G6	2,550	1,373	0,500	-	0,725	-0,225	-0,574
G7	0,318	1,373	0,500	-	0,725	-0,225	-0,072
G8	4,822	1,373	0,500	-	0,750	-0,250	-1,206
G9	0,121	1,373	0,500	-	0,750	-0,250	-0,030
G10	0,180	1,373	0,500	-	0,750	-0,250	-0,045
						Σ M =	1,964

- Perhitungan tulangan lentur

  Mu = 1,964 = 1,964 x 10^7 Nmm

  Mn = Mu/φ = 1,964 x 10^7/0,8 = 2,455 x 10^7 Nmm

  Tebal *counterfort*, syarat ≥ 200 m; diambil b = 300 m

  Jarak antar *counterfort* = 0,3 – 0,6 H → diambil = 2,675 m

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

  fy = 2400 kg/cm^2 = 240 N/mm^2

  Dicoba tulangan pokok Ø 20 mm

  d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

  a = tinggi *counterfort* sebagai balok = 1398 mm

  d = a – d' – Ø/2 = 1398 – 50 – 20/2 = 1338 mm

  k = Mn/(b.d^2.R_1) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

  k = 2,455 x 10^7 /(300 x 1338^2 x 19,125) = 2,390 x 10^{-3}

  F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*2{,}390 \text{ x } 10^{-3})}$ = 2,393 x 10^{-3}

  Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

  F < Fmax →tulangan tunggal *under reinforced*

  As = F.b.d.R_l/fy = (2,393 x 10^{-3} x 300 x 1338 x 19,125)/240 = 76,543 mm^2

  ρ = As/(b.d) = 76,543/(300 x 1338) = 1,907 x 10^{-4}

  ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

  ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

  atau ρ_{maks} = β_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)

  = 0,03629

  ρ < ρ_{min} → dipakai ρ_{min}

  As_{min} = ρ_{min}.b.d = 5,83 x 10^{-3} x 300 x 1338 = 2340,162 mm^2

  Tulangan terpasang = 8 Ø 20 (As = 2513 mm^2)

  (pemasangan 2 baris 4 tulangan)

  Cek :

  ρ = $As_{terpasang}$/b.d = 2513/(300x1338) = 6,261 x 10^{-3}

  ρ_{min} < ρ < ρ_{maks} → ok

- Perhitungan tulangan horisontal

$\sum P$ = Pa1 + Pa2 + Pa3 + Pa4 + Pa5 + Pa6 + Paw

= 1,458 + 0,691 + 2,840 + 0,344 + 1,311 + 0,049 + 0,11

= 6,803 t

= 68030 N

fy = $\sum P$/As

As = $\sum P$/fy

= 68030/240

= 283,458 mm^2 → (untuk 2 sisi)

= 141,729 mm^2 → (untuk 1 sisi)

Pemasangan tulangan untuk tiap 1 meter pias :

As = 141,729 / H = 141,729 / 4,12 = 34,400 mm^2

Tulangan terpasang = Ø 12 - 250 (As = 452 mm^2)

- Perhitungan tulangan vertikal

$\sum G$= Q + G1 + G2 + G3 + G4 + G5 + G6 + G7 + G8 + G9 +G10

= 1,6 + 0,72 + 0,941 + 1,92 + 2,856 + 2,632 + 2,55 + 0,318 + 4,822 +0,121 + 0,18

= 18,66 t

= 186600 N

fy = $\sum G$/As

As = $\sum G$/fy

= 186600/240

= 777,500 mm^2 → (untuk 2 sisi)

= 388,750 mm^2 → (untuk 1 sisi)

Pemasangan tulangan untuk tiap 1 meter pias :

As = 388,750 / b5 = 384,583 / 1,5 = 259,167 mm^2

Tulangan terpasang = Ø 12 - 250 (As = 452 mm^2)

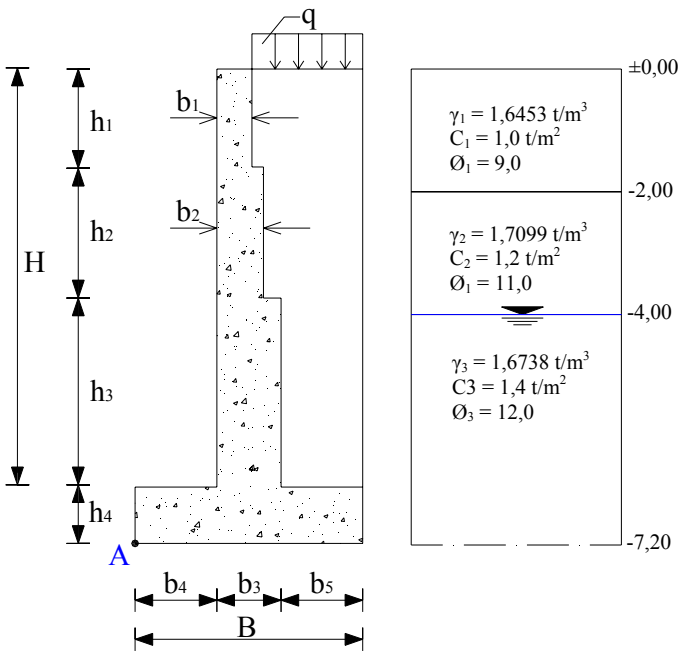
5.6.2 Perhitungan Konstruksi Dinding Gerbang B

Sebagai catatan, konstruksi dinding gerbang B direncanakan sama dengan konstruksi dinding kamar, karena tinggi (elevasi) muka air, elevasi dasar, dan profil tanahnya sama diantara keduanya. Oleh karena itu, hasil perhitungan konstruksi dinding gerbang B juga berlaku untuk konstruksi dinding kamar (nilai-nilainya sama). Dengan demikian, perhitungan konstruksi dinding kamar tidak dituliskan lagi.

5.6.2.1 Perhitungan Pembebanan

- Pendimensian Dinding Gerbang

  Rencana dimensi dinding gerbang Saluran Pintu Air adalah sebagai berikut :



Gambar 5.23 Pendimensian Dinding Gerbang B

Tabel 5.9 Dimensi Dinding Gerbang B

H4 = 1/4 H – 1/12 H ; diambil 0,58 m	b1 = 0,20 – 0,30 m ; diambil 0,30 m
H = 6,62 + 0,58 = 7,20 m	b2 = 0,45 m
H1 = 1,75 m	b3 = 1/12 H – 1/10 H; diambil 0,60 m

H2 = 2,12 m	b4 = 1/3 H ; diambil 2,40 m
H3 = 2,75 m	b5 =2,40 m
q = diambil 1,00 t/m^2	B = 0,4 – 0,7 H ; diambil 5,40 m

- Perhitungan Koefisien Tekanan Tanah Aktif

$$Ka = \frac{1-\sin\theta}{1+\sin\theta} \quad \text{atau} \quad Ka = \tan^2(45° - \varphi/2)$$

$$Ka_1 = \frac{1-\sin 9{,}0^0}{1+\sin 9{,}0^0}$$

$$Ka_1 = 0{,}729$$

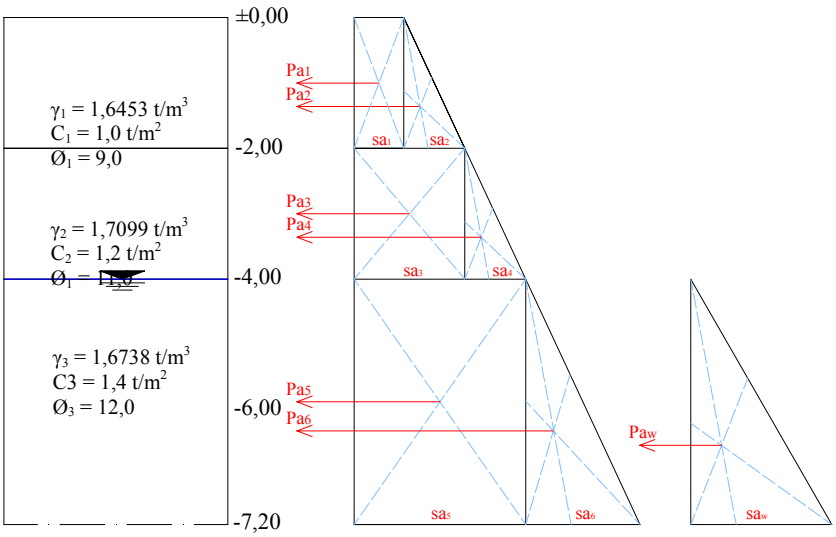
$$Ka_2 = \frac{1-\sin 11{,}0^0}{1+\sin 11{,}0^0}$$

$$Ka_2 = 0{,}679$$

$$Ka_3 = \frac{1-\sin 12{,}0^0}{1+\sin 12{,}0^0}$$

$$Ka_3 = 0{,}656$$

- Perhitungan Tegangan Tanah Aktif



Gambar 5.24 Diagram Tegangan Tanah Gerbang B

$\sigma a1 = Ka_1 \times q$

$= 0{,}729 \times 1$

$= 0{,}729\ t/m^2$

$\sigma a2 = (\gamma_1 \times h \times Ka_1) - (2 \times C_1 \times \sqrt{Ka_1})$

$= (1{,}6453 \times 2 \times 0{,}729) - (2 \times 1 \times \sqrt{0{,}729})$

$= 0{,}691\ t/m^2$

$\sigma a3 = \sigma a1 + \sigma a2$

$= 0{,}729 + 0{,}691$

$= 1{,}420\ t/m^2$

$\sigma a4 = (\gamma_2 \times h \times Ka_2) - (2 \times C_2 \times \sqrt{Ka_2})$

$= (1{,}7099 \times 2 \times 0{,}679) - (2 \times 1{,}2 \times \sqrt{0{,}679})$

$= 0{,}344\ t/m^2$

$\sigma a5 = (K_3 \times q) + ((\gamma_1 \times h_1) + (\gamma_2 \times h_2) \times Ka_3) - (2 \times C_3 \times \sqrt{Ka_3})$

$= (0{,}656x1)+((1{,}6453x2)+(1{,}7099x2)x0{,}656)-(2x1{,}4x\sqrt{0{,}656})$

$= 2{,}790\ t/m^2$

$\sigma a6 = \gamma_{sub3} \times h \times Ka_3$

$= 0{,}6738 \times 3{,}20 \times 0{,}656$

$= 1{,}414\ t/m^2$

$\sigma aw = \gamma_w \times 3{,}20 \times K_{air}$

$= 1 \times 3{,}20 \times 1$

$= 3{,}20\ t/m^2$

- Perhitungan Tekanan Tanah Aktif (per 1 m lebar)

Pa1 $= \sigma a1 \times h = 0{,}729 \times 2 = 1{,}458$ t

Pa2 $= ½ \times \sigma a2 \times h = ½ \times 0{,}691 \times 2 = 0{,}691$ t

Pa3 $= \sigma a3 \times h = 1{,}420 \times 2 = 2{,}840$ t

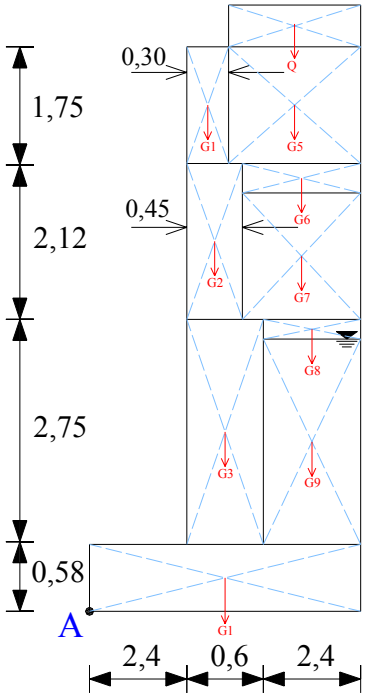
Pa4 $= ½ \times \sigma a4 \times h = ½ \times 0{,}344 \times 2 = 0{,}344$ t

Pa5 $= \sigma a5 \times h = 2{,}790 \times 3{,}20 = 8{,}928$ t

Pa6 $= ½ \times \sigma a6 \times h = ½ \times 1{,}414 \times 3{,}20 = 2{,}262$ t

Paw $= ½ \times \sigma aw \times 0{,}47 = ½ \times 3{,}2 \times 3{,}20 = 5{,}120$ t

➢ Perhitungan Gaya – Gaya Vertikal (per 1 m lebar)



Gambar 5.25 Gaya-Gaya Vertikal pada Dinding Gerbang B

Akibat beban merata :

Q = q (B-b1-b4) = 1 (5,40 – 0,30 – 2,40) = 2,700 t

Akibat berat sendiri struktur :

G1 = b1 x h1 x γ_c = 0,30 x 1,75 x 2,4 = 1,260 t

G2 = b2 x h2 x γ_c = 0,45 x 2,12 x 2,4 = 2,289 t

G3 = b3 x h3 x γ_c = 0,60 x 2,75x 2,4 = 3,960 t

G4 = B x h4 x γ_c = 5,40 x 0,58 x 2,4 = 7,517 t

Akibat berat tanah diatas struktur :

G5 = (b3 + b5 – b1) x h1 x γ_{tanah} = (0,60 + 2,40 – 0,30) x 1,75 x 1,6453
= 7,774 t

G6 = (b3+b5–b2)x(2 – h1)xγ_{tanah} = (0,60 + 2,40 – 0,45) x 0,25 x 1,6453
= 1,049 t

G7 = (b3+b5–b2) x 1,87 x γ_{tanah} = (0,60 + 2,40 – 0,45) x 1,87 x 1,7099
= 8,154 t

G8 = b5 x (4 – h1 – h2) x γ_{tanah} = 2,40 x (4 – 1,75 – 2,12) x 1,7099
= 0,533 t

G9 = b5 x (6,62 – 4) x γ_{sub} = 2,40 x 2,62 x 0,6738 = 4,237 t

Akibat berat air di atas struktur :

G10 = b5 x 2,62 x γ_w = 2,4 x 2,62 x 1 = 6,288 t

- Perhitungan Gaya Gempa

  Karena ketinggian dinding penahan tanah < 15 m yaitu 7,20 m maka tidak diperhitungkan gaya gempa.

- Perhitungan Momen terhadap Titik A

Tabel 5.10 Momen Aktif (Horisontal)

P	Gaya (ton)	Lengan (m)	M_{aktif} (tm)
Pa1	1,458	6,200	9,039
Pa2	0,691	5,867	4,054
Pa3	2,840	4,200	11,928
Pa4	0,344	3,867	1,330
Pa5	8,928	1,600	14,284
Pa6	2,262	1,067	2,414
Paw	5,120	1,067	5,463
	Σ P = 21,643		**Σ M_{aktif} = 48,512**

Tabel 5.11 Momen Pasif (Vertikal)

G	Gaya (ton)	Lengan (m)	M_{pasif} (tm)
Q	2,700	4,050	10,935
G1	1,260	2,550	3,213
G2	2,289	2,625	6,009
G3	3,960	2,700	10,692
G4	7,517	2,700	20,296
G5	7,774	4,050	31,485
G6	1,049	4,125	4,327
G7	8,154	4,125	33,635
G8	0,533	4,200	2,239

G9 G10	4,237 6,288	4,200 4,200	17,795 26,409
	Σ G = 45,761		**Σ M_{pasif} = 167,035**

Cek Stabilitas Struktur :

1. Kontrol terhadap guling

$$SF = \frac{\Sigma M_{pasif}}{\Sigma M_{aktif}} \geq 2$$

$$SF = \frac{167,035}{48,512} \geq 2$$

$SF = 3,44 \geq 2$ (aman)

2. Kontrol terhadap geser

$$SF = \frac{\Sigma G \tan\phi + B * C + \Sigma P_{pasif}}{\Sigma P} \geq 1,5$$

$$SF = \frac{45,761 \tan 12,0^0 + 5,4 * 1,4 + 0}{21,643} \geq 1,5$$

$SF = 0,799 < 1,5$ (perlu tiang pancang)

3. Kontrol terhadap eksentrisitas

e = ½.B - (ΣMp – ΣMa)/(ΣG)

$$e = 1/2B - \frac{\Sigma M_{pasif} - \Sigma M_{aktif}}{\Sigma G} \leq \frac{1}{6}B$$

$$e = 1/2x5,4 - \frac{167,035 - 48,512}{45,761} \leq \frac{1}{6}x5,4$$

$e = 2,7 - 2,59 \leq 0,567$

$e = 0,11 \leq 0,567$ (aman)

4. Daya dukung tanah

$$\text{Nc} = \frac{228 + 4,3\varphi}{40 - \varphi} = \frac{228 + 4,3 * 12,0^0}{40 - 12,0^0} = 9,986$$

$$Nq = \frac{40+5\varphi}{40-\varphi} = \frac{40+5*12,0^0}{40-12,0^0} = 3,571$$

$$N\gamma = \frac{6\varphi}{40-\varphi} = \frac{6*12,0^0}{40-12,0^0} = 2,571$$

q_{ult} = C.Nc + γ.D.Nq + ½.B.Nγ

= (1,4x9,986+(0,6738 x 0,58 x 3,571)+½ x 5,4 x 0,6738 x 2,571)

= 20,053 t/m^2

Daya dukung tanah yang diijinkan ditentukan dengan membagi q_{ult} dengan suatu faktor keamanan (SF) yaitu :

$q_{all} = {q_{ult}}/{SF}$ →diambil SF = 3

$q_{all} = {20,053}/{3}$

q_{all} = 6,684 t/m^2

5. Tegangan tanah yang terjadi

Kondisi yang harus diperhitungkan adalah pada saat kamar penuh, dengan berat air yang mempengaruhi dinding (pada bagian *toe* sepanjang 1,50 m).

W = H_{air} x b4 x γ_w = ((+16,00) – (+10,38)) x 2,40 x 1 = 13,488 t/m

Tegangan yang terjadi :

$$\sigma_{maks,\min} = \frac{(\Sigma G + W)}{B*L} x(1 \pm \frac{6*e}{B})$$

$$= \frac{(45,761+13,488)}{5,4*1} x(1 \pm \frac{6*0,11}{5,4})$$

σ_{maks} = 12,313 t/m^2 > q_{all} = 6,684 t/m^2 (perlu tiang pancang)

σ_{min} = 9,631 t/m^2

5.6.2.2 Perhitungan Bagian Tapak Dinding

A. Bagian Tapak Depan (*Toe*)

$$q = q_{toe} - \frac{(\sigma_{maks} - \sigma_{\min})}{B} * X$$

$q = (\sigma_{min} + Uplift - \gamma_c * h4) - \frac{(\sigma_{maks} - \sigma_{min})}{B} * b4$

$Uplift = P = H_{uplift} * \gamma_w = 3{,}2 \times 1 = 3{,}20\ t/m^2$

Sehingga :

$q = (9{,}631 + 3{,}20 - (2{,}4 \times 0{,}58)) - \frac{(12{,}313 - 9{,}631)}{5{,}4} * X$

$= 11{,}439 - 0{,}497\ (X)$

$= 11{,}439 - 0{,}497\ (2{,}4)$

$= 10{,}246\ t/m$

$v = \int_0^{1,5} q dx = 10{,}246\ X - 0{,}249\ X^2 = 23{,}156\ t$

$M = \int_0^{1,5} v dx = 5{,}123\ X^2 - 0{,}083\ X^3 = 28{,}361\ tm$

- Perhitungan Tulangan

h4 = 580 mm

$f'c = 225\ kg/cm^2 = 22{,}5\ N/mm^2$

$fy = 2400\ kg/cm^2 = 240\ N/mm^2$

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = h4 – d' – Ø/2 = 580 – 50 – 20/2 = 520 mm

$Mu = 28{,}631\ tm = 2{,}863 \times 10^8\ Nmm$

$Mn = Mu/\varphi = 2{,}863 \times 10^8/0{,}8 = 3{,}578 \times 10^8\ Nmm$

$k = Mn/(b.d^2.R_l) \rightarrow R_l = \beta_1.f'c = 0{,}85 \times 22{,}5 = 19{,}125\ N/mm^2$

$k = 3{,}578 \times 10^8/(1000 \times 520^2 \times 19{,}125) = 0{,}069$

$F = 1 - \sqrt{1 - 2k} = 1 - \sqrt{1 - (2 * 0{,}069)} = 0{,}072$

$Fmax = \beta_1.450/(600 + fy) = (0{,}85 \times 450)/(600 + 240) = 0{,}455$

F < Fmax →tulangan tunggal *under reinforced*

$As = F.b.d.R_l/fy = (0{,}072 \times 1000 \times 520 \times 19{,}125)/240 = 2983{,}5\ mm^2$

$\rho = As/(b.d) = 2983{,}5/(1000 \times 520) = 5{,}738 \times 10^{-3}$

$\rho_{min} = 1{,}4/\ fy = 1{,}4/240 = 5{,}83 \times 10^{-3}$

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy)$ = 0,85x450/(600+240)x(19,125/240)

= 0,03629

$\rho < \rho_{min}$ → dipakai ρ_{min}

$As_{min} = \rho_{min}.b.d$ = 5,83 x 10^{-3} x 1000 x 520 = 3031,6 mm^2

Tulangan pokok terpasang = Ø 20 – 100 (As = 3142 mm^2)

Cek :

$\rho = As_{terpasang}/b.d$ = 3142/(1000x520) = 6,042 x 10^{-3}

$\rho_{min} < \rho < \rho_{maks}$

Dicoba tulangan bagi Ø 12

Tulangan bagi = 20% x tulangan pokok = 20% x 3142 = 628,4 mm^2

Tulangan bagi terpasang = Ø 12 – 175 (As = 646 mm^2)

B. Bagian Tapak Belakang (*Heel*)

Konstruksi pelat yang ditumpu pada ketiga sisinya, dimana beban yang bekerja di atas *heel* mencakup beberapa beban sebagai berikut :

- Beban Merata (q) = 1,000 t/m^2
- Air Tanah = (6,62 – 4) x 1 = 2,620 t/m^2
- Tanah-1 = γ_{tanah} x 2 = 1,6453 x 2 = 3,291 t/m^2
- Tanah-2 = γ_{tanah} x 2 = 1,7099 x 2 = 3,419 t/m^2
- Tanah-3 = γ_{sub} x 2,62 = 0,6738 x 2,62 = 1,765 t/m^2
- Berat Sendiri = 2,4 x 2,4 x 0,58 = 3,341 t/m^2

  q_{heel} = 1+ 2,62 + 3,291 + 3,419 + 1,765 + 3,341 = 15,436 t/m^2

  $q_u = \varphi$ x q_{heel} = 1,2 x 15,436 = 18,523 t/m^2

Panjang Gerbang = (2 x m) + (4 x g) + (4 x t) + L

= (2 x 250) + ((2 x 26) + (2 x 20,5)) + (4 x 100) + 357

= 1350 cm = 13,50 m

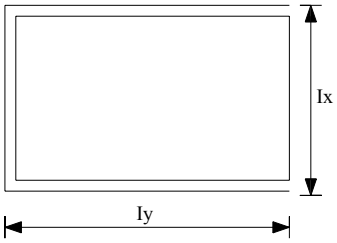
Jarak antar *counterfort* = 0,3 – 0,6 H → H = 7,2 m → diambil = 2,700 m

Iy = panjang *heel* = jarak antar *counterfort* = 2,700 m

Ix = lebar *heel* =2,4 m

Iy/Ix = 2,700/2,4 = 1,125

Maka momen untuk pelat yang terjepit pada tiga sisi adalah :



$M_{Ix} = 0{,}001.q_u.Ix^2.x$

$M_{Iy} = 0{,}001.q_u.Ix^2.x$

$M_{tx} = -0{,}001.q_u.Ix^2.x$

$M_{ty} = -0{,}001.q_u.Ix^2.x$

$M_{tiy} = ½.M_{Ix}$

Dimana nilai x berturut-turut berdasarkan perbandingan ly/lx = 1,125 adalah 33,6; 22,5; 66,3 dan 54,6 →(Tabel Hal 26 *Grafik dan Tabel Perhitungan Beton Bertulang, Seri Beton*-4, W.C. Vis, Gideon Kusuma)

$M_{Ix} = 0{,}001 \times 18{,}523 \times 2{,}4^2 \times 33{,}6 = 3{,}585$ tm

$M_{Iy} = 0{,}001 \times 18{,}523 \times 2{,}4^2 \times 22{,}5 = 2{,}401$ tm

$M_{tx} = -0{,}001 \times 18{,}523 \times 2{,}4^2 \times 66{,}3 = -7{,}074$ tm

$M_{ty} = -0{,}001 \times 18{,}523 \times 2{,}4^2 \times 54{,}6 = -5{,}825$ tm

$M_{tiy} = ½ \times 3{,}585 = 1{,}793$ tm

Diambil momen yang maksimum

❖ Tulangan Arah X (Tumpuan dan Lapangan)

Mu = 7,704 tm = $7{,}704 \times 10^7$ Nmm

Mn = Mu/φ = $7{,}704 \times 10^7/0{,}8 = 9{,}63 \times 10^7$ Nmm

h4 = 580 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = h4 – d' – Ø/2 = 580 – 50 – 20/2 = 520 mm

$k = Mn/(b.d^2.R_1) \rightarrow R_l = \beta_1.f'c = 0{,}85 \times 22{,}5 = 19{,}125$ N/mm^2

$k = 9{,}63 \times 10^7 / (1000 \times 520^2 \times 19{,}125) = 0{,}019$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0{,}019)} = 0{,}019$

$Fmax = \beta_1.450/(600 + fy) = (0{,}85 \times 450)/(600 + 240) = 0{,}455$

$F < Fmax$ →tulangan tunggal *under reinforced*

$As = F.b.d.R_l/fy = (0{,}019 \times 1000 \times 520 \times 19{,}125)/240 = 787{,}313\ mm^2$

$\rho = As/(b.d) = 787{,}313/(1000 \times 520) = 1{,}514 \times 10^{-3}$

$\rho_{min} = 1{,}4/ fy = 1{,}4/240 = 5{,}83 \times 10^{-3}$

$\rho_{maks} = 0{,}03635$ →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0{,}85x450/(600+240)x(19{,}125/240)$

$= 0{,}03629$

$\rho < \rho_{min}$ → dipakai ρ_{min}

$As_{min} = \rho_{min}.b.d = 5{,}83 \times 10^{-3} \times 1000 \times 520 = 3031{,}6\ mm^2$

Tulangan pokok terpasang = Ø 20 – 100 ($As = 3142\ mm^2$)

Cek :

$\rho = As_{terpasang}/b.d = 3142/(1000x520) = 6{,}042 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks}$ → ok

❖ Tulangan Arah Y (Tumpuan dan Lapangan)

$Mu = 5{,}825\ tm = 5{,}825 \times 10^7\ Nmm$

$Mn = Mu/\varphi = 5{,}825 \times 10^7/0{,}8 = 7{,}281 \times 10^7\ Nmm$

h4 = 580 mm

$f'c = 225\ kg/cm^2 = 22{,}5\ N/mm^2$

$fy = 2400\ kg/cm^2 = 240\ N/mm^2$

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = h4 – d' – Ø tulangan arah X - Ø/2 = 580 – 50 – 20 - 20/2 = 500 mm

$k = Mn/(b.d^2.R_l)$ → $R_l = \beta_1.f'c = 0{,}85 \times 22{,}5 = 19{,}125\ N/mm^2$

$k = 7{,}281 \times 10^7/(1000 \times 500^2 \times 19{,}125) = 0{,}015$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0{,}015)} = 0{,}015$

$Fmax = \beta_1.450/(600 + fy) = (0{,}85 \times 450)/(600 + 240) = 0{,}455$

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R_l/fy = (0,015 x 1000 x 500 x 19,125)/240 = 597,656 mm^2

ρ = As/(b.d) = 597,656/(1000 x 500) = 1,195 x 10^{-3}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ_{maks} = β_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)

= 0,03629

ρ < ρ_{min} → dipakai ρ_{min}

As_{min} = ρ_{min}.b.d = 5,83 x 10^{-3} x 1000 x 500 = 2915 mm^2

Tulangan pokok terpasang = Ø 20 – 100 (As = 3142 mm^2)

Cek :

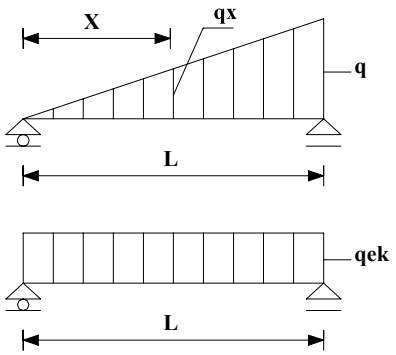
ρ = $As_{terpasang}$/b.d = 3142/(1000x500) = 6,284 x 10^{-3}

ρ_{min} < ρ < ρ_{maks} → ok

5.6.2.3 Perhitungan Konstruksi Dinding Tegak

A. Perataan beban segitiga untuk pembebanan dinding

Mmaks = q.L^2 / 9√3 untuk x = L / √3



Gambar 5.26 Perataan Beban Segitiga

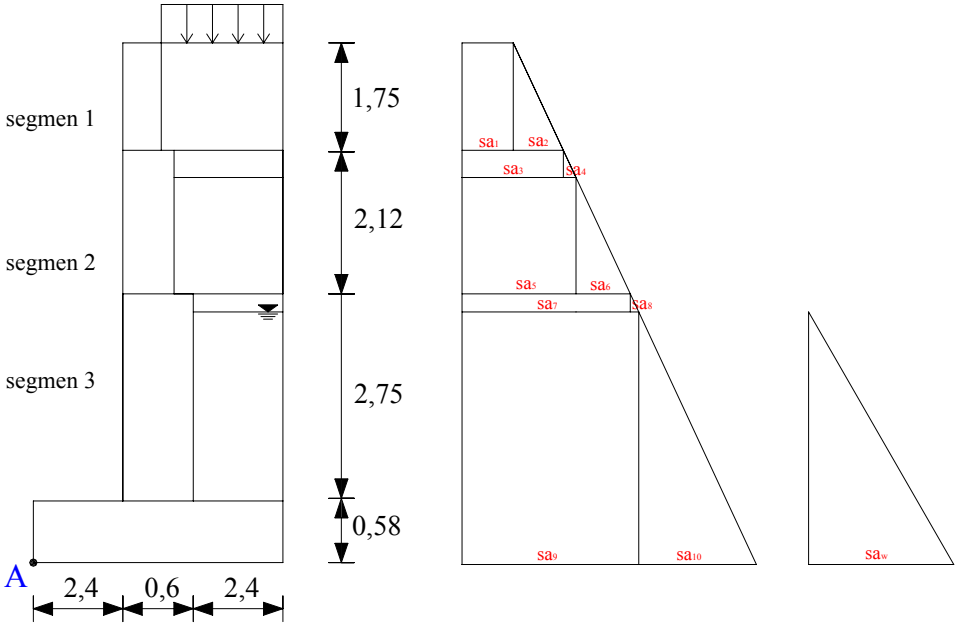
Besarnya beban ekivalen :

M_{maks} = M_x

$$M_x = \frac{q_{ek} * x * (L - x)}{2}$$

$$\frac{q * L^2}{9\sqrt{3}} = \frac{q_{ek} * x * (L - x)}{2} \quad \text{untuk } x = L / \sqrt{3}$$

$q_{ek} = 0{,}5246\ q$



Gambar 5.27 Diagram Tegangan Tanah Tiap Segmen Dinding Gerbang B

Segmen 1

$\sigma a1 = Ka_1 \times q$

$= 0{,}729 \times 1$

$= 0{,}729\ t/m^2$

$\sigma a2 = (\gamma_1 \times h \times Ka_1) - (2 \times C_1 \times \sqrt{Ka_1})$

$= (1{,}6453 \times 1{,}75 \times 0{,}729) - (2 \times 1 \times \sqrt{0{,}729})$

$= 0{,}391\ t/m^2$

$q_{ek} = \sigma a1 + (0{,}5246 \times \sigma a2)$

$= 0{,}729 + (0{,}5246 \times 0{,}391)$

$= 0{,}934\ t/m^2$

Segmen 2

$\sigma a3 = \sigma a1 + \sigma a2$

$= 0{,}729 + 0{,}391$

$= 1{,}120\ t/m^2$

$\sigma a4 = (\gamma_1 \times h \times Ka_1)$

$= (1,6453 \times 0,25 \times 0,729)$

$= 0,299\ t/m^2$

$\sigma a5 = \sigma a3 + \sigma a4$

$= 1,120 + 0,299$

$= 1,419\ t/m^2$

$\sigma a6 = (\gamma_2 x\ h \times Ka_2) - (2 \times C_1 \times \sqrt{Ka_1})$

$= (1,7099 \times 1,87 \times 0,679) – (2 \times 1,2 \times \sqrt{0,679})$

$= 0,193\ t/m^2$

$q_{ek} = \sigma a3 + (0,5246 \times \sigma a4) + \sigma a5 + (0,5246 \times \sigma a6)$

$= 1,120 + (0,5246 \times 0,299) + 1,419 + (0,5246 \times 0,193)$

$= 2,797\ t/m^2$

Segmen 3

$\sigma a7 = \sigma a5 + \sigma a6$

$= 1,419 + 0,193$

$= 1,612\ t/m^2$

$\sigma a8 = \gamma_2 x\ h \times Ka_2$

$= 1,7099 \times 0,13 \times 0,679$

$= 0,151\ t/m^2$

$\sigma a9 = (K_3 \times q) + ((\gamma_1 \times h_1) + (\gamma_2 \times h_2) \times Ka_3) – (2 \times C_3 \times \sqrt{Ka_3})$

$= (0,656 \times 1) + ((1,6453x2) + (1,7099x2) \times 0,6738) – (2 \times 1,4 \times \sqrt{0,656})$

$= 2,790\ t/m^2$

$\sigma a10 = \gamma_{sub3} \times h \times Ka_3$

$= 0,6738 \times 3,2 \times 0,656$

$= 1,414\ t/m^2$

$\sigma aw = \gamma_w \times h \times Kw$

$= 1 \times 3,2 \times 1$

$= 3,2\ t/m^2$

$q_{ek} = \sigma a7 + (0,5246 \times \sigma a8) + \sigma a9 + (0,5246 \times \sigma a10) + (0,5246 \times \sigma aw)$

$= 1,612 + (0,5246 \times 0,151) + 2,790 + (0,5246 \times 1,414) + (0,5246 \times 3,2)$

$= 6,902\ t/m^2$

B. Penulangan Dinding

Segmen 1

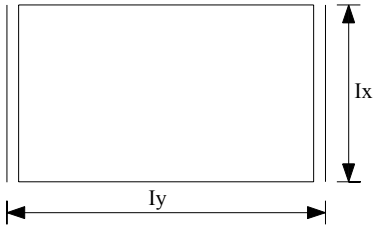
q = 0,934 t/m^2

Ix = h1 = 1,75 m

Iy = jarak antar *counterfort* = 2,700 m

Iy/Ix = 2,700/1,75 = 1,543

Diasumsikan pelat terjepit di kedua sisinya



$M_{Ix} = 0{,}001.q_u.Ix^2.x$

$M_{Iy} = 0{,}001.q_u.Ix^2.x$

$M_{ty} = -0{,}001.q_u.Ix^2.x$

$M_{tiy} = ½.M_{Ix}$

Dimana nilai x berturut-turut berdasarkan perbandingan ly/lx = 1,543 adalah 59; 29,9; dan 102,7 →(Tabel Hal 26 *Grafik dan Tabel Perhitungan Beton Bertulang, Seri Beton*-4, W.C. Vis, Gideon Kusuma)

M_{Ix} = 0,001 x 0,934 x 1,75^{2}x 59 = 0,169 tm

M_{Iy} = 0,001 x 0,934 x 1,75^{2}x 29,9 = 0,086 tm

M_{ty} = -0,001 x 0,934 x 1,75^{2}x 102,7 = -0,294 tm

M_{tiy} = ½ x 0,169 = 0,085 tm

Diambil momen yang maksimum

❖ Tulangan Arah X (Tumpuan dan Lapangan)

Mu = 0,169 tm = 1,69 x 10^6 Nmm

Mn = Mu/φ = 1,69 x 10^6/0,8 = 2,113 x 10^6 Nmm

b1 = 300 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = b1 – d' – Ø/2 = 300 – 50 – 20/2 = 240 mm

k = Mn/(b.d^2.R_1) → $R_l = \beta_1$.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 2,113 x 10^6/(1000 x 240^2 x 19,125) = 1,918 x 10^{-3}

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*1{,}918 \text{ x } 10^{-3})}$ = 1,919 x 10^{-3}

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R_l/fy = (1,919x 10^{-3} x 1000 x 240 x 19,125)/240 = 36,700 mm^2

ρ = As/(b.d) = 36,700/(1000 x 240) = 1,529 x 10^{-4}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ_{maks} = β_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)
= 0,03629

ρ < ρ_{min} → dipakai ρ_{min}

As_{min} = ρ_{min}.b.d = 5,83 x 10^{-3} x 1000 x 240 = 1399,2 mm^2

Tulangan pokok terpasang = Ø 20 – 200 (As = 1571 mm^2)

Cek :

ρ = $As_{terpasang}$/b.d = 1571/(1000x240) = 6,546 x 10^{-3}

ρ_{min} < ρ < ρ_{maks} → ok

❖ Tulangan Arah Y (Tumpuan dan Lapangan)

Mu = 0,294 tm = 2,94 x 10^6 Nmm

Mn = Mu/φ = 2,94 x 10^6/0,8 = 3,675 x 10^6 Nmm

b1 = 300 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = b1 – d' – Ø tulangan arah X - Ø/2 = 300 – 50 – 20 - 20/2 = 220 mm

$k = Mn/(b.d^2.R_1) \rightarrow R_l = \beta_1.f'c = 0,85 \times 22,5 = 19,125\ N/mm^2$

$k = 3,675 \times 10^6/(1000 \times 220^2 \times 19,125) = 3,970 \times 10^{-3}$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*3,970 \times 10^{-3})} = 3,978 \times 10^{-3}$

$Fmax = \beta_1.450/(600 + fy) = (0,85 \times 450)/(600 + 240) = 0,455$

$F < Fmax$ →tulangan tunggal *under reinforced*

$As = F.b.d.R_l/fy = (3,978 \times 10^{-3} \times 1000 \times 220 \times 19,125)/240 = 69,741\ mm^2$

$\rho = As/(b.d) = 69,741/(1000 \times 220) = 3,170 \times 10^{-4}$

$\rho_{min} = 1,4/ fy = 1,4/240 = 5,83 \times 10^{-3}$

$\rho_{maks} = 0,03635$ →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0,85 \times 450/(600+240) \times (19,125/240)$

$= 0,03629$

$\rho < \rho_{min}$ → dipakai ρ_{min}

$As_{min} = \rho_{min}.b.d = 5,83 \times 10^{-3} \times 1000 \times 220 = 1282,6\ mm^2$

Tulangan pokok terpasang = Ø 20 – 225 ($As = 1396\ mm^2$)

Cek :

$\rho = As_{terpasang}/b.d = 1396/(1000 \times 220) = 6,345 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks}$ → ok

Segmen 2

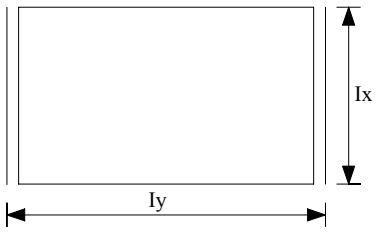
$q = 2,797\ t/m^2$

Ix = h2 = 2,12 m

Iy = jarak antar *counterfort* = 2,700 m

Iy/Ix = 2,700/2,12 = 1,274

Diasumsikan pelat terjepit di kedua sisinya



$M_{Ix} = 0{,}001.q_u.Ix^2.x$

$M_{Iy} = 0{,}001.q_u.Ix^2.x$

$M_{ty} = -0{,}001.q_u.Ix^2.x$

$M_{tiy} = ½.M_{Ix}$

Dimana nilai x berturut-turut berdasarkan perbandingan ly/lx = 1,274 adalah 40,8; 32,6; dan 89,4 →(Tabel Hal 26 *Grafik dan Tabel Perhitungan Beton Bertulang, Seri Beton*-4, W.C. Vis, Gideon Kusuma)

$M_{Ix} = 0{,}001 \times 2{,}797 \times 2{,}12^2 \times 40{,}8 = 0{,}513$ tm

$M_{Iy} = 0{,}001 \times 2{,}797 \times 2{,}12^2 \times 32{,}6 = 0{,}409$ tm

$M_{ty} = -0{,}001 \times 2{,}797 \times 2{,}12^2 \times 89{,}4 = -1{,}124$ tm

$M_{tiy} = ½ \times 0{,}513 = 0{,}257$ tm

Diambil momen yang maksimum

- Tulangan Arah X (Tumpuan dan Lapangan)

Mu = 0,513 tm = $5{,}13 \times 10^6$ Nmm

Mn = Mu/φ = $5{,}13 \times 10^6/0{,}8 = 6{,}413 \times 10^6$ Nmm

b2 = 450 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = b2 – d' – Ø/2 = 450 – 50 – 20/2 = 390 mm

$k = Mn/(b.d^2.R_1) \rightarrow R_l = \beta_1.f'c = 0{,}85 \times 22{,}5 = 19{,}125$ N/mm^2

$k = 6{,}413 \times 10^6/(1000 \times 390^2 \times 19{,}125) = 2{,}205 \times 10^{-3}$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2 * 2{,}205 \times 10^{-3})} = 2{,}207 \times 10^{-3}$

$Fmax = \beta_1.450/(600 + fy) = (0{,}85 \times 450)/(600 + 240) = 0{,}455$

F < Fmax →tulangan tunggal *under reinforced*

$As = F.b.d.R_l/fy = (2{,}207 \times 10^{-3} \times 1000 \times 390 \times 19{,}125)/240 = 68{,}591$ mm^2

$\rho = As/(b.d) = 68{,}591/(1000 \times 390) = 1{,}759 \times 10^{-4}$

$\rho_{min} = 1{,}4/ fy = 1{,}4/240 = 5{,}83 \times 10^{-3}$

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy)$ = 0,85x450/(600+240)x(19,125/240)

= 0,03629

$\rho < \rho_{min}$ → dipakai ρ_{min}

$As_{min} = \rho_{min}.b.d$ = 5,83 x 10^{-3} x 1000 x 390 = 2273,7 mm^2

Tulangan pokok terpasang = Ø 20 – 125 (As = 2513 mm^2)

Cek :

$\rho = As_{terpasang}/b.d$ = 2513/(1000x390) = 6,444 x 10^{-3}

$\rho_{min} < \rho < \rho_{maks}$ → ok

❖ Tulangan Arah Y (Tumpuan dan Lapangan)

Mu = 1,124 tm = 1,124 x 10^7 Nmm

Mn = Mu/φ = 1,124 x 10^7/0,8 = 1,405 x 10^7 Nmm

b2 = 450 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = b2 – d' – Ø tulangan arah X - Ø/2 = 450 – 50 – 20 - 20/2 = 370 mm

$k = Mn/(b.d^2.R_l)$ → $R_l = \beta_1.f'c$ = 0,85 x 22,5 = 19,125 N/mm^2

k = 1,405 x 10^7/(1000 x 370^2 x 19,125) = 5,366 x 10^{-3}

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*5{,}366 \text{ x } 10^{-3})}$ = 5,380 x 10^{-3}

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = $F.b.d.R_l/fy$ = (5,380x10^{-3} x 1000 x 370 x 19,125)/240 = 158,626 mm^2

ρ = As/(b.d) = 158,626/(1000 x 370) = 4,287 x 10^{-4}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy)$ = 0,85x450/(600+240)x(19,125/240)

= 0,03629

$\rho < \rho_{min}$ → dipakai ρ_{min}

$As_{min} = \rho_{min}.b.d = 5,83 \times 10^{-3} \times 1000 \times 370 = 2157,1\ mm^2$

Tulangan pokok terpasang = Ø 20 – 125 ($As = 2513\ mm^2$)

Cek :

$\rho = As_{terpasang}/b.d = 2513/(1000x370) = 6,792 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks}$ → ok

Segmen 3

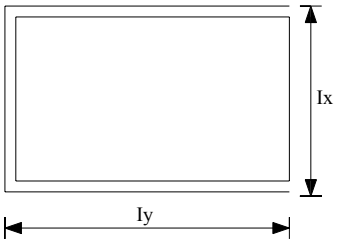
$q = 6,902\ t/m^2$

Ix = h3 = 2,75 m

Iy = jarak antar *counterfort* = 2,700 m

Iy/Ix = 2,700/2,75 = 0,982

Diasumsikan pelat terjepit di ketiga sisinya :



$M_{Ix} = 0,001.q_u.Ix^2.x$

$M_{Iy} = 0,001.q_u.Ix^2.x$

$M_{tx} = -0,001.q_u.Ix^2.x$

$M_{ty} = -0,001.q_u.Ix^2.x$

$M_{tiy} = ½.M_{Ix}$

Dimana nilai x berturut-turut berdasarkan perbandingan ly/lx = 0,982 adalah 28, 25, 60, dan 54 →(Tabel Hal 26 *Grafik dan Tabel Perhitungan Beton Bertulang, Seri Beton*-4, W.C. Vis, Gideon Kusuma)

$M_{Ix} = 0,001 \times 6,902 \times 2,75^2 \times 28 = 1,461$ tm

$M_{Iy} = 0,001 \times 6,902 \times 2,75^2 \times 25 = 1,305$ tm

$M_{tx} = -0,001 \times 6,902 \times 2,75^2 \times 60 = -3,132$ tm

$M_{ty} = -0,001 \times 6,902 \times 2,75^2 \times 54 = -2,819$ tm

M_{tiy} = ½ x 1,461 = 0,731 tm

Diambil momen yang maksimum

- Tulangan Arah X (Tumpuan dan Lapangan)

Mu = 3,132 tm = 3,132 x 10^7 Nmm

Mn = Mu/φ = 3,132 x 10^7/0,8 = 3,915 x 10^7 Nmm

b3 = 600 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = b3 – d' – Ø/2 = 600 – 50 – 20/2 = 540 mm

k = Mn/(b.d^2.R_l) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 3,915 x 10^7/(1000 x 540^2 x 19,125) = 7,020 x 10^{-3}

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*7{,}020 \times 10^{-3})}$ = 7,045 x 10^{-3}

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R_l/fy = (7,045x10^{-3}x 1000 x 540 x 19,125)/240 = 303,147 mm^2

ρ = As/(b.d) = 303,147/(1000 x 540) = 5,614 x 10^{-4}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ_{maks} = β_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)

= 0,03629

ρ < ρ_{min} → dipakai ρ_{min}

As_{min} = ρ_{min}.b.d = 5,83 x 10^{-3} x 1000 x 540 = 3148,2 mm^2

Tulangan pokok terpasang = Ø 20 – 75 (As = 4189 mm^2)

Cek :

ρ = $As_{terpasang}$/b.d = 4189/(1000x540) = 7,757 x 10^{-3}

ρ_{min} < ρ < ρ_{maks} → ok

❖ Tulangan Arah Y (Tumpuan dan Lapangan)

Mu = 2,819 tm = 2,819 x 10^7 Nmm

Mn = Mu/φ = 2,819 x 10^7/0,8 = 3,524 x 10^7 Nmm

b3 = 600 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = b3 – d' – Ø tulangan arah X - Ø/2 = 600 – 50 – 20 - 20/2 = 520 mm

k = Mn/(b.d^2.R_1) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 3,524 x 10^7/(1000 x 520^2 x 19,125) = 6,814 x 10^{-3}

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*6{,}814 \text{ x } 10^{-3})}$ = 6,837 x 10^{-3}

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R_l/fy = (6,837x10^{-3} x 1000 x 520 x 19,125)/240 = 283,308 mm^2

ρ = As/(b.d) = 283,308/(1000 x 520) = 5,448 x 10^{-4}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ_{maks} = β_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)

= 0,03629

ρ < ρ_{min} → dipakai ρ_{min}

As_{min} = ρ_{min}.b.d = 5,83 x 10^{-3} x 1000 x 520 = 3031,6 mm^2

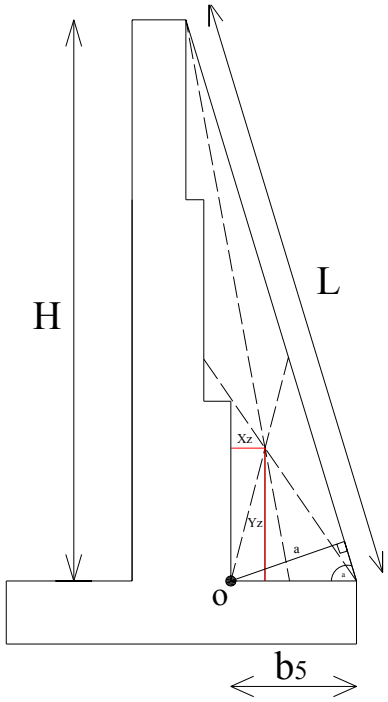
Tulangan pokok terpasang = Ø 20 – 100 (As = 3142 mm^2)

Cek :

ρ = $As_{terpasang}$/b.d = 3142/(1000x520) = 6,042 x 10^{-3}

ρ_{min} < ρ < ρ_{maks} → ok

5.6.2.4 Perhitungan Bagian Perkuatan Dinding (*Counterfort*)



- Perhitungan titik berat *counterfort* sebagai balok

  Xz = 1/3.b5 = 1/3 x 2,40 = 0,800 m

  Yz = 1/3.H = 1/3 x 6,62 = 2,207 m

- Perhitungan tinggi balok

  Tan α = H/(b5+b3-b1) =6,62/(2,4+0,6-0,3) = 2,452 → α = $67{,}813^0$

  a = Sin α.b5 = Sin $67{,}813^0$ x 2,40 = 2,222 m

  L = (b5+b3-b1)/Cos α = (2,4+0,6+0,3)/Cos $67{,}813^0$ = 7,149 m

- Pembebanan *counterfort*

Tabel 5.12 Pembebanan *Counterfort*

P	Gaya	Titik Berat		Terhadap Titik O		Lengan	Momen
	(ton)	Yz (m)	Xz (m)	Y (m)	X (m)	(m)	(tm)
Pa1	1,458	2,207	0,800	5,620	-	3,413	4,976
Pa2	0,691	2,207	0,800	5,287	-	3,080	2,128
Pa3	2,840	2,207	0,800	3,620	-	1,413	4,013
Pa4	0,344	2,207	0,800	3,287	-	1,080	0,372
Pa5	8,928	2,207	0,800	1,020	-	-1,187	-10,598

Pa6	2,262	2,207	0,800	0,487	-	-1,720	-3,891
Paw	5,120	2,207	0,800	0,487	-	-1,720	-8,806
Q	2,700	2,207	0,800	-	1,050	-0,250	-0,675
G1	1,260	2,207	0,800	-	-0,450	1,250	1,575
G2	2,289	2,207	0,800	-	-0,375	1,175	2,689
G3	3,960	2,207	0,800	-	-0,300	1,100	4,356
G4	7,517	2,207	0,800	-	-0,300	1,100	8,269
G5	7,774	2,207	0,800	-	1,050	-0,250	-1,944
G6	1,049	2,207	0,800	-	1,125	-0,325	-0,341
G7	8,154	2,207	0,800	-	1,125	-0,325	-2,650
G8	0,533	2,207	0,800	-	1,200	-0,400	-0,213
G9	4,237	2,207	0,800	-	1,200	-0,400	-1,695
G10	6,288	2,207	0,800	-	1,200	-0,400	-2,515
						Σ M =	-4,950

- Perhitungan tulangan lentur

  Mu = 4,950 = 4,950 x 10^7 Nmm

  Mn = Mu/φ = 4,950 x 10^7/0,8 = 6,188 x 10^7 Nmm

  Tebal *counterfort*, syarat ≥ 200 m; diambil b = 300 m

  Jarak antar *counterfort* = 0,3 – 0,6 H → diambil = 2,700 m

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

  fy = 2400 kg/cm^2 = 240 N/mm^2

  Dicoba tulangan pokok Ø 20 mm

  d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

  a = tinggi *counterfort* sebagai balok = 2222 mm

  d = a – d' – Ø/2 = 2222 – 50 – 20/2 = 2162 mm

  k = Mn/(b.d^2.R_1) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

  k = 6,188 x 10^7 /(300 x 2162^2 x 19,125) = 2,307 x 10^{-3}

  F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*2{,}307 \times 10^{-3})}$ = 2,310 x 10^{-3}

  Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

  F < Fmax →tulangan tunggal *under reinforced*

$As = F.b.d.R_l/fy = (2{,}310 \times 10^{-3} \times 300 \times 2162 \times 19{,}125)/240 = 119{,}393\ mm^2$

$\rho = As/(b.d) = 119{,}393/(300 \times 2162) = 1{,}841 \times 10^{-4}$

$\rho_{min} = 1{,}4/\ fy = 1{,}4/240 = 5{,}83 \times 10^{-3}$

$\rho_{maks} = 0{,}03635$ →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0{,}85x450/(600+240)x(\ 19{,}125/240)$

$= 0{,}03629$

$\rho < \rho_{min}$ → dipakai ρ_{min}

$As_{min} = \rho_{min}.b.d = 5{,}83 \times 10^{-3} \times 300 \times 2162 = 3781{,}338\ mm^2$

Tulangan terpasang = 15 Ø 20 (As = 4712 mm^2)

(pemasangan 3 baris 5 tulangan)

Cek :

$\rho = As_{terpasang}/b.d = 4712/(300x2162) = 7{,}265 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks}$ → ok

- Perhitungan tulangan horisontal

$\sum P = Pa1 + Pa2 + Pa3 + Pa4 + Pa5 + Pa6 + Paw$

$= 1{,}458 + 0{,}691 + 2{,}840 + 0{,}344 + 8{,}928 + 2{,}262 + 5{,}12$

$= 21{,}643$ t

$= 216430$ N

$fy = \sum P/As$

$As = \sum P/fy$

$= 216430/240$

$= 901{,}792\ mm^2$ → (untuk 2 sisi)

$= 450{,}896\ mm^2$ → (untuk 1 sisi)

Pemasangan tulangan untuk tiap 1 meter pias :

$As = 450{,}896 / H = 450{,}896 / 6{,}62 = 68{,}111\ mm^2$

Tulangan terpasang = Ø 12 - 250 (As = 452 mm^2)

- Perhitungan tulangan vertikal

$\sum G = Q + G1 + G2 + G3 + G4 + G5 + G6 + G7 + G8 + G9 + G10$

$= 2{,}7 + 1{,}26 + 2{,}289 + 3{,}96 + 7{,}517 + 7{,}774 + 1{,}049 + 8{,}154 + 0{,}533$

+ 4,237 + 6,288

= 45,761 t

= 457610 N

fy = ∑G/As

As = ∑G/fy

= 457610/240

= 1906,708 mm^2 → (untuk 2 sisi)

= 953,354 mm^2 → (untuk 1 sisi)

Pemasangan tulangan untuk tiap 1 meter pias :

As = 953,354 / b5 = 953,354 / 2,4 = 397,231 mm^2

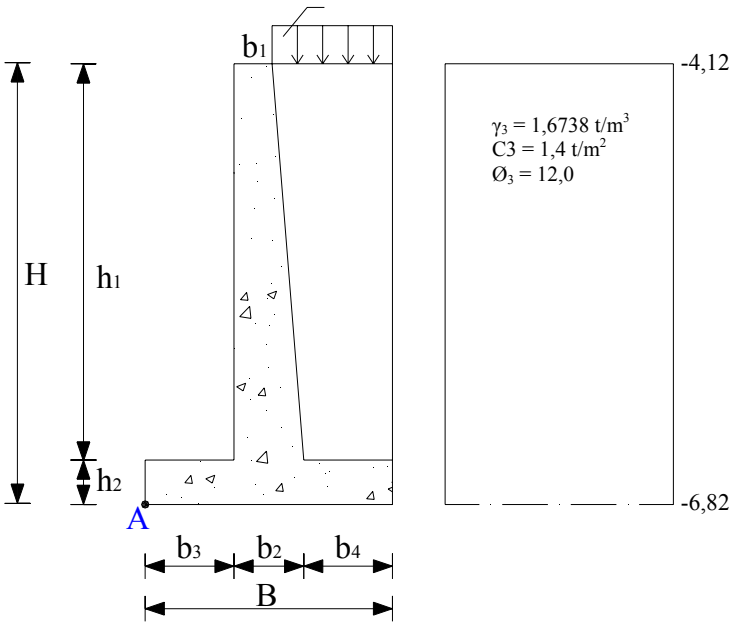
Tulangan terpasang = Ø 12 - 250 (As = 452 mm^2)

5.6.3 Perhitungan Konstruksi Dinding Beda Elevasi Saluran

5.6.3.1 Perhitungan Pembebanan

- Pendimensian Dinding Gerbang

  Rencana dimensi dinding beda elevasi saluran adalah sebagai berikut :



Gambar 5.28 Pendimensian Dinding Beda Elevasi Saluran

Tabel 5.13 Dimensi Dinding Beda Elevasi Saluran

H2 = 1/4 H – 1/12 H ; diambil 0,2 m	b2 = 1/12 H – 1/10 H; diambil 0,25 m
H = ((+12,88 - +10,38) + 0,2 = 2,7 m	b3 = 1/3 H; diambil 0,80 m
H1 = 2,5 m	b4 = 0,80 m
b1 = 0,20 – 0,30 m ; diambil 0,20 m	B = 0,4 – 0,7 H ; diambil 1,85 m

- Perhitungan Koefisien Tekanan Tanah Aktif

$Ka = \frac{1-\sin\theta}{1+\sin\theta}$ atau Ka = $\tan^2(45° - \varphi/2)$

$$Ka_1 = \frac{1-\sin 9{,}0^0}{1+\sin 9{,}0^0}$$

$$Ka_1 = 0{,}729$$

$$Ka_2 = \frac{1-\sin 11{,}0^0}{1+\sin 11{,}0^0}$$

$$Ka_2 = 0{,}679$$

$$Ka_3 = \frac{1-\sin 12{,}0^0}{1+\sin 12{,}0^0}$$

$$Ka_3 = 0{,}656$$

Mencari nilai q :

Akibat beban pelat beton (D) = 0,12 x 2400 = 288 kg/m^2

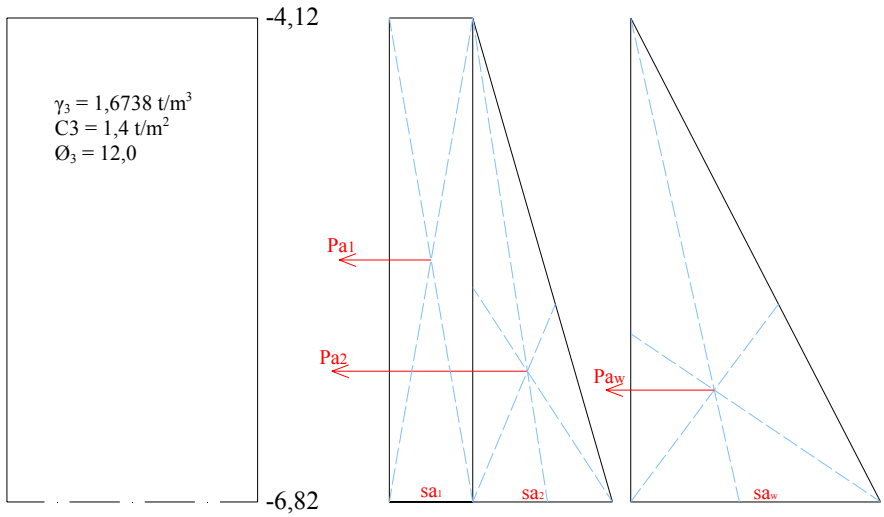
Akibat beban hidup (L) = diambil 1400 kg/m^2

Kombinasi pembebanan, q = 1,2.D + 1,6.L

= 1,2.288 + 1,6.1400

= 2585,6 kg/m^2

➢ Perhitungan Tegangan Tanah Aktif



Gambar 5.29 Diagram Tegangan Tanah Beda Elevasi Saluran

$\sigma a1 = Ka_3 \times q - (2 \times C_3 \times \sqrt{Ka_3})$

$= 0{,}656 \times 2{,}5856 - (2 \times 1{,}4 \times \sqrt{0{,}656})$

$= -0{,}572\ t/m^2$

$\sigma a2 = \gamma_{sub3} \times h \times Ka_3$

$= 0{,}6738 \times 2{,}7 \times 0{,}656$

$= 1{,}194\ t/m^2$

$\sigma aw = \gamma_w \times 2{,}70 \times K_{air}$

$= 1 \times 2{,}70 \times 1$

$= 2{,}70\ t/m^2$

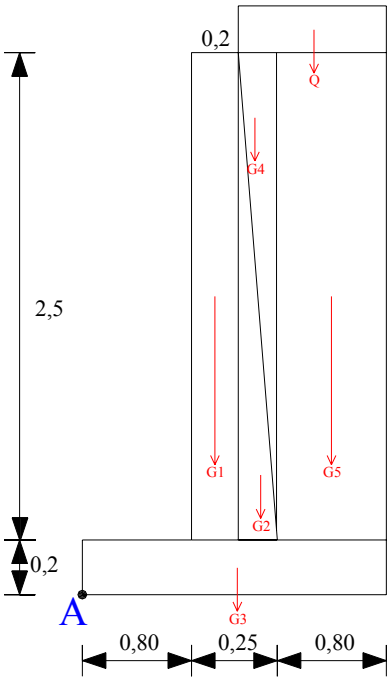
➢ Perhitungan Tekanan Tanah Aktif (per 1 m lebar)

Pa1 = σa1 x h = -0,572 x 2,7 = -1,544 t

Pa2 = ½ x σa2 x h = ½ x 1,194 x 2,7 = 1,612 t

Paw = ½ x σaw x 0,47 = ½ x 2,7 x 2,7 = 3,645 t

➢ Perhitungan Gaya – Gaya Vertikal (per 1 m lebar)



Gambar 5.30 Gaya-Gaya Vertikal Beda Elevasi Saluran

Akibat beban merata :

Q = q (b4+b2-b1) = 2,5856 (0,80 + 0,25 – 0,20) = 2,198 t

Akibat berat sendiri struktur :

G1 = b1 x h1 x γ_c = 0,20 x 2,5 x 2,4 = 1,260 t

G2 = ½ (b2 – b1) x h1 x γ_c = ½ x (0,25 – 0,20) x 2,5 x 2,4 = 0,150 t

G3 = B x h2 x γ_c = 1,85 x 0,20 x 2,4 = 0,888 t

Akibat berat tanah diatas struktur :

G4 = ½ x (b2 – b1) x h1 x γ_{sub3} = ½ x (0,25 – 0,20) x 2,5 x 0,6738

= 0,041 t

G5 = b4 x h2 x γ_{sub3} = 0,80 x 0,20 x 0,6738

= 0,108 t

Akibat berat air di atas struktur :

G6 = b4 x 2,70 x γ_w = 2,4 x 2,7 x 1 = 6,480 t

- Perhitungan Gaya Gempa

  Karena ketinggian dinding penahan tanah < 15 m yaitu 7,20 m maka tidak diperhitungkan gaya gempa.

- Perhitungan Momen terhadap Titik A

Tabel 5.14 Momen Aktif (Horisontal)

P	Gaya (ton)	Lengan (m)	M_{aktif} (tm)
Pa1	-1,544	1,350	-2,084
Pa2	1,612	0,900	1,451
Paw	3,645	0,900	3,281
	Σ P = 3,713		**Σ M_{aktif} = 2,648**

Tabel 5.15 Momen Pasif (Vertikal)

G	Gaya (ton)	Lengan (m)	M_{pasif} (tm)
Q	2,198	1,425	3,132
G1	1,260	0,900	1,134
G2	0,150	1,017	0,153
G3	0,888	0,925	0,821
G4	0,041	1,033	0,042
G5	0,108	1,450	0,157
G6	6,480	1,450	9,396
	Σ G = 11,125		**Σ M_{pasif} = 14,835**

<u>Cek Stabilitas Struktur :</u>

1. Kontrol terhadap guling

$$SF = \frac{\Sigma M_{pasif}}{\Sigma M_{aktif}} \geq 2$$

$$SF = \frac{14,835}{2,648} \geq 2$$

$SF = 5,60 \geq 2$ (aman)

2. Kontrol terhadap geser

$$SF = \frac{\Sigma G \tan\phi + B * C + \Sigma P_{pasif}}{\Sigma P} \geq 1,5$$

$$SF = \frac{11,125 \tan 12,0^0 + 1,85 * 1,4 + 0}{3,713} \geq 1,5$$

$SF = 1,334 < 1,5$ (perlu tiang pancang)

3. Kontrol terhadap eksentrisitas

e = ½.B - (ΣMp – ΣMa)/(ΣG)

$$e = 1/2B - \frac{\Sigma M_{pasif} - \Sigma M_{aktif}}{\Sigma G} \leq \frac{1}{6} B$$

$$e = 1/2x1,85 - \frac{14,835 - 2,648}{11,125} \leq \frac{1}{6} x1,85$$

$$e = 0,925 - 1,095 \leq 0,308$$

$e = -0,170 \leq 0,308$ (aman)

4. Daya dukung tanah

$$Nc = \frac{228 + 4,3\varphi}{40 - \varphi} = \frac{228 + 4,3 * 12,0^0}{40 - 12,0^0} = 9,986$$

$$Nq = \frac{40 + 5\varphi}{40 - \varphi} = \frac{40 + 5 * 12,0^0}{40 - 12,0^0} = 3,571$$

$$N\gamma = \frac{6\varphi}{40 - \varphi} = \frac{6 * 12,0^0}{40 - 12,0^0} = 2,571$$

q_{ult} = C.Nc + γ.D.Nq + ½.B.Nγ

= (1,4x9,986+(0,6738 x 0,2 x 3,571)+½ x 1,85 x 0,6738 x 2,571)

= 16,064 t/m^2

Daya dukung tanah yang diijinkan ditentukan dengan membagi q_{ult} dengan suatu faktor keamanan (SF) yaitu :

$q_{all} = {q_{ult}}/{SF}$ →diambil SF = 3

$q_{all} = {16,064}/{3}$

q_{all} = 5,355 t/m^2

5. Tegangan tanah yang terjadi

Kondisi yang harus diperhitungkan adalah pada saat kamar penuh, dengan berat air yang mempengaruhi dinding (pada bagian *toe* sepanjang 1,00 m).

W = H_{air} x b3 x γ_w = ((+16,00) – (+10,38)) x 0,80 x 1 = 4,496 t/m

Tegangan yang terjadi :

$$\sigma_{maks,\min} = \frac{(\Sigma G + W)}{B * L} x(1 \pm \frac{6 * e}{B})$$

$$= \frac{(11{,}125 + 4{,}496)}{1{,}85 * 1} x(1 \pm \frac{6 * -0{,}17}{1{,}85})$$

σ_{maks} = 13,099 t/m^2 > q_{all} = 5,355 t/m^2 (perlu tiang pancang)

σ_{min} = 3,788 t/m^2

5.6.3.2 Perhitungan Bagian Tapak Dinding

A. Bagian Tapak Depan (*Toe*)

$$q = q_{toe} - \frac{(\sigma_{maks} - \sigma_{\min})}{B} * X$$

$$q = (\sigma_{\min} + Uplift - \gamma_c * h2) - \frac{(\sigma_{maks} - \sigma_{\min})}{B} * b3$$

Uplift = P = H_{uplift} * γ_w = 2,7 x 1 = 2,70 t/m^2

Sehingga :

$$q = (3{,}788 + 2{,}70 - (2{,}4 \text{ x } 0{,}2)) - \frac{(13{,}099 - 3{,}788)}{1{,}85} * X$$

= 6,008 – 5,033 (X)

= 6,008 – 5,033 (0,8)

= 1,982 t/m

$$v = \int_0^{1,5} q dx = 6{,}008 \text{ X} - 2{,}517 \text{ X}^2 = 3{,}196 \text{ t}$$

$$M = \int_0^{1,5} v dx = 3{,}004 \text{ X}^2 - 0{,}839 \text{ X}^3 = 1{,}493 \text{ tm}$$

- Perhitungan Tulangan

h2 = 200 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = h2 – d' – Ø/2 = 200 – 50 – 20/2 = 140 mm

Mu = 1,493 tm = 1,493 x 10^7 Nmm

Mn = Mu/φ = 1,493 x 10^7/0,8 = 1,866 x 10^7 Nmm

k = Mn/(b.d^2.R$_l$) → R$_l$ = β$_1$.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 1,866 x 10^7/(1000 x 140^2 x 19,125) = 0,049

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0{,}049)}$ = 0,051

Fmax = β$_1$.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R$_l$/fy = (0,051 x 1000 x 140 x 19,125)/240 = 568,969 mm^2

ρ = As/(b.d) = 568,969/(1000 x 140) = 4,193 x 10^{-3}

ρ$_{min}$ = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ$_{maks}$ = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ$_{maks}$ = β$_1$.450/(600 + fy).(R$_l$/fy) = 0,85x450/(600+240)x(19,125/240)
= 0,03629

ρ < ρ$_{min}$ → dipakai ρ$_{min}$

As$_{min}$ = ρ$_{min}$.b.d = 5,83 x 10^{-3} x 1000 x 140 = 816,2 mm^2

Tulangan pokok terpasang = Ø 20 – 250 (As = 1257 mm^2)

Cek :

ρ = As$_{terpasang}$/b.d = 1257/(1000x140) = 8,979 x 10^{-3}

ρ$_{min}$ < ρ < ρ$_{maks}$

Dicoba tulangan bagi Ø 12

Tulangan bagi = 20% x tulangan pokok = 20% x 1257 = 251,4 mm^2

Tulangan bagi terpasang = Ø 12 – 250 (As = 452 mm^2)

B. Bagian Tapak Belakang (*Heel*)

Konstruksi pelat yang ditumpu pada ketiga sisinya, dimana beban yang bekerja di atas *heel* mencakup beberapa beban sebagai berikut :

- Beban Merata (q) = 2,5856 t/m^2
- Air Tanah = 2,50 x 1 = 2,50 t/m^2
- Tanah = γ_{sub3} x 2,5 = 0,6738 x 2,50 = 1,685 t/m^2
- Berat Sendiri = 2,4 x 0,8 x 0,2 = 0,384 t/m^2

q_{heel} = 2,5856+ 2,50 + 1,685 + 0,384 = 7,155 t/m^2

$q_u = \varphi$ x q_{heel} = 1,2 x 7,155 = 8,586 t/m^2

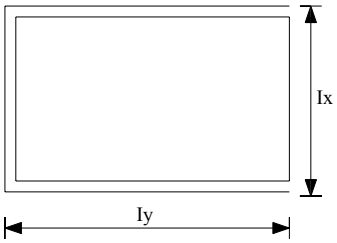
Jarak antar *counterfort* = 0,3 – 0,6 H → H = 7,2 m → diambil = 2,700 m

Iy = panjang *heel* = jarak antar *counterfort* = 2,700 m

Ix = lebar *heel* =0,80 m

Iy/Ix = 2,700/0,8 = 3,375

Maka momen untuk pelat yang terjepit pada tiga sisi adalah :



$M_{Ix} = 0{,}001.q_u.Ix^2.x$

$M_{Iy} = 0{,}001.q_u.Ix^2.x$

$M_{tx} = -0{,}001.q_u.Ix^2.x$

$M_{ty} = -0{,}001.q_u.Ix^2.x$

$M_{tiy} = ½.M_{Ix}$

Dimana nilai x berturut-turut berdasarkan perbandingan ly/lx = 3,375 adalah 65, 16, 83 dan 49 →(Tabel Hal 26 *Grafik dan Tabel Perhitungan Beton Bertulang, Seri Beton*-4, W.C. Vis, Gideon Kusuma)

M_{Ix} = 0,001 x 8,586 x 0,8^{2}x 65 = 0,357 tm

M_{Iy} = 0,001 x 8,586 x 0,8^{2}x 16 = 0,088 tm

M_{tx} = -0,001 x 8,586 x 0,8^{2}x 83 = -0,456 tm

M_{ty} = -0,001 x 8,586 x $0,8^2$x 49 = -0,269 tm

M_{tiy} = ½ x 0,357 = 0,179 tm

Diambil momen yang maksimum

- Tulangan Arah X (Tumpuan dan Lapangan)

Mu = 0,456 tm = 4,560 x 10^6 Nmm

Mn = Mu/φ = 4,560 x 10^6/0,8 = 5,700 x 10^6 Nmm

h2 =200 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = h2 – d' – Ø/2 = 200 – 50 – 20/2 = 140 mm

k = Mn/(b.d^2.R_l) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 5,700 x 10^6 /(1000 x 140^2 x 19,125) = 0,015

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0{,}015)}$ = 0,015

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R_l/fy = (0,015 x 1000 x 140 x 19,125)/240 = 167,344 mm^2

ρ = As/(b.d) = 167,344/(1000 x 140) = 1,195 x 10^{-3}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ_{maks} = β_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)

= 0,03629

ρ < ρ_{min} → dipakai ρ_{min}

As_{min} = ρ_{min}.b.d = 5,83 x 10^{-3} x 1000 x 140 = 816,2 mm^2

Tulangan pokok terpasang = Ø 20 – 250 (As = 1257 mm^2)

Cek :

ρ = $As_{terpasang}$/b.d = 1257/(1000x140) = 8,979 x 10^{-3}

ρ_{min} < ρ < ρ_{maks} → ok

❖ Tulangan Arah Y (Tumpuan dan Lapangan)

Mu = 0,269 tm = 2,690 x 10^6 Nmm

Mn = Mu/φ = 2,690 x 10^6/0,8 = 3,363 x 10^6 Nmm

h2 = 200 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = h2 – d' – Ø tulangan arah X - Ø/2 = 200 – 50 – 20 - 20/2 = 120 mm

k = Mn/(b.d^2.R_1) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 3,363 x 10^6/(1000 x 120^2 x 19,125) = 0,012

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0{,}012)}$ = 0,012

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R_l/fy = (0,012 x 1000 x 120 x 19,125)/240 = 114,75 mm^2

ρ = As/(b.d) = 114,75/(1000 x 120) = 9,563 x 10^{-4}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ_{maks} = β_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)

= 0,03629

ρ < ρ_{min} → dipakai ρ_{min}

As_{min} = ρ_{min}.b.d = 5,83 x 10^{-3} x 1000 x 120 = 699,6 mm^2

Tulangan pokok terpasang = Ø 20 – 250 (As = 1257 mm^2)

Cek :

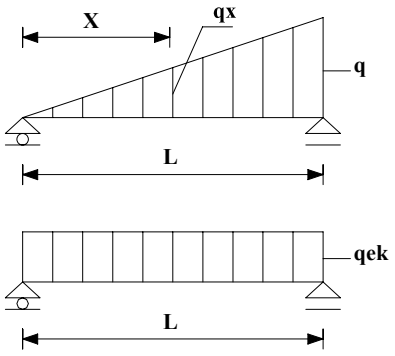
ρ = $As_{terpasang}$/b.d = 1257/(1000x120) = 0,010

ρ_{min} < ρ < ρ_{maks} → ok

5.6.3.3 Perhitungan Konstruksi Dinding Tegak

A. Perataan beban segitiga untuk pembebanan dinding

Mmaks = q.L^2 / 9√3 untuk x = L / √3



Gambar 5.31 Perataan Beban Segitiga

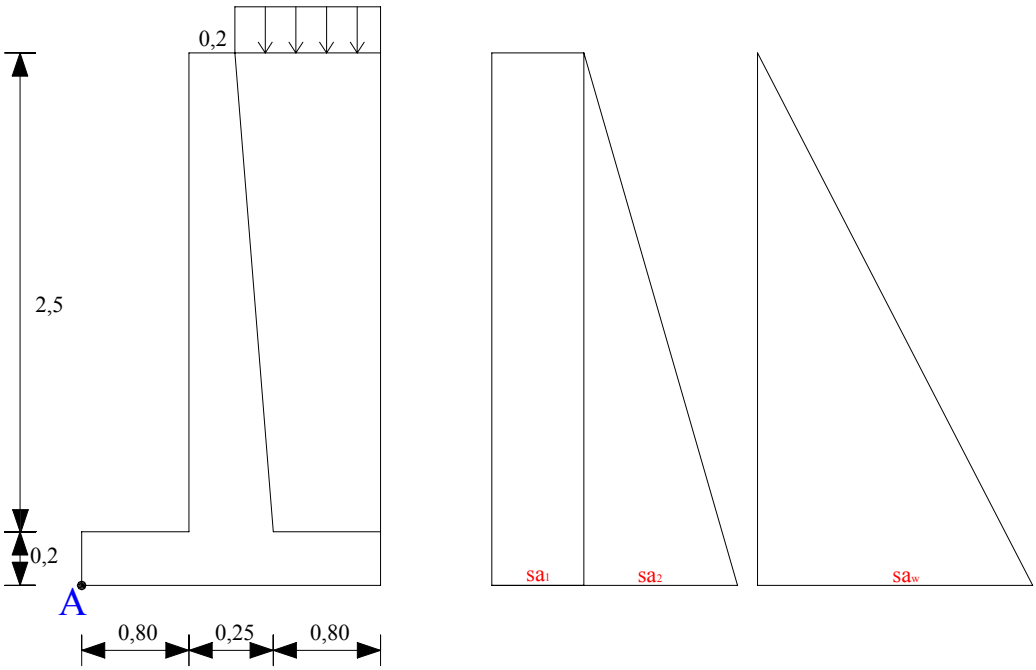
Besarnya beban ekivalen :

$M_{maks} = M_x$

$$M_x = \frac{q_{ek} * x * (L - x)}{2}$$

$$\frac{q * L^2}{9\sqrt{3}} = \frac{q_{ek} * x * (L - x)}{2} \quad \text{untuk } x = L / \sqrt{3}$$

$q_{ek} = 0{,}5246\ q$



Gambar 5.32 Diagram Tegangan Tanah Beda Elevasi

$\sigma a1 = Ka_3 \times q - (2 \times C_3 \times \sqrt{Ka_3})$

$= 0{,}656 \times 2{,}5856 - (2 \times 1{,}4 \times \sqrt{0{,}656})$

$= -0{,}572\ t/m^2$

$\sigma a2 = \gamma_{sub3} \times h \times Ka_3$

$= 0{,}6738 \times 2{,}7 \times 0{,}656$

$= 1{,}194\ t/m^2$

$\sigma aw = \gamma_w \times 2{,}70 \times K_{air}$

$= 1 \times 2{,}70 \times 1$

$= 2{,}70\ t/m^2$

$q_{ek} = \sigma a1 + (0{,}5246 \times \sigma a2) + (0{,}5246 \times \sigma aw)$

$= -0{,}572 + (0{,}5246 \times 1{,}194) + (0{,}5246 \times 2{,}7)$

$= 1{,}471\ t/m^2$

B. Penulangan Dinding

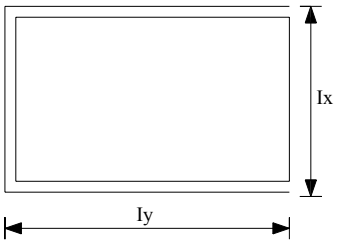
$q = 1{,}471\ t/m^2$

Ix = h1 = 2,50 m

Iy = jarak antar *counterfort* = 2,700 m

Iy/Ix = 2,700/2,5 = 1,080

Maka momen untuk pelat yang terjepit pada tiga sisi adalah :



$M_{Ix} = 0{,}001.q_u.Ix^2.x$

$M_{Iy} = 0{,}001.q_u.Ix^2.x$

$M_{tx} = -0{,}001.q_u.Ix^2.x$

$M_{ty} = -0{,}001.q_u.Ix^2.x$

$M_{tiy} = ½.M_{Ix}$

Dimana nilai x berturut-turut berdasarkan perbandingan ly/lx = 1,080 adalah 31,6; 23,4; 64 dan 54,4 →(Tabel Hal 26 *Grafik dan Tabel Perhitungan Beton Bertulang, Seri Beton*-4, W.C. Vis, Gideon Kusuma)

M_{Ix} = 0,001 x 1,471 x $2,5^2$x 31,6 = 0,291 tm

M_{Iy} = 0,001 x 1,471 x $2,5^2$x 23,4 = 0,215 tm

M_{tx} = -0,001 x 1,471 x $2,5^2$x 64 = -0,588 tm

M_{ty} = -0,001 x 1,471 x $2,5^2$x 54,4 = -0,500 tm

M_{tiy} = ½ x 0,291 = 0,146 tm

Diambil momen yang maksimum

- Tulangan Arah X (Tumpuan dan Lapangan)

Mu = 0,588 tm = 5,880 x 10^6 Nmm

Mn = Mu/φ = 5,880 x 10^6/0,8 = 7,350 x 10^6 Nmm

b = (b1 +b2)/2 = (200 + 250)/2 = 225 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = b – d' – Ø/2 = 225 – 50 – 20/2 = 165 mm

k = Mn/(b.d^2.R_1) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 7,35 x 10^6/(1000 x 165^2 x 19,125) = 0,014

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0,014)}$ = 0,014

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R_l/fy = (0,014 x 1000 x 165 x 19,125)/240 = 184,078 mm^2

ρ = As/(b.d) = 184,078/(1000 x 165) = 1,116 x 10^{-3}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ_{maks} = β_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)

= 0,03629

$\rho < \rho_{min} \rightarrow$ dipakai ρ_{min}

$As_{min} = \rho_{min}.b.d = 5,83 \times 10^{-3} \times 1000 \times 165 = 961,950\ mm^2$

Tulangan pokok terpasang = Ø 20 – 250 (As = 1257 mm^2)

Cek :

$\rho = As_{terpasang}/b.d = 1257/(1000x165) = 7,618 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

❖ Tulangan Arah Y (Tumpuan dan Lapangan)

Mu = 0,500 tm = 5×10^6 Nmm

Mn = Mu/φ = $5 \times 10^6/0,8 = 6,25 \times 10^6$ Nmm

b = (b1 +b2)/2 = (200 + 250)/2 = 225 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 20 mm

d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

d = b – d' – Ø tulangan arah X - Ø/2 = 225 – 50 – 20 - 20/2 = 145 mm

$k = Mn/(b.d^2.R_1) \rightarrow R_l = \beta_1.f'c = 0,85 \times 22,5 = 19,125\ N/mm^2$

$k = 6,25 \times 10^6/(1000 \times 145^2 \times 19,125) = 0,016$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0,016)} = 0,016$

$Fmax = \beta_1.450/(600 + fy) = (0,85 \times 450)/(600 + 240) = 0,455$

F < Fmax →tulangan tunggal *under reinforced*

$As = F.b.d.R_l/fy = (0,016 \times 1000 \times 145 \times 19,125)/240 = 184,875\ mm^2$

$\rho = As/(b.d) = 184,875/(1000 \times 145) = 1,275 \times 10^{-3}$

$\rho_{min} = 1,4/ fy = 1,4/240 = 5,83 \times 10^{-3}$

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)$

$= 0,03629$

$\rho < \rho_{min} \rightarrow$ dipakai ρ_{min}

$As_{min} = \rho_{min}.b.d = 5,83 \times 10^{-3} \times 1000 \times 145 = 845,35\ mm^2$

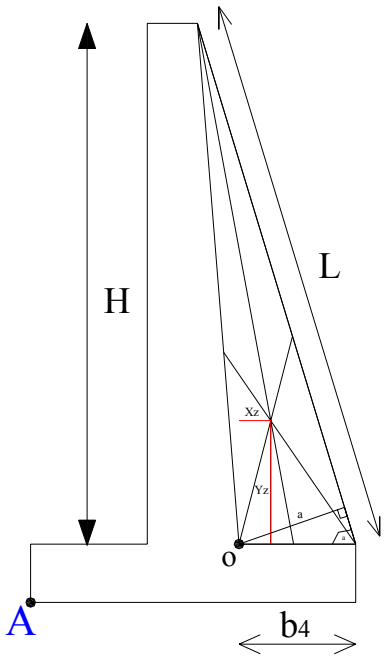
Tulangan pokok terpasang = Ø 20 – 250 (As = 1257 mm^2)

Cek :

$\rho = As_{terpasang}/b.d = 1257/(1000x145) = 8,669 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

5.6.3.4 Perhitungan Bagian Perkuatan Dinding (*Counterfort*)



- Perhitungan titik berat *counterfort* sebagai balok

  Xz = 1/3.b4 = 1/3 x 0,80 = 0,267 m

  Yz = 1/3.H = 1/3 x 2,5 = 0,833 m

- Perhitungan tinggi balok

  Tan α = H/(b4+b2-b1) =2,5/(0,8+0,25-0,2) = 2,941→ α = $71,222^0$

  a = Sin α.b4 = Sin $71,222^0$ x 0,8 = 0,757 m

  L = (b4+b2-b1)/Cos α = (0,8+0,25+0,2)/Cos $71,222^0$ = 2,641 m

- Pembebanan *counterfort*

Tabel 5.16 Pembebanan *Counterfort*

P	Gaya	Titik Berat		Terhadap Titik O		Lengan	Momen
	(ton)	Yz (m)	Xz (m)	Y (m)	X (m)	(m)	(tm)
Pa1	-1,544	0,833	0,267	1,350	-	0,517	-0,798

Pa2	1,612	0,833	0,267	0,900	-	0,067	0,108
Paw	3,645	0,833	0,267	0,900	-	0,067	0,244
Q	2,198	0,833	0,267	-	0,375	-0,108	-0,237
G1	1,260	0,833	0,267	-	-0,150	0,417	0,525
G2	0,150	0,833	0,267	-	-0,033	0,300	0,045
G3	0,888	0,833	0,267	-	-0,125	0,392	0,338
G4	0,041	0,833	0,267	-	-0,017	0,284	0,012
G5	0,108	0,833	0,267	-	0,400	-0,133	-0,014
G6	6,480	0,833	0,267	-	0,400	-0,133	-0,862
						Σ M =	-0,639

- Perhitungan tulangan lentur

  Mu = 0,639 = 6,39 x 10^6 Nmm

  Mn = Mu/φ = 6,39 x 10^6/0,8 = 7,988 x 10^6 Nmm

  Tebal *counterfort*, syarat ≥ 200 m; diambil b = 300 m

  Jarak antar *counterfort* = 0,3 – 0,6 H → diambil = 2,700 m

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

  fy = 2400 kg/cm^2 = 240 N/mm^2

  Dicoba tulangan pokok Ø 20 mm

  d' = tebal selimut beton = 50 mm (Tabel 3. *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton –1,*W. C. Vis, Gideon Kusuma)

  a = tinggi *counterfort* sebagai balok = 757 mm

  d = a – d' – Ø/2 = 757 – 50 – 20/2 = 697 mm

  k = Mn/(b.d^2.R_1) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

  k = 7,988 x 10^6 /(300 x 697^2 x 19,125) = 2,866 x 10^{-3}

  F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*2{,}866 \times 10^{-3})}$ = 2,870 x 10^{-3}

  Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

  F < Fmax →tulangan tunggal *under reinforced*

  As = F.b.d.R_l/fy = (2,870 x 10^{-3} x 300 x 697 x 19,125)/240 = 47,822 mm^2

  ρ = As/(b.d) = 47,822/(300 x 697) = 2,287 x 10^{-4}

  ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy)$ = 0,85x450/(600+240)x(19,125/240)

= 0,03629

$\rho < \rho_{min}$ → dipakai ρ_{min}

$As_{min} = \rho_{min}.b.d$ = 5,83 x 10^{-3} x 300 x 697 = 1219,053 mm^2

Tulangan terpasang = 4 Ø 20 (As = 1257 mm^2)

(pemasangan 2 baris 2 tulangan)

Cek :

$\rho = As_{terpasang}/b.d$ = 1257/(300x697) = 6,011 x 10^{-3}

$\rho_{min} < \rho < \rho_{maks}$ → ok

- Perhitungan tulangan horisontal

∑P = Pa1 + Pa2 + Paw

= -1,544 + 1,612 + 3,645

= 3,713 t

= 37130N

fy = ∑P/As

As = ∑P/fy

= 37130/240

= 154,708 mm^2 → (untuk 2 sisi)

= 77,354 mm^2 → (untuk 1 sisi)

Pemasangan tulangan untuk tiap 1 meter pias :

As = 77,354 / H = 77,354 / 2,5 = 30,932 mm^2

Tulangan terpasang = Ø 12 - 250 (As = 452 mm^2)

- Perhitungan tulangan vertikal

∑G= Q + G1 + G2 + G3 + G4 + G5 + G6

= 2,198 + 1,26 + 0,15 + 0,888 + 0,041 + 0,108 + 6,480

= 11,125 t

= 111250 N

fy = ∑G/As

As = ∑G/fy

= 111250/240

= 463,542 mm^2 → (untuk 2 sisi)

= 231,771 mm^2 → (untuk 1 sisi)

Pemasangan tulangan untuk tiap 1 meter pias :

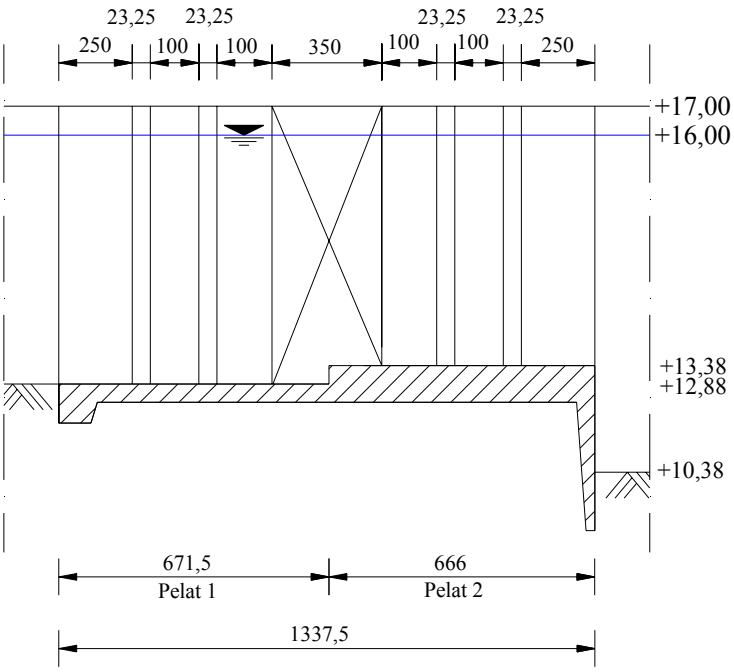
As = 231,771/ b4 = 231,771 / 0,8 = 289,714 mm^2

Tulangan terpasang = Ø 12 - 250 (As = 452 mm^2)

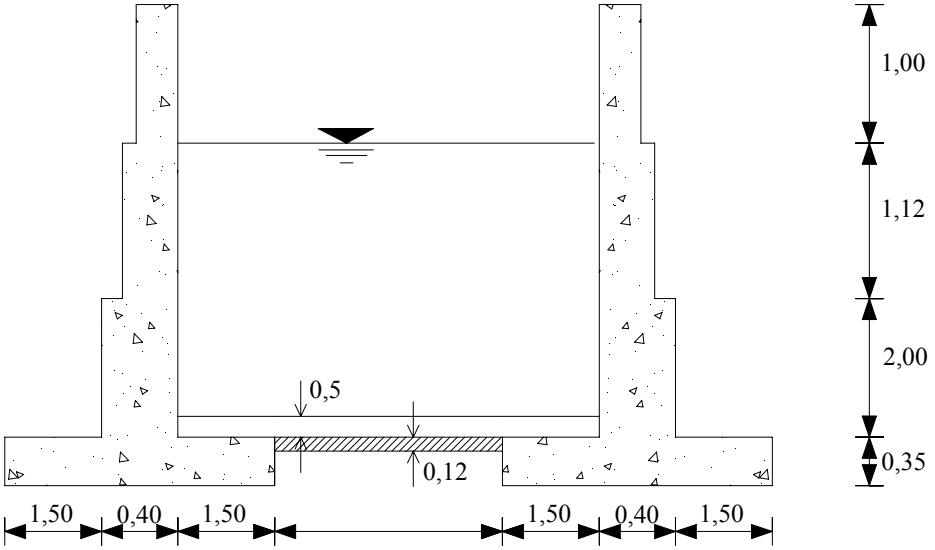
5.7 Perhitungan Pelat Lantai dan Balok Lantai

5.7.1 Perhitungan Pelat Lantai dan Balok Lantai Gerbang A

5.7.1.1 Perhitungan Pelat Lantai



Gambar 5.33 Tampak Samping Gerbang A



Gambar 5.34 Potongan Melintang Gerbang A

1. Pembebanan Pelat Lantai

   f'c = 225 kg/cm^2

   fy = 2400 kg/cm^2

   $$h_{\min} = \frac{\ln(0{,}8 + fy/1500)}{36 + 9\beta}$$

   ln = Iy = 3020 mm

   lx = 2675 mm (sama dengan jarak antar *counterfort* dinding gerbang A)

   β = ly/lx = 1,13

   $$h_{\min} = \frac{3020(0{,}8 + 240/1500)}{36 + (9x1{,}13)}$$

   $$h_{\min} = 62{,}794\,\text{mm}$$

   $$h_{maks} = \frac{\ln(0{,}8 + fy/1500)}{36}$$

   $$h_{maks} = \frac{3020(0{,}8 + 240/1500)}{36}$$

   $$h_{maks} = 80{,}533\,\text{mm}$$

   Syarat :

   Iy/Ix< 2,0 →h_{min} = 120 mm

   Iy/Ix ≥2,0 →h_{min} = 90 mm

Berdasarkan Tabel 10 *Dasar-Dasar Perencanaan Beton Bertulang-Seri Beton 1,* h_{min}= (1/27).lx = (1/27) x 2675 =99,074 mm

Dipilih tebal pelat = 120 mm (paling aman)

Konstruksi Ambang (Trap), direncanakan tidak terbuat dari material beton bertulang dan hanya digunakan untuk memperkuat stabilitas konstruksi pintu gerbang, maka ketebalan pelat lantai sebenarnya tetap sebesar 120 mm.

- Kondisi 1 (Gerbang Kosong)

  H = tinggi muka air tanah dari dasar pelat

  H = (+13,00) – (+12,88) = 0,12 m, dengan pembebanan :

  - Beban akibat kostruksi ambang = ditentukan sebesar -900 kg/m^2
  - Berat sendiri pelat = 0,12 x 2400 = -288 kg/m^2
  - Gaya *Uplift* = 0,12 x 1000 = 120 kg/m^2

  q_{total} = (-900) + (-288) + (120) = -1068 kg/m^2 (ke bawah)

- Kondisi 2 (Gerbang Penuh Air)

  H = tinggi muka air dalam gerbang

  H = (+16,00) – (+12,88) = 3,12 m, dengan pembebanan :

  - Beban akibat kostruksi ambang = ditentukan sebesar -900 kg/m^2
  - Berat sendiri pelat = 0,12 x 2400 = -288 kg/m^2
  - Gaya *Uplift* = 0,12 x 1000 = 120 kg/m^2
  - Berat Air = 3,12 x 1000 = -3120 kg/m^2

  q_{total} = (-900) + (-288) + (120) + (-3120) = -4188 kg/m^2 (ke bawah)

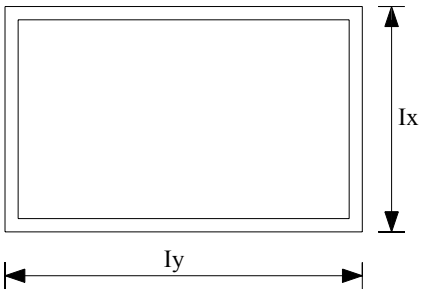
Pembebanan pada saat gerbang terisi air lebih besar daripada saat gerbang kosong, sehingga untuk perhitungan pelat lantai didasarkan pada pembebanan kondisi 2 dengan arah pembebanan ke bawah.

2. Penulangan Pelat Lantai

   q = 4188 kg/m^2

   Iy/Ix = 1,13

Untuk kondisi terjepit di keempat sisinya :



$M_{Ix} = 0{,}001.q_u.Ix^2.x$

$M_{Iy} = 0{,}001.q_u.Ix^2.x$

$M_{tx} = -0{,}001.q_u.Ix^2.x$

$M_{ty} = -0{,}001.q_u.Ix^2.x$

Dimana nilai x berturut-turut berdasarkan perbandingan ly/lx = 1,130 adalah 30; 23,5; 58 dan 52,5 → (Tabel Hal 26 *Grafik dan Tabel Perhitungan Beton Bertulang, Seri Beton*-4, W.C. Vis, Gideon Kusuma)

$M_{Ix} = 0{,}001 \times 4188 \times 2{,}675^2 \times 30 = 899{,}033$ kgm

$M_{Iy} = 0{,}001 \times 4188 \times 2{,}675^2 \times 23{,}5 = 704{,}242$ kgm

$M_{tx} = -0{,}001 \times 4188 \times 2{,}675^2 \times 58 = -1738{,}129$ kgm

$M_{ty} = -0{,}001 \times 4188 \times 2{,}675^2 \times 52{,}5 = -1573{,}307$ kgm

Diambil momen yang maksimum

❖ Tulangan Lapangan Arah X

Mu = 899,033 kgm = $8{,}99 \times 10^6$ Nmm

Mn = Mu/φ = $8{,}99 \times 10^6/0{,}8 = 11{,}238 \times 10^6$ Nmm

h = 120 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba menggunakan tulangan Ø 12 mm

d' = tebal selimut beton = 40 mm

d = h – d' – Ø/2 = 120 – 40 – 12/2 = 74 mm

$k = Mn/(b.d^2.R_1)$ → $R_l = \beta_1.f'c = 0{,}85 \times 22{,}5 = 19{,}125$ N/mm^2

$k = 11{,}238 \times 10^6 /(1000 \times 74^2 \times 19{,}125) = 0{,}107$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0{,}107)} = 0{,}113$

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R_l/fy = (0,113 x 1000 x 74 x 19,125)/240 = 666,347 mm^2

ρ = As/(b.d) = 666,347/(1000 x 74) = 9,005 x 10^{-3}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ_{maks} = β_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)

= 0,03629

atau $\rho_{maks} = \dfrac{F_{max}.R_1}{fy} = \dfrac{0{,}455x19{,}125}{240}$ =0,03626

ρ > ρ_{min} → dipakai ρ

As = ρ.b.d = 9,005 x 10^{-3} x 1000 x 74 = 666,37 mm^2

Tulangan terpasang = Ø 12 – 150 (As = 754 mm^2)

Cek terhadap rasio tulangan:

ρ = $As_{terpasang}$/b.d = 754/(1000x74) = 0,01

ρ_{min} < ρ < ρ_{maks} → ok

❖ Tulangan Lapangan Arah Y

Mu = 704,242 kgm = 7,042x 10^6 Nmm

Mn = Mu/φ = 7,042 x 10^6/0,8 = 8,803 x 10^6 Nmm

h = 120 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 12 mm

d' = tebal selimut beton = 40 mm

d = h4 – d' – Ø tulangan arah X - Ø/2 = 120 – 40 – 12 – 12/2 = 62 mm

k = Mn/(b.d^2.R_1) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 8,803 x 10^6/(1000 x 62^2 x 19,125) = 0,119

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0{,}119)} = 0{,}127$

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R_l/fy = (0,127 x 1000 x 62 x 19,125)/240 = 627,459 mm^2

ρ = As/(b.d) = 627,459/(1000 x 62) = 0,01

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ_{maks} = β_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)

= 0,03629

ρ > ρ_{min} → dipakai ρ

As = ρ.b.d = 0,01 x 1000 x 62 = 620 mm^2

Tulangan terpasang = Ø 12 – 175 (As = 646 mm^2)

Cek terhadap rasio tulangan :

ρ = $As_{terpasang}$/b.d = 646/(1000x62) = 0,01

ρ_{min} < ρ < ρ_{maks} → ok

❖ Tulangan Tumpuan Arah X

Mu = 1738,13 kgm = 1,738 x 10^7 Nmm

Mn = Mu/φ = 1,738 x 10^7/0,8 = 2,173 x 10^7 Nmm

h = 120 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba menggunakan tulangan Ø 12 mm

d' = tebal selimut beton = 40 mm

d = h – d' – Ø/2 = 120 – 40 – 12/2 = 74 mm

k = Mn/(b.d^2.R_1) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 2,173 x 10^7 /(1000 x 74^2 x 19,125) = 0,207

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0{,}207)}$ = 0,234

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R_l/fy = (0,234 x 1000 x 74 x 19,125)/240 = 1379,869 mm^2

ρ = As/(b.d) = 1379,869/(1000 x 74) = 0,019

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy)$ = 0,85x450/(600+240)x(19,125/240)

= 0,03629

atau $\rho_{maks} = \dfrac{F_{max}.R_1}{fy} = \dfrac{0,455x19,125}{240}$ =0,03626

$\rho > \rho_{min}$ → dipakai ρ

As = ρ.b.d = 0,019 x 1000 x 74 = 1406 mm^2

Tulangan terpasang = Ø 12 – 75 (As = 1508 mm^2)

Cek terhadap rasio tulangan:

$\rho = As_{terpasang}/b.d$ = 1508/(1000x74) = 0,02

$\rho_{min} < \rho < \rho_{maks}$ → ok

❖ Tulangan Tumpuan Arah Y

Mu = 1573,307 kgm = 1,573 x 10^7 Nmm

Mn = Mu/φ = 1,573 x 10^7/0,8 = 1,966 x 10^7 Nmm

h = 120 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 12 mm

d' = tebal selimut beton = 40 mm

d = h4 – d' – Ø tulangan arah X - Ø/2 = 120 – 40 – 12 – 12/2 = 62 mm

$k = Mn/(b.d^2.R_1)$ → $R_l = \beta_1.f'c$ = 0,85 x 22,5 = 19,125 N/mm^2

k = 1,966 x 10^7/(1000 x 62^2 x 19,125) = 0,267

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0,267)}$ = 0,317

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = $F.b.d.R_l/fy$ = (0,317x 1000 x 62 x 19,125)/240 = 1566,178 mm^2

ρ = As/(b.d) = 1566,178/(1000 x 62) = 0,025

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0{,}85x450/(600+240)x(\ 19{,}125/240)$

$= 0{,}03629$

$\rho > \rho_{min} \rightarrow$ dipakai ρ

$As = \rho.b.d = 0{,}025 \times 1000 \times 62 = 1550\ mm^2$

Tulangan terpasang = Ø 12 – 50 ($As = 2262\ mm^2$)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d = 2262/(1000x62) = 0{,}0360$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

5.7.1.2 Perhitungan Balok Lantai

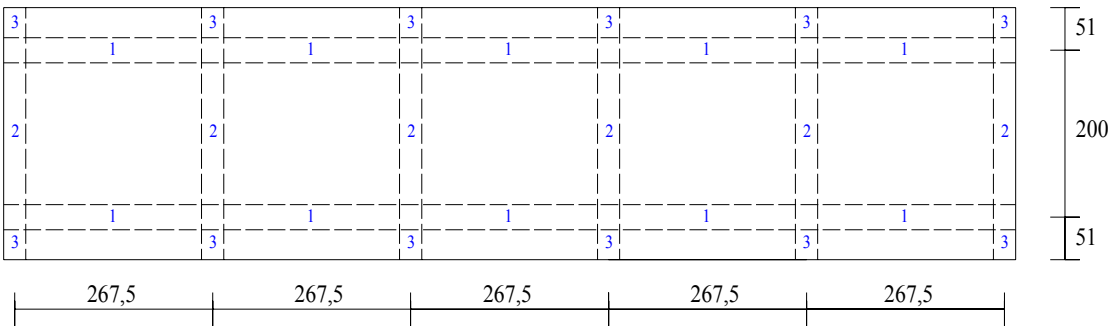
Diketahui panjang gerbang A = 13,375 m

Direncanakan panjang balok memanjang = 2,675 (sama dengan jarak antar *counterfort*)

Ly = 2,675 m

Lx = 2,000 m

- Pendimensian Balok Lantai



Gambar 5.35 Denah Balok Lantai Gerbang A

Meskipun panjang atau bentang balok lantai berbeda-beda, yaitu ada 3 macam (lihat Gambar 4.35), akan tetapi di sini perhitungan dimensi hanya didasarkan pada balok lantai yang mempunyai bentang terpanjang, yaitu balok 1

(untuk balok memanjang) dan balok 2 (untuk balok melintang), dengan pertimbangan kemudahan pelaksanaan di lapangan dan faktor keamanan.

- **Balok melintang**

  I = 200 cm = 2000 mm

  fy = 240 Mpa

  Faktor pengali (FP) $= (0{,}4 + \frac{fy}{700})$

  $= 0{,}743$

  $h_{min} = \frac{I}{28.FP} = \frac{2000}{28x0{,}743} = 96{,}135$ mm

  h = I/15 = 2000/15 = 133,333 mm

  Dipakai h = 450 mm

  b = 2/3.h = 2/3 x 450 = 300 mm

  Dimensi balok melintang = 450 x 300

- **Balok memanjang**

  I = 267,5 cm = 2675 mm

  fy = 240 Mpa

  Faktor pengali (FP) $= (0{,}4 + \frac{fy}{700})$

  $= 0{,}743$

  $h_{min} = \frac{I}{28.FP} = \frac{2675}{28x0{,}743} = 128{,}581$ mm

  h = I/15 = 2675/15 = 178,333 mm

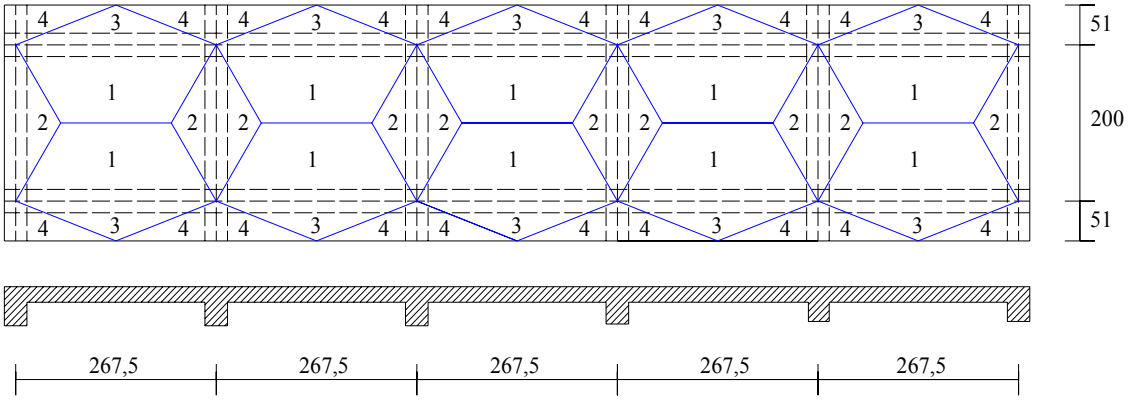
  Dipakai h = 450 mm

  b = 2/3.h = 2/3 x 450 = 300 mm

  Dimensi balok melintang = 450 x 300

➢ Pembebanan Balok Lantai

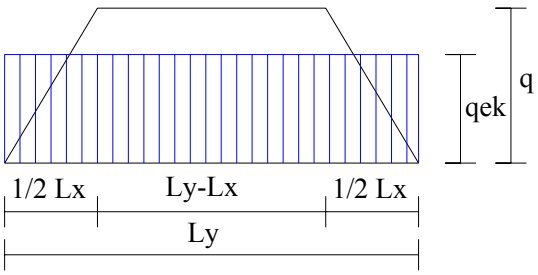
- Perataan beban



Gambar 5.36 Lay Out Balok Lantai Gerbang A serta Pola Pembebanan dengan Metode Amplop

Meskipun luasan segmen pelat berbeda-beda, yaitu ada 4 macam (lihat Gambar 5.36), akan tetapi untuk perataan beban hanya diambil segmen pelat dengan luasan terbesar, yaitu segmen 1 untuk perataan beban trapesium, dan segmen 2 serta segmen 3 untuk perataan beban segitiga.

<u>Beban Trapesium</u>



Gambar 5.37 Perataan Beban Trapesium

M_{max}Trapesium = 1/48.q.Lx.(3.Ly^2 – Lx^2)

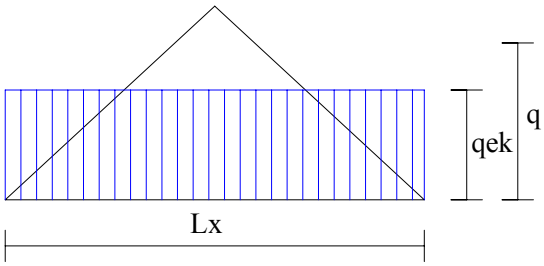
M_{max}Segi Empat = 1/8.q_{ek}.Ly^2

M_{max}Trapesium = M_{max} Segi Empat

1/48.q.Lx.(3.Ly^2 – Lx^2) = 1/8. q_{ek}.Ly^2

$$q_{ek} = \frac{q.Lx.(3.Ly^2 - Lx^2)}{6.Ly^2}$$

<u>Beban Segitiga</u>



Gambar 5.38 Perataan Beban Segitiga

M_{max}Segitiga = 1/12.q.Lx^2

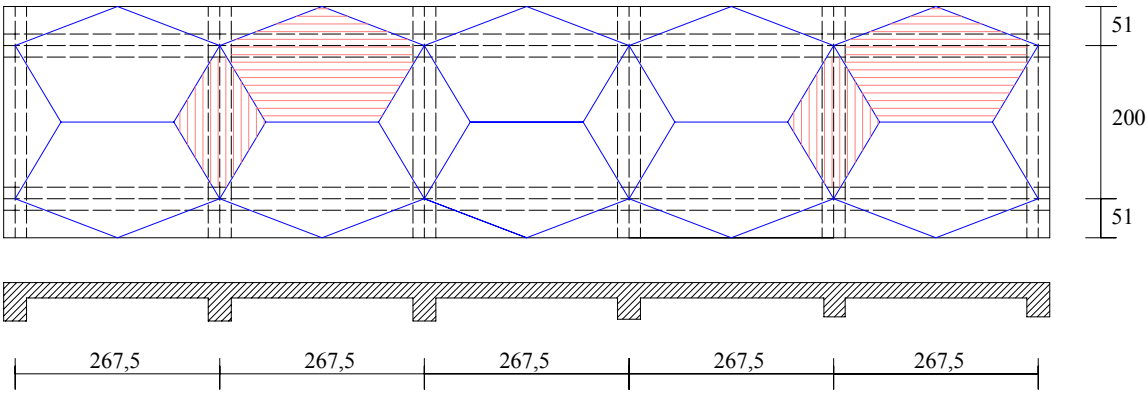
M_{max}Segi Empat = 1/8.q_{ek}.Ly^2

M_{max}Segitiga = M_{max} Segi Empat

1/12.q.Lx^2 = 1/8.q_{ek}.Ly^2

q_{ek} = 2/3.q

- Pembebanan



Gambar 5.39 Pola Pembebanan Balok Lantai Gerbang A

q = 4188 kg/m^2 (lihat perhitungan pelat lantai)

- **Balok Melintang**

  Akibat beban pelat segitiga :

q_{ek} = 2/3.q = 2/3 x 4188 = 2792 kg/m

2 x q_{ek} = 2 x 2792 = 5584 kg/m

Akibat berat sendiri balok :

= 0,45 x 0,30 x 2400 = 324 kg/m

$\mathbf{q_{total}}$ = 5584 + 324 = 5908 kg/m

- **Balok Memanjang**

  Akibat beban pelat segitiga :

  q_{ek} = 2/3.q = 2/3 x 4188 = 2792 kg/m

  Akibat beban pelat trapesium :

  $$q_{ek} = \frac{q.Lx.(3.Ly^2 - Lx^2)}{6.Ly^2} = \frac{4188x2(3x2{,}675^2 - 2^2)}{6x2{,}675^2} = 3407{,}635 \text{ kg/m}$$

  Akibat berat sendiri balok :

  = 0,45 x 0,30 x 2400 = 324 kg/m

  $\mathbf{q_{total}}$ = 2792 + 3407,635 + 324 = 6523,635 kg/m

Asumsi : Balok menerus di atas dua tumpuan, sehingga :

M_{lap} = 1/16.q.L^2

M_{tump} = 1/11.q.L^2

Vu = ½ .q.L

➢ Pembebanan Balok Lantai

<u>Balok Melintang</u>

1. Tulangan Lentur Lapangan

   M_{lap} = 1/16.q.L^2 = 1/16 x 5908 x 2^2 = 1477kgm = 1,477 Nmm

   Direncanakan menggunakan :

   Tulangan utama = Ø 16 mm

   Tulangan sengkang = Ø 8 mm

   fy = 240 Mpa = 240 N/mm^2 = 2400 kg/cm^2

   h = 450 mm

   b = 300 mm

   d' = tebal selimut beton = 40 mm

   d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

$= 450 – 40 – ½ \times 16 – 8 = 394$ mm

$Mn = Mu/\varphi = 1,477 \times 10^7/0,8 = 1,846 \times 10^7$ Nmm

$k = Mn/(b.d^2.R_l) \rightarrow R_l = \beta_1.f'c = 0,85 \times 22,5 = 19,125$ N/mm^2

$k = 1,846 \times 10^7/(300 \times 394^2 \times 19,125) = 0,021$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0,021)} = 0,021$

$F_{max} = \beta_1.450/(600 + fy) = (0,85 \times 450)/(600 + 240) = 0,455$

$k_{max} = (1 – ½.Fmax).Fmax = (1 – ½ \times 0,455) \times 0,455 = 0,351$

$F < F_{max}$ →tulangan tunggal *under reinforced*

$k < k_{max}$→ tidak perlu tulangan tekan, akan tetapi tetap direncanakan di daerah tekan terdapat tulangan praktis 2 buah diameter 16 mm dengan pertimbangan kemudahan pelaksanaan di lapangan. Maka perlu diperiksa apakah tulangan ini perlu diperhitungkan ataukah tidak.

$As' = 2 \times (1/4.\pi.Ø^2) = 2 \times (1/4 \times 3,14 \times 16^2) = 401,92$ mm^2

$As_2 = As' = 401,92$ mm^2

$M_2 = As'.fy.(d – d') = 401,92 \times 240 \times (394 – 40) = 34147123,2$ Nmm

$M_1 = Mn – M_2 = 1,846 \times 10^7 - 34147123,2 = -15687123,2$ Nmm

$M_1 \leq 0$ → maka As' dianggap 0, perhitungan tulangan tunggal :

$As = F.b.d.R_l/fy = (0,021 \times 300 \times 394 \times 19,125)/240 = 197,800$ mm^2

$\rho = As/(b.d) = 197,800/(300 \times 394) = 1,673 \times 10^{-3}$

$\rho_{min} = 1,4/ fy = 1,4/240 = 5,83 \times 10^{-3}$

$\rho_{maks} = 0,03635$ →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0,85 \times 450/(600+240) \times (19,125/240)$

$= 0,03629$

$\rho < \rho_{min}$ → dipakai ρ_{min}

$As = \rho.b.d = 5,83 \times 10^{-3} \times 300 \times 394 = 689,106$ mm^2

Tulangan terpasang = 4 Ø 16 (As = 804 mm^2)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d = 804/(300 \times 394) = 6,802 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks}$ → ok

2. Tulangan Lentur Tumpuan

M_{tump} = 1/11.q.L^2 = 1/11 x 5908 x 2^2 = 2148,364 kgm = 2,148 x 10^7 Nmm

Direncanakan menggunakan :

Tulangan utama = Ø 16 mm

Tulangan sengkang = Ø 8 mm

fy = 240 Mpa = 240 N/mm^2 = 2400 kg/cm^2

h = 450 mm

b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

Mn = Mu/φ = 2,148 x 10^7/0,8 = 2,685 x 10^7 Nmm

k = Mn/(b.d^2.R_1) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 2,685 x 10^7/(300 x 394^2 x 19,125) = 0,033

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0{,}033)}$ = 0,034

F_{max} = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

k_{max} = (1 – ½.Fmax).Fmax = (1 – ½ x 0,455) x 0,455 = 0,351

F < F_{max} →tulangan tunggal *under reinforced*

k < k_{max}→ tidak perlu tulangan tekan, akan tetapi tetap direncanakan di daerah tekan terdapat tulangan praktis 2 buah diameter 16 mm dengan pertimbangan kemudahan pelaksanaan di lapangan. Maka perlu diperiksa apakah tulangan ini perlu diperhitungkan ataukah tidak.

As' = 2 x (1/4.π. $Ø^2$) = 2 x (1/4 x 3,14 x 16^2) = 401,92 mm^2

As_2 = As' = 401,92 mm^2

M_2 = As'.fy.(d – d') = 401,92 x 240 x (394 – 40) = 34147123,2 Nmm

M_1 = Mn – M_2 = 2,685 x 10^7 - 34147123,2 = -7297123,2 Nmm

$M_1 \leq 0$ → maka As' dianggap 0, perhitungan tulangan tunggal :

As = F.b.d.R_l/fy = (0,033x 300 x 394 x 19,125)/240 = 295,051 mm^2

ρ = As/(b.d) = 295,051 /(300 x 394) = 2,629 x 10^{-3}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)$

$= 0,03629$

$\rho < \rho_{min} \rightarrow$ dipakai ρ_{min}

$As = \rho.b.d = 5,83 \times 10^{-3} \times 300 \times 394 = 689,106\ mm^2$

Tulangan terpasang = 4 Ø 16 ($As = 804\ mm^2$)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d = 804/(300x394) = 6,802 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

3. Tulangan Sengkang

Vu = ½.q.L = ½ x 5908 x 2 = 5908 kg

Direncanakan menggunakan :

Tulangan sengkang = Ø 8 mm

$fy = 140\ Mpa = 140\ N/mm^2 = 1400\ kg/cm^2$

$f'c = 22,5\ Mpa = 22,5\ N/mm^2 = 225\ kg/cm^2$

h = 450 mm

b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

Di tumpuan Vu = 5908 kg

Pada jarak sejauh d dari tumpuan:

Vu = 5908 – q.d = 5908 – 5908 x 0,394 = 3580,248 kg

$Vn = Vu/\varphi = 3580,248/0,6 = 5967,08$ kg

$Vc = 0,17.b.d.\sqrt{f'c} = 0,17 \times 30 \times 39,4 \times \sqrt{225} = 3014,1$ kg

$(Vn – Vc) < 2/3.\sqrt{f'c}.b.d$

$(5967,08 – 3014,1) < 2/3 \times \sqrt{225} \times 30 \times 39,4$

$2952,98 < 11820 \rightarrow$ penampang cukup ukurannya

$\varphi.(Vc/2) = 0,6 \times (3014,1/2) = 904,23$ kg

$Vu > \varphi.(Vc/2) \rightarrow$ perlu tulangan geser di sepanjang tumpuan sampai terjadinya $\varphi.(Vc/2)$

$\varphi.Vc = 0,6 \times 3014,1 = 1808,46$ kg

Vu > φ.Vc → perlu tulangan geser

Direncanakan menggunakan sengkang Ø 8 mm

Av = jumlah luas penampang dua kali sengkang

= 2 x (1/4 x 3,14 x 8^2) = 100,48 mm^2 = 1,0048 cm^2

S = jarak sengkang

= Av.d.fy/(Vn – Vc)

= 1,0048 x 39,4 x 1400/(5967,08 – 3014,1) = 18,769 cm

Syarat : S < d/2

18,769 < 39,4/2

18,769 < 19,7 (ok)

Dipakai sengkang Ø 8 – 100 (tumpuan) dan Ø 8 – 140 (lapangan)

- Cek Terhadap Pengaruh Geser Lentur

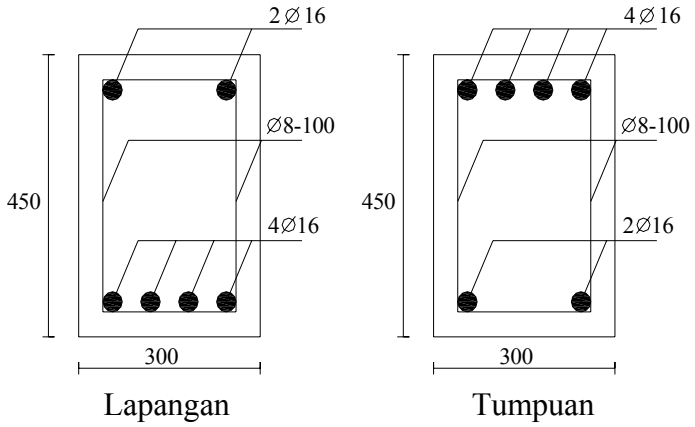
$$\tau = \frac{8.V}{7.b.h} = \frac{8x5908}{7x30x45} = 5{,}001 \text{ kg/cm}^2$$

Syarat :

$\tau \leq \tau_{bm}$

5,001 ≤ 1,35 $\sqrt{\sigma'_{bk}}$ = 1,35 x √225

5,001 ≤ 20,25 kg/cm^2 (aman)



Gambar 5.40 Penulangan Balok Melintang Lantai Gerbang A

<u>Balok Melintang</u>

1. Tulangan Lentur Lapangan

M_{lap} = 1/16.q.L^2 = 1/16 x 6523,635 x $2{,}675^2$ = 2917,542 kgm

= 2,918 x 10^7 Nmm

Direncanakan menggunakan :

Tulangan utama = Ø 16 mm

Tulangan sengkang = Ø 8 mm

fy = 240 Mpa = 240 N/mm^2 = 2400 kg/cm^2

h = 450 mm

b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

Mn = Mu/φ = 2,918 x 10^7/0,8 = 3,648 x 10^7 Nmm

k = Mn/(b.d^2.R$_1$) → R$_l$ = β$_1$.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 3,648 x 10^7/(300 x 394^2 x 19,125) = 0,041

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0{,}041)}$ = 0,042

F$_{max}$ = β$_1$.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

k$_{max}$ = (1 – ½.Fmax).Fmax = (1 – ½ x 0,455) x 0,455 = 0,351

F < F$_{max}$ →tulangan tunggal *under reinforced*

k < k$_{max}$→ tidak perlu tulangan tekan, akan tetapi tetap direncanakan di daerah tekan terdapat tulangan praktis 2 buah diameter 16 mm dengan pertimbangan kemudahan pelaksanaan di lapangan. Maka perlu diperiksa apakah tulangan ini perlu diperhitungkan ataukah tidak.

As' = 2 x (1/4.π. Ø^2) = 2 x (1/4 x 3,14 x 16^2) = 401,92 mm^2

As$_2$ = As' = 401,92 mm^2

M$_2$ = As'.fy.(d – d') = 401,92 x 240 x (394 – 40) = 34147123,2 Nmm

M$_1$ = Mn – M$_2$ = 3,648 x 10^7 - 34147123,2 = 2332876,8 Nmm

M$_1$ > 0 → maka As' diperhitungkan

k = M$_1$/(b.d^2.R$_1$) = 2332876,8/(300 x 394^2 x 19,125) = 2,619 x 10^{-3}

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*2{,}619\times10^{-3})}$ = 2,623 x 10^{-3}

F$_{max}$ = β$_1$.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < F$_{max}$ →tulangan tunggal *under reinforced*

As$_1$ = F.b.d.R$_l$/fy = (2,623 x 10^{-3} x 300 x 394 x 19,125)/240 = 24,706 mm^2

As = As$_1$ + As$_2$ = 24,706 + 401,92 = 426,626

ρ = As/(b.d) = 426,626/(300 x 394) = 3,609 x 10^{-3}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy)$ = 0,85x450/(600+240)x(19,125/240)

= 0,03629

$\rho < \rho_{min}$ → dipakai ρ_{min}

As = ρ.b.d = 5,83 x 10^{-3} x 300 x 394 = 689,106 mm^2

Tulangan terpasang = 4 Ø 16 (As = 804 mm^2)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d$ = 804/(300x394) = 6,802 x 10^{-3}

$\rho_{min} < \rho < \rho_{maks}$ → ok

Periksa :

$d'/d \leq d'/d_{max}$

$40/394 \leq 0,7.F_{max}$

0,102 ≤ 0,7 x 0,455

0,102 ≤ 0,319 (ok)

2. Tulangan Lentur Tumpuan

M_{tump} = 1/11.q.L^2 = 1/11 x 6523,635 x $2,675^2$ = 4243,7 kgm

= 4,244 x 10^7 Nmm

Direncanakan menggunakan :

Tulangan utama = Ø 16 mm

Tulangan sengkang = Ø 8 mm

fy = 240 Mpa = 240 N/mm^2 = 2400 kg/cm^2

h = 450 mm

b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

Mn = Mu/φ = 4,244 x 10^7/0,8 = 5,305 x 10^7 Nmm

k = Mn/(b.d^2.R_1) → $R_l = \beta_1.f'c$ = 0,85 x 22,5 = 19,125 N/mm^2

k = 5,305 x 10^7/(300 x 394^2 x 19,125) = 0,06

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0{,}06)} = 0{,}062$

$F_{max} = \beta_1.450/(600 + fy) = (0{,}85 \times 450)/(600 + 240) = 0{,}455$

$k_{max} = (1 - ½.Fmax).Fmax = (1 - ½ \times 0{,}455) \times 0{,}455 = 0{,}351$

$F < F_{max}$ →tulangan tunggal *under reinforced*

$k < k_{max}$→ tidak perlu tulangan tekan, akan tetapi tetap direncanakan di daerah tekan terdapat tulangan praktis 2 buah diameter 16 mm dengan pertimbangan kemudahan pelaksanaan di lapangan. Maka perlu diperiksa apakah tulangan ini perlu diperhitungkan ataukah tidak.

$As' = 2 \times (1/4.\pi.\ Ø^2) = 2 \times (1/4 \times 3{,}14 \times 16^2) = 401{,}92\ mm^2$

$As_2 = As' = 401{,}92\ mm^2$

$M_2 = As'.fy.(d - d') = 401{,}92 \times 240 \times (394 - 40) = 34147123{,}2$ Nmm

$M_1 = Mn - M_2 = 5{,}305 \times 10^7 - 34147123{,}2 = 18902876{,}8$ Nmm

$M_1 > 0$ → maka As' diperhitungkan

$k = M_1/(b.d^2.R_1) = 18902876{,}8/(300 \times 394^2 \times 19{,}125) = 0{,}02$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0{,}02)} = 0{,}02$

$F_{max} = \beta_1.450/(600 + fy) = (0{,}85 \times 450)/(600 + 240) = 0{,}455$

$F < F_{max}$ →tulangan tunggal *under reinforced*

$As_1 = F.b.d.R_l/fy = (0{,}02 \times 300 \times 394 \times 19{,}125)/240 = 188{,}381\ mm^2$

$As = As_1 + As_2 = 188{,}381 + 401{,}92 = 590{,}301\ mm^2$

$\rho = As/(b.d) = 590{,}301 /(300 \times 394) = 4{,}994 \times 10^{-3}$

$\rho_{min} = 1{,}4/ fy = 1{,}4/240 = 5{,}83 \times 10^{-3}$

$\rho_{maks} = 0{,}03635$ →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0{,}85x450/(600+240)x(19{,}125/240)$

$= 0{,}03629$

$\rho < \rho_{min}$ → dipakai ρ_{min}

$As = \rho.b.d = 5{,}83 \times 10^{-3} \times 300 \times 394 = 689{,}106\ mm^2$

Tulangan terpasang = 4 Ø 16 ($As = 804\ mm^2$)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d = 804/(300x394) = 6{,}802 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks}$ → ok

3. Tulangan Sengkang

Vu = ½.q.L = ½ x 6523,635 x 2,675 = 8725,362 kg

Direncanakan menggunakan :

Tulangan sengkang = Ø 8 mm

fy = 140 Mpa = 140 N/mm^2 = 1400 kg/cm^2

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

h = 450 mm

b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

Di tumpuan Vu = 8725,362 kg

Pada jarak sejauh d dari tumpuan:

Vu = 8725,362 – q.d = 8725,362 – 6523,635 x 0,394 = 6155,05 kg

Vn = Vu/φ = 6155,05/0,6 = 10258,417 kg

Vc = 0,17.b.d.√f'c = 0,17 x 30 x 39,4 x√225 = 3014,1 kg

(Vn – Vc) < 2/3.√f'c.b.d

(10258,417 – 3014,1) < 2/3 x √225 x 30 x 39,4

7217,317 < 11820 → penampang cukup ukurannya

φ.(Vc/2) = 0,6 x (3014,1/2) = 904,23 kg

Vu > φ.(Vc/2) → perlu tulangan geser di sepanjang tumpuan sampai terjadinya φ.(Vc/2)

φ.Vc = 0,6 x 3014,1 = 1808,46 kg

Vu > φ.Vc → perlu tulangan geser

Direncanakan menggunakan sengkang Ø 8 mm

Av = jumlah luas penampang dua kali sengkang

= 2 x (1/4 x 3,14 x 8^2) = 100,48 mm^2 = 1,0048 cm^2

S = jarak sengkang

= Av.d.fy/(Vn – Vc)

= 1,0048 x 39,4 x 1400/(10258,417 – 3014,1) = 7,651 cm

Syarat : S < d/2

7,651 < 39,4/2

7,651 < 19,7 (ok)

Dipakai sengkang Ø 8 – 100 (tumpuan) dan Ø 8 – 140 (lapangan)

- Cek Terhadap Pengaruh Geser Lentur

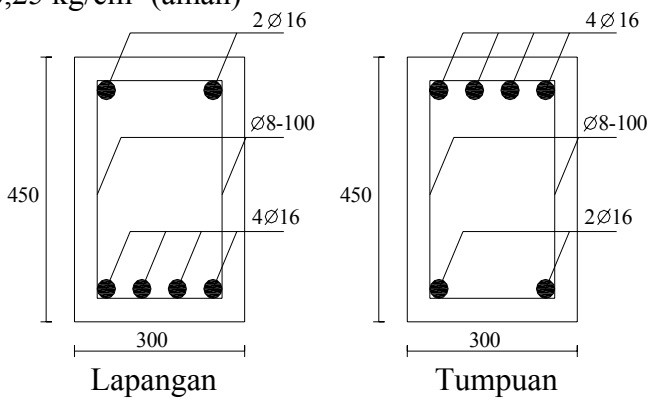
$$\tau = \frac{8.V}{7.b.h} = \frac{8x8725,362}{7x30x45} = 7,387 \text{ kg/cm}^2$$

Syarat :

$\tau \leq \tau_{bm}$

$7,387 \leq 1,35 \sqrt{\sigma'_{bk}} = 1,35 \text{ x } \sqrt{225}$

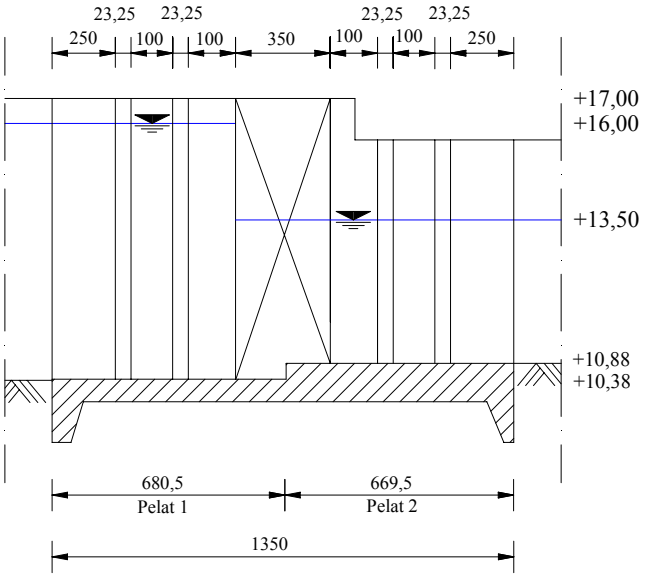
$7,387 \leq 20,25 \text{ kg/cm}^2$ (aman)



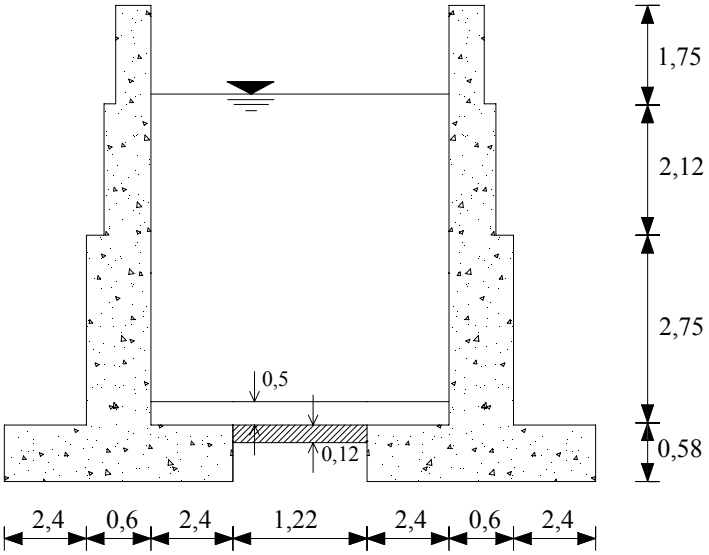
Gambar 5.41 Penulangan Balok Memanjang Lantai Gerbang A

5.7.2 Perhitungan Pelat Lantai dan Balok Lantai Gerbang B

5.7.2.1 Perhitungan Pelat Lantai



Gambar 5.42 Tampak Samping Gerbang B



Gambar 5.43 Potongan Melintang Gerbang B

1. Pembebanan Pelat Lantai

   f'c = 225 kg/cm^2

   fy = 2400 kg/cm^2

   $$h_{\min} = \frac{\ln(0{,}8 + + fy/1500)}{36 + 9\beta}$$

   ln = Iy = 2700 mm (sama dengan jarak antar *counterfort* dinding gerbang B)

   Ix = 1220

   β = ly/lx = 2,213

   $$h_{\min} = \frac{2700(0{,}8 + 240/1500)}{36 + (9x2{,}231)}$$

   $$h_{\min} = 46{,}354 \text{ mm}$$

   $$h_{maks} = \frac{\ln(0{,}8 + fy/1500)}{36}$$

   $$h_{maks} = \frac{2700(0{,}8 + 240/1500)}{36}$$

   $$h_{maks} = 72 \text{ mm}$$

   Syarat :

   Iy/Ix< 2,0 →h_{min} = 120 mm

   Iy/Ix ≥2,0 →h_{min} = 90 mm

Berdasarkan Tabel 10 *Dasar-Dasar Perencanaan Beton Bertulang-Seri Beton 1,* h_{min}= (1/27).lx = (1/27) x 1220 = 45,185 mm

Dipilih tebal pelat = 120 mm (paling aman)

Konstruksi Ambang (Trap), direncanakan tidak terbuat dari material beton bertulang dan hanya digunakan untuk memperkuat stabilitas konstruksi pintu gerbang, maka ketebalan pelat lantai sebenarnya tetap sebesar 120 mm.

- Kondisi 1 (Gerbang Kosong)

  H = tinggi muka air tanah dari dasar pelat

  H = (+13,00) – (+10,38) = 2,62 m, dengan pembebanan :

  - Beban akibat kostruksi ambang = ditentukan sebesar -900 kg/m^2
  - Berat sendiri pelat = 0,12 x 2400 = -288 kg/m^2
  - Gaya *Uplift* = 2,62 x 1000 = 2620 kg/m^2

  q_{total} = (-900) + (-288) + (2620) = 1432 kg/m^2 (ke atas)

- Kondisi 2 (Gerbang Penuh Air)

  H = tinggi muka air dalam gerbang

  H = (+16,00) – (+10,38) = 5,62 m, dengan pembebanan :

  - Beban akibat kostruksi ambang = ditentukan sebesar -900 kg/m^2
  - Berat sendiri pelat = 0,12 x 2400 = -288 kg/m^2
  - Gaya *Uplift* = 2,62 x 1000 = 2620 kg/m^2
  - Berat Air = 5,62 x 1000 = -5620 kg/m^2

  q_{total} = (-900) + (-288) + (2620) + (-5620) = -4188 kg/m^2 (ke bawah)

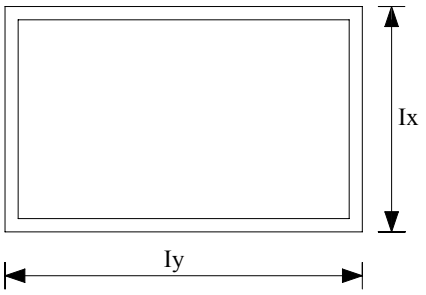
Pembebanan pada saat gerbang terisi air lebih besar daripada saat gerbang kosong, sehingga untuk perhitungan pelat lantai didasarkan pada pembebanan kondisi 2 dengan arah pembebanan ke bawah.

2. Penulangan Pelat Lantai

   q = 4188 kg/m^2

   Iy/Ix = 2,213

Untuk kondisi terjepit di keempat sisinya :



$M_{Ix} = 0{,}001.q_u.Ix^2.x$

$M_{Iy} = 0{,}001.q_u.Ix^2.x$

$M_{tx} = -0{,}001.q_u.Ix^2.x$

$M_{ty} = -0{,}001.q_u.Ix^2.x$

Dimana nilai x berturut-turut berdasarkan perbandingan ly/lx = 2,213 adalah 59,7; 14,6; 82,4 dan 52,1 → (Tabel Hal 26 *Grafik dan Tabel Perhitungan Beton Bertulang, Seri Beton*-4, W.C. Vis, Gideon Kusuma)

$M_{Ix} = 0{,}001 \times 4188 \times 1{,}220^2 \times 59{,}7 = 372{,}135$ kgm

$M_{Iy} = 0{,}001 \times 4188 \times 1{,}220^2 \times 14{,}6 = 91{,}008$ kgm

$M_{tx} = -0{,}001 \times 4188 \times 1{,}220^2 \times 82{,}4 = -513{,}634$ kgm

$M_{ty} = -0{,}001 \times 4188 \times 1{,}220^2 \times 52{,}1 = -324{,}761$ kgm

Diambil momen yang maksimum

❖ Tulangan Lapangan Arah X

Mu = 372,135 kgm = $3{,}721 \times 10^6$ Nmm

Mn = Mu/φ = $3{,}721 \times 10^6/0{,}8 = 4{,}651 \times 10^6$ Nmm

h = 120 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba menggunakan tulangan Ø 12 mm

d' = tebal selimut beton = 40 mm

d = h – d' – Ø/2 = 120 – 40 – 12/2 = 74 mm

$k = Mn/(b.d^2.R_1) \rightarrow R_l = \beta_1.f'c = 0{,}85 \times 22{,}5 = 19{,}125$ N/mm^2

$k = 4{,}561 \times 10^6 /(1000 \times 74^2 \times 19{,}125) = 0{,}044$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0{,}044)} = 0{,}045$

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R_l/fy = (0,045 x 1000 x 74 x 19,125)/240 = 265,359 mm^2

ρ = As/(b.d) = 265,359/(1000 x 74) = 3,586 x 10^{-3}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ_{maks} = β_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)

= 0,03629

atau $\rho_{maks} = \frac{F_{max}.R_1}{fy} = \frac{0,455x19,125}{240}$ =0,03626

ρ < ρ_{min} → dipakai ρ_{min}

As = ρ.b.d = 5,83 x 10^{-3} x 1000 x 74 = 431,42 mm^2

Tulangan terpasang = Ø 12 – 250 (As = 452 mm^2)

Cek terhadap rasio tulangan:

ρ = $As_{terpasang}$/b.d = 452/(1000x74) = 6,108 x 10^{-3}

ρ_{min} < ρ < ρ_{maks} → ok

❖ Tulangan Lapangan Arah Y

Mu = 91,008 kgm = 9,101x 10^5 Nmm

Mn = Mu/φ = 9,101x 10^5/0,8 = 1,138 x 10^6 Nmm

h = 120 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 12 mm

d' = tebal selimut beton = 40 mm

d = h4 – d' – Ø tulangan arah X - Ø/2 = 120 – 40 – 12 – 12/2 = 62 mm

k = Mn/(b.d^2.R_1) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 1,138 x 10^6/(1000 x 62^2 x 19,125) = 0,015

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0,015)}$ = 0,015

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

$As = F.b.d.R_l/fy = (0{,}015 \times 1000 \times 62 \times 19{,}125)/240 = 74{,}109\ mm^2$

$\rho = As/(b.d) = 74{,}109/(1000 \times 62) = 1{,}195 \times 10^{-3}$

$\rho_{min} = 1{,}4/\ fy = 1{,}4/240 = 5{,}83 \times 10^{-3}$

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0{,}85x450/(600+240)x(\ 19{,}125/240)$

$= 0{,}03629$

$\rho < \rho_{min}$ → dipakai ρ_{min}

$As = \rho.b.d = 5{,}83 \times 10^{-3} \times 1000 \times 62 = 361{,}46\ mm^2$

Tulangan terpasang = Ø 12 – 250 ($As = 452\ mm^2$)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d = 452/(1000x62) = 7{,}290 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks}$ → ok

❖ Tulangan Tumpuan Arah X

$Mu = 513{,}634\ kgm = 5{,}136 \times 10^6\ Nmm$

$Mn = Mu/\varphi = 5{,}136 \times 10^6/0{,}8 = 6{,}420 \times 10^6\ Nmm$

h = 120 mm

$f'c = 225\ kg/cm^2 = 22{,}5\ N/mm^2$

$fy = 2400\ kg/cm^2 = 240\ N/mm^2$

Dicoba menggunakan tulangan Ø 12 mm

d' = tebal selimut beton = 40 mm

d = h – d' – Ø/2 = 120 – 40 – 12/2 = 74 mm

$k = Mn/(b.d^2.R_1)$ → $R_l = \beta_1.f'c = 0{,}85 \times 22{,}5 = 19{,}125\ N/mm^2$

$k = 6{,}420 \times 10^6/(1000 \times 74^2 \times 19{,}125) = 0{,}061$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0{,}061)} = 0{,}063$

$Fmax = \beta_1.450/(600 + fy) = (0{,}85 \times 450)/(600 + 240) = 0{,}455$

F < Fmax →tulangan tunggal *under reinforced*

$As = F.b.d.R_l/fy = (0{,}063x\ 1000 \times 74 \times 19{,}125)/240 = 371{,}503\ mm^2$

$\rho = As/(b.d) = 371{,}503/(1000 \times 74) = 5{,}020 \times 10^{-3}$

$\rho_{min} = 1{,}4/\ fy = 1{,}4/240 = 5{,}83 \times 10^{-3}$

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy)$ = 0,85x450/(600+240)x(19,125/240)

= 0,03629

atau $\rho_{maks} = \dfrac{F_{max}.R_1}{fy} = \dfrac{0,455x19,125}{240}$ =0,03626

$\rho < \rho_{min}$ → dipakai ρ_{min}

As = ρ.b.d = 5,83 x 10^{-3} x 1000 x 74 = 431,42 mm^2

Tulangan terpasang = Ø 12 – 250 (As = 452 mm^2)

Cek terhadap rasio tulangan:

$\rho = As_{terpasang}/b.d$ = 452/(1000x74) = 6,108 x 10^{-3}

$\rho_{min} < \rho < \rho_{maks}$ → ok

❖ Tulangan Tumpuan Arah Y

Mu = 324,761 kgm = 3,248 x 10^6 Nmm

Mn = Mu/φ = 3,248 x 10^6/0,8 = 4,060 x 10^6 Nmm

h = 120 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 12 mm

d' = tebal selimut beton = 40 mm

d = h4 – d' – Ø tulangan arah X - Ø/2 = 120 – 40 – 12 – 12/2 = 62 mm

$k = Mn/(b.d^2.R_1)$ → $R_l = \beta_1.f'c$ = 0,85 x 22,5 = 19,125 N/mm^2

k = 4,060 x 10^6/(1000 x 62^2 x 19,125) = 0,055

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0,055)}$ = 0,057

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = $F.b.d.R_l/fy$ = (0,057x 1000 x 62 x 19,125)/240 = 281,616 mm^2

ρ = As/(b.d) = 281,616 /(1000 x 62) = 4,542 x 10^{-3}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0{,}85x450/(600+240)x(\ 19{,}125/240)$

$= 0{,}03629$

$\rho < \rho_{min} \rightarrow$ dipakai ρ_{min}

$As = \rho.b.d = 5{,}83 \times 10^{-3} \times 1000 \times 62 = 361{,}46\ mm^2$

Tulangan terpasang = Ø 12 – 250 ($As = 452\ mm^2$)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d = 452/(1000x62) = 7{,}290 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

5.7.2.2 Perhitungan Balok Lantai

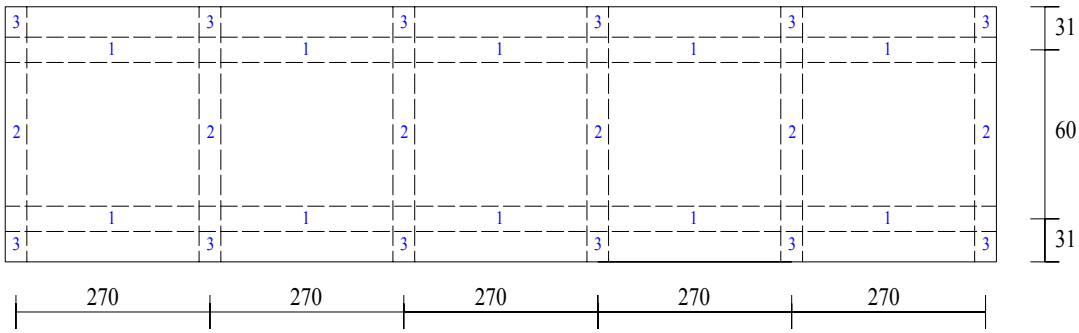
Diketahui panjang gerbang A = 13,50 m

Direncanakan panjang balok memanjang = 2,700 (sama dengan jarak antar *counterfort*)

Ly = 2,700 m

Lx = 0,600 m

- Pendimensian Balok Lantai



Gambar 5.44 Denah Balok Lantai Gerbang B

Meskipun panjang atau bentang balok lantai berbeda-beda, yaitu ada 3 macam (lihat Gambar 5.44), akan tetapi di sini perhitungan dimensi hanya didasarkan pada balok lantai yang mempunyai bentang terpanjang, yaitu balok 1 (untuk balok memanjang) dan balok 2 (untuk balok melintang), dengan

pertimbangan kemudahan pelaksanaan di lapangan. Dengan demikian, semua dimensi balok melintang maupun memanjang adalah sama.

- **Balok melintang**

  I = 60 cm = 600 mm

  fy = 240 Mpa

  Faktor pengali (FP) $= (0{,}4 + \frac{fy}{700})$

  $= 0{,}743$

  $h_{min} = \frac{I}{28.FP} = \frac{600}{28x0{,}743} = 28{,}841$ mm

  h = I/15 = 600/15 = 40 mm

  Dipakai h = 450 mm

  b = 2/3.h = 2/3 x 450 = 300 mm

  Dimensi balok melintang = 450 x 300

- **Balok memanjang**

  I = 270 cm = 2700 mm

  fy = 240 Mpa

  Faktor pengali (FP) $= (0{,}4 + \frac{fy}{700})$

  $= 0{,}743$

  $h_{min} = \frac{I}{28.FP} = \frac{2700}{28x0{,}743} = 129{,}783$ mm

  h = I/15 = 2700/15 = 180 mm

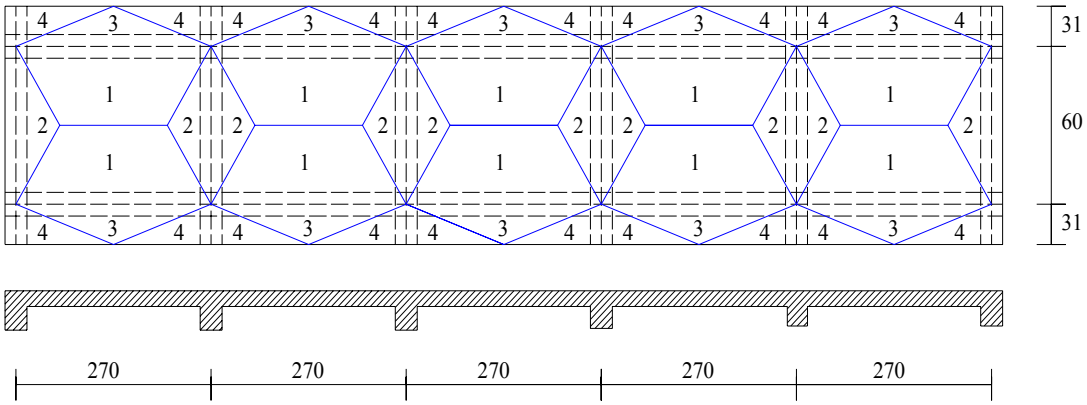
  Dipakai h = 450 mm

  b = 2/3.h = 2/3 x 450 = 300 mm

  Dimensi balok melintang = 450 x 300

➢ Pembebanan Balok Lantai

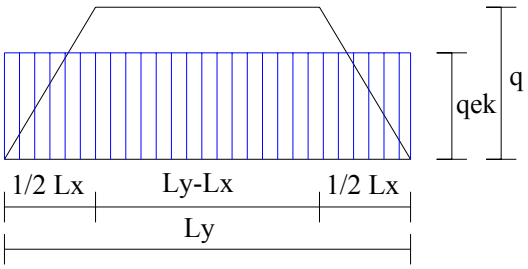
- Perataan beban



Gambar 5.45 Lay Out Balok Lantai Gerbang B serta Pola Pembebanan dengan Metode Amplop

Meskipun luasan segmen pelat berbeda-beda, yaitu ada 4 macam (lihat Gambar 5.45), akan tetapi untuk perataan beban hanya diambil segmen pelat dengan luasan terbesar, yaitu segmen 1 untuk perataan beban trapesium, dan segmen 2 serta segmen 3 untuk perataan beban segitiga.

<u>Beban Trapesium</u>



Gambar 5.46 Perataan Beban Trapesium

M_{max}Trapesium = $1/48.q.Lx.(3.Ly^2 - Lx^2)$

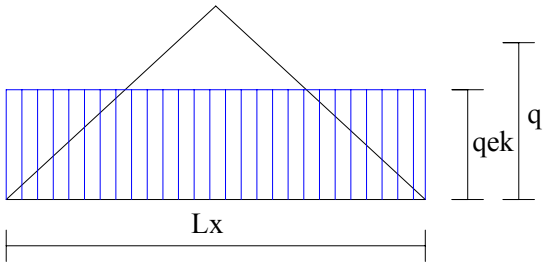
M_{max}Segi Empat = $1/8.q_{ek}.Ly^2$

M_{max}Trapesium = M_{max} Segi Empat

$1/48.q.Lx.(3.Ly^2 - Lx^2) = 1/8.\ q_{ek}.Ly^2$

$$q_{ek} = \frac{q.Lx.(3.Ly^2 - Lx^2)}{6.Ly^2}$$

<u>Beban Segitiga</u>



Gambar 5.47 Perataan Beban Segitiga

M_{max}Segitiga = 1/12.q.Lx^2

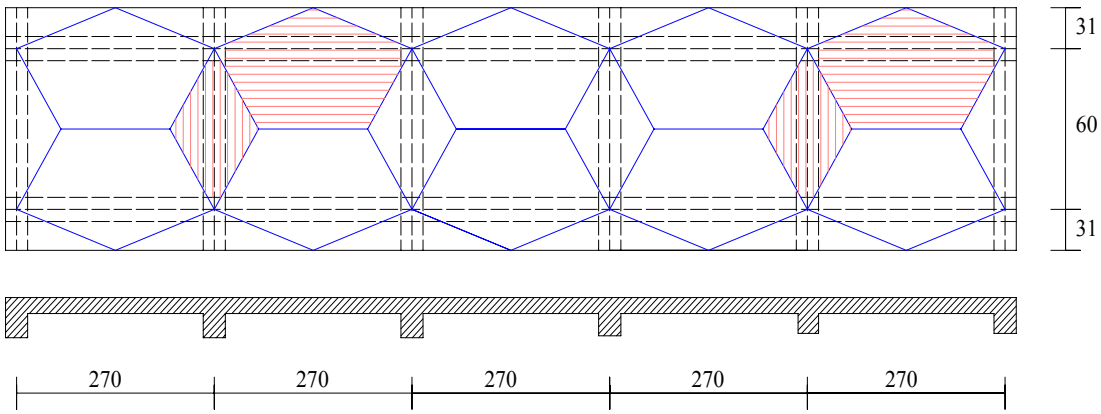
M_{max}Segi Empat = 1/8.q_{ek}.Ly^2

M_{max}Segitiga = M_{max} Segi Empat

1/12.q.Lx^2 = 1/8.q_{ek}.Ly^2

q_{ek} = 2/3.q

- Pembebanan



Gambar 5.48 Pola Pembebanan Balok Lantai Gerbang B

q = 4188 kg/m^2 (lihat perhitungan pelat lantai)

- **Balok Melintang**

  Akibat beban pelat segitiga :

q_{ek} = 2/3.q = 2/3 x 4188 = 2792 kg/m

2 x q_{ek} = 2 x 2792 = 5584 kg/m

Akibat berat sendiri balok :

= 0,45 x 0,30 x 2400 = 324 kg/m

$\mathbf{q_{total}}$ = 5584 + 324 = 5908 kg/m

- **Balok Memanjang**

  Akibat beban pelat segitiga :

  q_{ek} = 2/3.q = 2/3 x 4188 = 2792 kg/m

  Akibat beban pelat trapesium :

  $$q_{ek} = \frac{q.Lx.(3.Ly^2 - Lx^2)}{6.Ly^2} = \frac{4188x0,6(3x2,700^2 - 0,6^2)}{6x2,700^2} = 1235,719 \text{ kg/m}$$

  Akibat berat sendiri balok :

  = 0,45 x 0,30 x 2400 = 324 kg/m

  $\mathbf{q_{total}}$ = 2792 + 1235,719 + 324 = 4351,719 kg/m

Asumsi : Balok menerus di atas dua tumpuan, sehingga :

M_{lap} = 1/16.q.L^2

M_{tump} = 1/11.q.L^2

Vu = ½ .q.L

➢ Pembebanan Balok Lantai

<u>Balok Melintang</u>

1. Tulangan Lentur Lapangan

   M_{lap} = 1/16.q.L^2 = 1/16 x 5908 x $0,6^2$ = 312,93 kgm = 1,329 x 10^6 Nmm

   Direncanakan menggunakan :

   Tulangan utama = Ø 16 mm

   Tulangan sengkang = Ø 8 mm

   fy = 240 Mpa = 240 N/mm^2 = 2400 kg/cm^2

   h = 450 mm

   b = 300 mm

   d' = tebal selimut beton = 40 mm

   d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

$= 450 – 40 – ½ \times 16 – 8 = 394$ mm

$Mn = Mu/\varphi = 1{,}329 \times 10^6/0{,}8 = 1{,}661 \times 10^6$ Nmm

$k = Mn/(b.d^2.R_l) \rightarrow R_l = \beta_1.f'c = 0{,}85 \times 22{,}5 = 19{,}125$ N/mm^2

$k = 1{,}661 \times 10^6/(300 \times 394^2 \times 19{,}125) = 1{,}865 \times 10^{-3}$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*1{,}865 \times 10^{-3})} = 1{,}867 \times 10^{-3}$

$F_{max} = \beta_1.450/(600 + fy) = (0{,}85 \times 450)/(600 + 240) = 0{,}455$

$k_{max} = (1 – ½.Fmax).Fmax = (1 – ½ \times 0{,}455) \times 0{,}455 = 0{,}351$

$F < F_{max}$ →tulangan tunggal *under reinforced*

$k < k_{max}$→ tidak perlu tulangan tekan, akan tetapi tetap direncanakan di daerah tekan terdapat tulangan praktis 2 buah diameter 16 mm dengan pertimbangan kemudahan pelaksanaan di lapangan. Maka perlu diperiksa apakah tulangan ini perlu diperhitungkan ataukah tidak.

$As' = 2 \times (1/4.\pi.\ Ø^2) = 2 \times (1/4 \times 3{,}14 \times 16^2) = 401{,}92$ mm^2

$As_2 = As' = 401{,}92$ mm^2

$M_2 = As'.fy.(d – d') = 401{,}92 \times 240 \times (394 – 40) = 34147123{,}2$ Nmm

$M_1 = Mn – M_2 = 1{,}661 \times 10^6 - 34147123{,}2 = -32486123{,}2$ Nmm

$M_1 \leq 0$ → maka As' dianggap 0, perhitungan tulangan tunggal :

$As = F.b.d.R_l/fy = (1{,}867 \times 10^{-3} \times 300 \times 394 \times 19{,}125)/240 = 17{,}585$ mm^2

$\rho = As/(b.d) = 17{,}585/(300 \times 394) = 1{,}488 \times 10^{-4}$

$\rho_{min} = 1{,}4/ fy = 1{,}4/240 = 5{,}83 \times 10^{-3}$

$\rho_{maks} = 0{,}03635$ →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0{,}85 \times 450/(600+240) \times (19{,}125/240)$

$= 0{,}03629$

$\rho < \rho_{min}$ → dipakai ρ_{min}

$As = \rho.b.d = 5{,}83 \times 10^{-3} \times 300 \times 394 = 689{,}106$ mm^2

Tulangan terpasang = 4 Ø 16 (As = 804 mm^2)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d = 804/(300 \times 394) = 6{,}802 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks}$ → ok

2. Tulangan Lentur Tumpuan

M_{tump} = 1/11.q.L^2 = 1/11 x 5908 x $0,6^2$ = 193,353 kgm=1,934 x 10^6 Nmm

Direncanakan menggunakan :

Tulangan utama = Ø 16 mm

Tulangan sengkang = Ø 8 mm

fy = 240 Mpa = 240 N/mm^2 = 2400 kg/cm^2

h = 450 mm

b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

Mn = Mu/φ = 1,934 x 10^6/0,8 = 2,418 x 10^6 Nmm

k = Mn/(b.d^2.R_1) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 2,418 x 10^6/(300 x 394^2 x 19,125) = 2,715 x 10^{-3}

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*2,715 \text{ x } 10^{-3})}$ = 2,719 x 10^{-3}

F_{max} = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

k_{max} = (1 – ½.Fmax).Fmax = (1 – ½ x 0,455) x 0,455 = 0,351

F < F_{max} →tulangan tunggal *under reinforced*

k < k_{max}→ tidak perlu tulangan tekan, akan tetapi tetap direncanakan di daerah tekan terdapat tulangan praktis 2 buah diameter 16 mm dengan pertimbangan kemudahan pelaksanaan di lapangan. Maka perlu diperiksa apakah tulangan ini perlu diperhitungkan ataukah tidak.

As' = 2 x (1/4.π. $Ø^2$) = 2 x (1/4 x 3,14 x 16^2) = 401,92 mm^2

As_2 = As' = 401,92 mm^2

M_2 = As'.fy.(d – d') = 401,92 x 240 x (394 – 40) = 34147123,2 Nmm

M_1 = Mn – M_2 = 2,418 x 10^6 - 34147123,2 = -31729123,2 Nmm

$M_1 \leq 0$ → maka As' dianggap 0, perhitungan tulangan tunggal :

As = F.b.d.R_l/fy = (2,719 x 10^{-3}x 300 x 394 x 19,125)/240 = 25,610 mm^2

ρ = As/(b.d) = 25,610 /(300 x 394) = 2,167 x 10^{-4}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)$

$= 0,03629$

$\rho < \rho_{min} \rightarrow$ dipakai ρ_{min}

$As = \rho.b.d = 5,83 \times 10^{-3} \times 300 \times 394 = 689,106\ mm^2$

Tulangan terpasang = 4 Ø 16 ($As = 804\ mm^2$)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d = 804/(300x394) = 6,802 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

3. Tulangan Sengkang

Vu = ½.q.L = ½ x 5908 x 0,6 = 1772,4 kg

Direncanakan menggunakan :

Tulangan sengkang = Ø 8 mm

$fy = 140\ Mpa = 140\ N/mm^2 = 1400\ kg/cm^2$

$f'c = 22,5\ Mpa = 22,5\ N/mm^2 = 225\ kg/cm^2$

h = 450 mm

b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

Di tumpuan Vu = 1772,4 kg

Pada jarak sejauh d dari tumpuan:

Vu = 1772,4 – q.d = 1772,4 – 5908 x 0,394 = -555,352 kg

$Vn = Vu/\varphi = -555,352/0,6 = -925,587$ kg

$Vc = 0,17.b.d.\sqrt{f'c} = 0,17 \times 30 \times 39,4 \times \sqrt{225} = 3014,1$ kg

$(Vn – Vc) < 2/3.\sqrt{f'c}.b.d$

$(-925,587 – 3014,1) < 2/3 \times \sqrt{225} \times 30 \times 39,4$

-3939,687 < 11820 → penampang cukup ukurannya

$\varphi.(Vc/2) = 0,6 \times (3014,1/2) = 904,23$ kg

$Vu < \varphi.(Vc/2) \rightarrow$ tidak perlu tulangan geser di sepanjang tumpuan

$\varphi.Vc = 0,6 \times 3014,1 = 1808,46$ kg

Vu < φ.Vc → tidak perlu tulangan geser, tetapi demi pertimbangan kemudahan pelaksanaan di lapangan maka tetap direncanakan adanya tulangan geser.

Direncanakan menggunakan sengkang Ø 8 mm

Av = jumlah luas penampang dua kali sengkang

= 2 x (1/4 x 3,14 x 8^2) = 100,48 mm^2 = 1,0048 cm^2

S = jarak sengkang

= Av.d.fy/(Vn – Vc)

= 1,0048 x 39,4 x 1400/(-925,587– 3014,1) = -14,068 cm

Syarat : S < d/2

-14,068 < 39,4/2

-14,068 < 19,7 (ok)

Dipakai sengkang Ø 8 – 100 (tumpuan) dan Ø 8 – 140 (lapangan)

- Cek Terhadap Pengaruh Geser Lentur

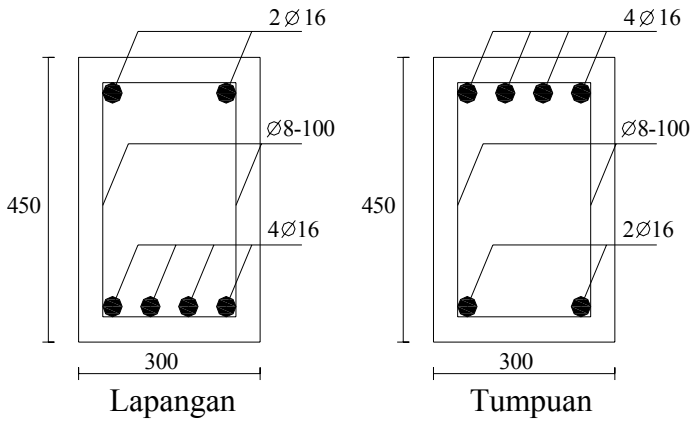
$$\tau = \frac{8.V}{7.b.h} = \frac{8x1772,4}{7x30x45} = 1,5 \text{ kg/cm}^2$$

Syarat :

$\tau \leq \tau_{bm}$

1,5 ≤ 1,35 $\sqrt{\sigma'_{bk}}$ = 1,35 x √225

1,5 ≤ 20,25 kg/cm^2 (aman)



Gambar 5.49 Penulangan Balok Melintang Lantai Gerbang B

Balok Melintang

1. Tulangan Lentur Lapangan

M_{lap} = 1/16.q.L^2 = 1/16 x 4351,719 x $2,700^2$ = 1982,752 kgm

= 1,983 x 10^7 Nmm

Direncanakan menggunakan :

Tulangan utama = Ø 16 mm

Tulangan sengkang = Ø 8 mm

fy = 240 Mpa = 240 N/mm^2 = 2400 kg/cm^2

h = 450 mm

b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

Mn = Mu/φ = 1,983 x 10^7/0,8 = 2,479 x 10^7 Nmm

k = Mn/(b.d^2.R_1) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 2,479 x 10^7/(300 x 394^2 x 19,125) = 0,028

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0,028)}$ = 0,028

F_{max} = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

k_{max} = (1 – ½.Fmax).Fmax = (1 – ½ x 0,455) x 0,455 = 0,351

F < F_{max} →tulangan tunggal *under reinforced*

k < k_{max}→ tidak perlu tulangan tekan, akan tetapi tetap direncanakan di daerah tekan terdapat tulangan praktis 2 buah diameter 16 mm dengan pertimbangan kemudahan pelaksanaan di lapangan. Maka perlu diperiksa apakah tulangan ini perlu diperhitungkan ataukah tidak.

As' = 2 x (1/4.π. $Ø^2$) = 2 x (1/4 x 3,14 x 16^2) = 401,92 mm^2

As_2 = As' = 401,92 mm^2

M_2 = As'.fy.(d – d') = 401,92 x 240 x (394 – 40) = 34147123,2 Nmm

M_1 = Mn – M_2 = 2,479 x 10^7 - 34147123,2 = -9357123,2 Nmm

M_1 < 0 → maka As' dianggap 0, perhitungan tulangan tunggal :

As_1 = F.b.d.R_l/fy = (0,028x 300 x 394 x 19,125)/240 = 263,734 mm^2

ρ = As/(b.d) = 263,734/(300 x 394) = 2,231 x 10^{-3}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy)$ = 0,85x450/(600+240)x(19,125/240)

= 0,03629

$\rho < \rho_{min}$ → dipakai ρ_{min}

As = ρ.b.d = 5,83 x 10^{-3} x 300 x 394 = 689,106 mm^2

Tulangan terpasang = 4 Ø 16 (As = 804 mm^2)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d$ = 804/(300x394) = 6,802 x 10^{-3}

$\rho_{min} < \rho < \rho_{maks}$ → ok

2. Tulangan Lentur Tumpuan

M_{tump} = 1/11.q.L^2 = 1/11 x 4351,719 x $2,700^2$ = 2884,003 kgm

= 2,884 x 10^7 Nmm

Direncanakan menggunakan :

Tulangan utama = Ø 16 mm

Tulangan sengkang = Ø 8 mm

fy = 240 Mpa = 240 N/mm^2 = 2400 kg/cm^2

h = 450 mm

b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

Mn = Mu/φ = 2,884 x 10^7 /0,8 = 3,605 x 10^7 Nmm

k = Mn/(b.d^2.R_l) → $R_l = \beta_1.f'c$ = 0,85 x 22,5 = 19,125 N/mm^2

k = 3,605 x 10^7/(300 x 394^2 x 19,125) = 0,040

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0,04)}$ = 0,041

$F_{max} = \beta_1.450/(600 + fy)$ = (0,85 x 450)/(600 + 240) = 0,455

k_{max} = (1 – ½.Fmax).Fmax = (1 – ½ x 0,455) x 0,455 = 0,351

F < F_{max} →tulangan tunggal *under reinforced*

k < k_{max}→ tidak perlu tulangan tekan, akan tetapi tetap direncanakan di daerah tekan terdapat tulangan praktis 2 buah diameter 16 mm dengan

pertimbangan kemudahan pelaksanaan di lapangan. Maka perlu diperiksa apakah tulangan ini perlu diperhitungkan ataukah tidak.

$As' = 2 \times (1/4.\pi.\ Ø^2) = 2 \times (1/4 \times 3{,}14 \times 16^2) = 401{,}92\ mm^2$

$As_2 = As' = 401{,}92\ mm^2$

$M_2 = As'.fy.(d - d') = 401{,}92 \times 240 \times (394 - 40) = 34147123{,}2$ Nmm

$M_1 = Mn - M_2 = 3{,}605 \times 10^7 - 34147123{,}2 = 1902876{,}8$ Nmm

$M_1 > 0 \rightarrow$ maka As' diperhitungkan

$k = M_1/(b.d^2.R_1) = 1902876{,}8/(300 \times 394^2 \times 19{,}125) = 2{,}136 \times 10^{-3}$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*2{,}136 \times 10^{-3})} = 2{,}138 \times 10^{-3}$

$F_{max} = \beta_1.450/(600 + fy) = (0{,}85 \times 450)/(600 + 240) = 0{,}455$

$F < F_{max} \rightarrow$tulangan tunggal *under reinforced*

$As_1 = F.b.d.R_l/fy = (2{,}138 \times 10^{-3} \times 300 \times 394 \times 19{,}125)/240 = 20{,}138\ mm^2$

$As = As_1 + As_2 = 20{,}138 + 401{,}92 = 422{,}058\ mm^2$

$\rho = As/(b.d) = 422{,}058/(300 \times 394) = 3{,}571 \times 10^{-3}$

$\rho_{min} = 1{,}4/fy = 1{,}4/240 = 5{,}83 \times 10^{-3}$

$\rho_{maks} = 0{,}03635 \rightarrow$(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0{,}85 \times 450/(600+240) \times (19{,}125/240)$

$= 0{,}03629$

$\rho < \rho_{min} \rightarrow$ dipakai ρ_{min}

$As = \rho.b.d = 5{,}83 \times 10^{-3} \times 300 \times 394 = 689{,}106\ mm^2$

Tulangan terpasang = 4 Ø 16 ($As = 804\ mm^2$)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d = 804/(300 \times 394) = 6{,}802 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

Periksa :

$d'/d \leq d'/d_{max}$

$40/394 \leq 0{,}7.F_{max}$

$0{,}102 \leq 0{,}7 \times 0{,}455$

$0{,}102 \leq 0{,}319$ (ok)

3. Tulangan Sengkang

Vu = ½.q.L = ½ x 4351,719 x 2,700 = 5874,821 kg

Direncanakan menggunakan :

Tulangan sengkang = Ø 8 mm

fy = 140 Mpa = 140 N/mm^2 = 1400 kg/cm^2

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

h = 450 mm

b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

Di tumpuan Vu = 5874,821 kg

Pada jarak sejauh d dari tumpuan:

Vu = 5874,821 – q.d = 5874,821 – 4351,719 x 0,394 = 4160,244 kg

Vn = Vu/φ = 4160,244/0,6 = 6933,74 kg

Vc = 0,17.b.d.√f'c = 0,17 x 30 x 39,4 x√225 = 3014,1 kg

(Vn – Vc) < 2/3.√f'c.b.d

(6933,74 – 3014,1) < 2/3 x √225 x 30 x 39,4

3919,64 < 11820 → penampang cukup ukurannya

φ.(Vc/2) = 0,6 x (3014,1/2) = 904,23 kg

Vu > φ.(Vc/2) → perlu tulangan geser di sepanjang tumpuan sampai terjadinya φ.(Vc/2)

φ.Vc = 0,6 x 3014,1 = 1808,46 kg

Vu > φ.Vc → perlu tulangan geser

Direncanakan menggunakan sengkang Ø 8 mm

Av = jumlah luas penampang dua kali sengkang

= 2 x (1/4 x 3,14 x 8^2) = 100,48 mm^2 = 1,0048 cm^2

S = jarak sengkang

= Av.d.fy/(Vn – Vc)

= 1,0048 x 39,4 x 1400/(6933,74 – 3014,1) = 14,140 cm

Syarat : S < d/2

14,14 < 39,4/2

14,14 < 19,7 (ok)

Dipakai sengkang Ø 8 – 100 (tumpuan) dan Ø 8 – 140 (lapangan)

- Cek Terhadap Pengaruh Geser Lentur

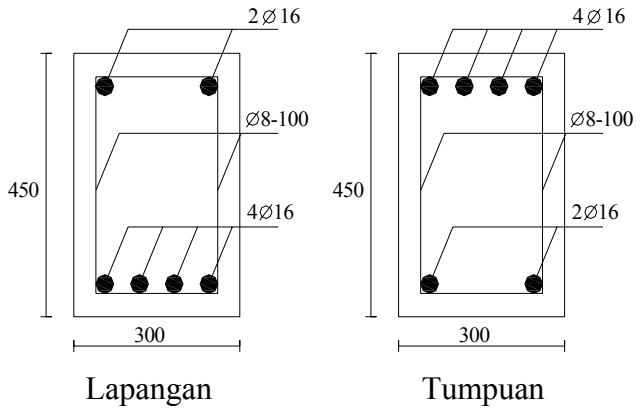
$$\tau = \frac{8.V}{7.b.h} = \frac{8x5874,821}{7x30x45} = 4,973 \text{ kg/cm}^2$$

Syarat :

$\tau \leq \tau_{bm}$

$4,973 \leq 1,35 \sqrt{\sigma'_{bk}} = 1,35 \text{ x } \sqrt{225}$

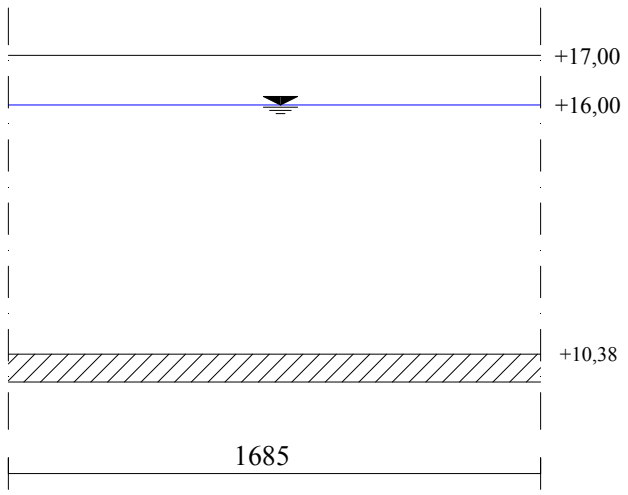
$4,973 \leq 20,25 \text{ kg/cm}^2$ (aman)



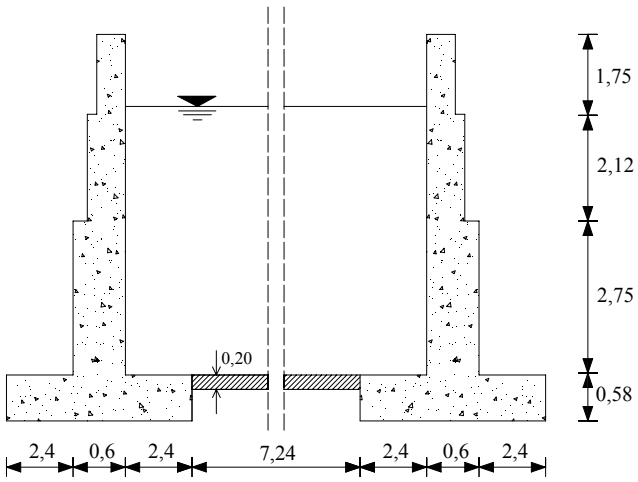
Gambar 5.50 Penulangan Balok Memanjang Lantai Gerbang B

5.7.3 Perhitungan Pelat Lantai dan Balok Lantai Kamar

5.7.3.1 Perhitungan Pelat lantai



Gambar 5.51 Tampak Samping Kamar



Gambar 5.52 Potongan Melintang Kamar

1. Pembebanan Pelat Lantai

   f'c = 225 kg/cm^2

   fy = 2400 kg/cm^2

   $$h_{\min} = \frac{\ln(0{,}8 + fy/1500)}{36 + 9\beta}$$

   ln = Iy = 7240 mm

   Ix = 2700 mm (diambil sama dengan jarak antar *counterfort* dinding kamar atau gerbang B)

   β = ly/lx = 2,681

   $$h_{\min} = \frac{7240(0{,}8 + 240/1500)}{36 + 9x2{,}681}$$

   $$h_{\min} = 115{,}591 \text{ mm}$$

   $$h_{maks} = \frac{\ln(0{,}8 + fy/1500)}{36}$$

   $$h_{maks} = \frac{7240(0{,}8 + 240/1500)}{36}$$

   $$h_{maks} = 193{,}067 \text{ mm}$$

   Syarat :

   Iy/Ix< 2,0 →h_{min} = 120 mm

   Iy/Ix ≥2,0 →h_{min} = 90 mm

Berdasarkan Tabel 10 *Dasar-Dasar Perencanaan Beton Bertulang-Seri Beton 1,* h_{min}= (1/27).lx = (1/27) x 2700 = 100 mm

Dipilih tebal pelat = 200 mm

- Kondisi 1 (Gerbang Kosong)

  H = tinggi muka air tanah dari dasar pelat

  H = (+13,00) – (+10,38) = 2,62 m, dengan pembebanan :

  - Berat sendiri pelat = 0,24 x 2400 = -480 kg/m^2
  - Gaya *Uplift* = 2,62 x 1000 = 2620 kg/m^2

  q_{total} = (-480) + (2620) = 2140 kg/m^2 (ke atas)

- Kondisi 2 (Gerbang Penuh Air)

  H = tinggi muka air dalam gerbang

  H = (+16,00) – (+10,38) = 5,62 m, dengan pembebanan :

  - Berat sendiri pelat = 0,20 x 2400 = -480 kg/m^2
  - Gaya *Uplift* = 2,62 x 1000 = 2620 kg/m^2
  - Berat Air = 5,62 x 1000 = -5620 kg/m^2

  q_{total} = (-480) + (2620) + (-5620) = -3480 kg/m^2 (ke bawah)

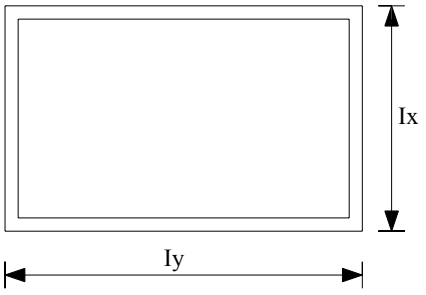
Pembebanan pada saat gerbang terisi air lebih besar daripada saat gerbang kosong, sehingga untuk perhitungan pelat lantai didasarkan pada pembebanan kondisi 2 dengan arah pembebanan ke bawah.

2. Penulangan Pelat Lantai

   q = 3480 kg/m^2

   Iy/Ix = 2,681

   Untuk kondisi terjepit di keempat sisinya :



$M_{Ix} = 0{,}001.q_u.Ix^2.x$

$M_{Iy} = 0{,}001.q_u.Ix^2.x$

$M_{tx} = -0{,}001.q_u.Ix^2.x$

$M_{ty} = -0{,}001.q_u.Ix^2.x$

Dimana nilai x berturut-turut berdasarkan perbandingan ly/lx = 2,681 adalah 63,1; 14; 83 dan 50,3 → (Tabel Hal 26 *Grafik dan Tabel Perhitungan Beton Bertulang, Seri Beton*-4, W.C. Vis, Gideon Kusuma)

$M_{Ix} = 0{,}001 \times 3480 \times 2{,}700^2 \times 63{,}1 = 1600{,}797$ kgm

$M_{Iy} = 0{,}001 \times 3480 \times 2{,}700^2 \times 14 = 355{,}169$ kgm

$M_{tx} = -0{,}001 \times 3480 \times 2{,}700^2 \times 83 = -2105{,}644$ kgm

$M_{ty} = -0{,}001 \times 3480 \times 2{,}700^2 \times 50{,}3 = -1276{,}071$ kgm

Diambil momen yang maksimum

❖ Tulangan Lapangan Arah X

Mu = 1600,797 kgm = $1{,}601 \times 10^7$ Nmm

Mn = Mu/φ = $1{,}601 \times 10^7/0{,}8 = 2{,}001 \times 10^7$ Nmm

h = 200 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba menggunakan tulangan Ø 16 mm

d' = tebal selimut beton = 40 mm

d = h – d' – Ø/2 = 200 – 40 – 16/2 = 152 mm

$k = Mn/(b.d^2.R_l)$ → $R_l = \beta_1.f'c = 0{,}85 \times 22{,}5 = 19{,}125$ N/mm^2

$k = 2{,}001 \times 10^7/(1000 \times 152^2 \times 19{,}125) = 0{,}045$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0{,}045)} = 0{,}046$

$Fmax = \beta_1.450/(600 + fy) = (0{,}85 \times 450)/(600 + 240) = 0{,}455$

F < Fmax →tulangan tunggal *under reinforced*

$As = F.b.d.R_l/fy = (0{,}046 \times 1000 \times 152 \times 19{,}125)/240 = 557{,}175$ mm^2

$\rho = As/(b.d) = 557{,}175/(1000 \times 152) = 3{,}666 \times 10^{-3}$

$\rho_{min} = 1{,}4/fy = 1{,}4/240 = 5{,}83 \times 10^{-3}$

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy)$ = 0,85x450/(600+240)x(19,125/240)

= 0,03629

atau $\rho_{maks} = \frac{F_{max}.R_1}{fy} = \frac{0,455x19,125}{240}$ =0,03626

$\rho < \rho_{min}$ → dipakai ρ_{min}

As = ρ.b.d = 5,83 x 10^{-3} x 1000 x 152 = 886,160 mm^2

Tulangan terpasang = Ø 16 – 225 (As = 894 mm^2)

Cek terhadap rasio tulangan:

$\rho = As_{terpasang}/b.d$ = 894/(1000x152) = 5,882 x 10^{-3}

$\rho_{min} < \rho < \rho_{maks}$ → ok

❖ Tulangan Lapangan Arah Y

Mu = 355,169 kgm = 3,552 x 10^6 Nmm

Mn = Mu/φ = 3,552 x 10^6/0,8 = 4,440 x 10^6 Nmm

h = 200 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 16 mm

d' = tebal selimut beton = 40 mm

d = h4 – d' – Ø tulangan arah X - Ø/2 = 200 – 40 – 16 – 16/2 = 136 mm

$k = Mn/(b.d^2.R_1)$ → $R_l = \beta_1.f'c$ = 0,85 x 22,5 = 19,125 N/mm^2

k = 4,440 x 10^6/(1000 x 136^2 x 19,125) = 0,013

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0,013)}$ = 0,013

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = $F.b.d.R_l/fy$ = (0,013 x 1000 x 136 x 19,125)/240 = 140,888 mm^2

ρ = As/(b.d) = 140,888/(1000 x 136) = 1,036 x 10^{-3}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)$

$= 0,03629$

$\rho < \rho_{min} \rightarrow$ dipakai ρ_{min}

$As = \rho.b.d = 5,83 \times 10^{-3} \times 1000 \times 136 = 792,880\ mm^2$

Tulangan terpasang = Ø 16 – 250 ($As = 804\ mm^2$)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d = 804/(1000x136) = 5,912 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

❖ Tulangan Tumpuan Arah X

$Mu = 2105,644\ kgm = 2,106 \times 10^7\ Nmm$

$Mn = Mu/\varphi = 2,106 \times 10^7/0,8 = 2,633 \times 10^7\ Nmm$

h = 200 mm

$f'c = 225\ kg/cm^2 = 22,5\ N/mm^2$

$fy = 2400\ kg/cm^2 = 240\ N/mm^2$

Dicoba menggunakan tulangan Ø 16 mm

d' = tebal selimut beton = 40 mm

d = h – d' – Ø/2 = 200 – 40 – 16/2 = 152 mm

$k = Mn/(b.d^2.R_1) \rightarrow R_l = \beta_1.f'c = 0,85 \times 22,5 = 19,125\ N/mm^2$

$k = 2,633 \times 10^7/(1000 \times 152^2 \times 19,125) = 0,059$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0,059)} = 0,061$

$Fmax = \beta_1.450/(600 + fy) = (0,85 \times 450)/(600 + 240) = 0,455$

F < Fmax →tulangan tunggal *under reinforced*

$As = F.b.d.R_l/fy = (0,061x\ 1000 \times 152 \times 19,125)/240 = 738,863\ mm^2$

$\rho = As/(b.d) = 738,863/(1000 \times 152) = 4,861 \times 10^{-3}$

$\rho_{min} = 1,4/ fy = 1,4/240 = 5,83 \times 10^{-3}$

$\rho_{maks} = 0,03635$ →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)$

$= 0,03629$

atau $\rho_{maks} = \dfrac{F_{max}.R_1}{fy} = \dfrac{0,455x19,125}{240} = 0,03626$

$\rho < \rho_{min} \rightarrow$ dipakai ρ_{min}

As = ρ.b.d = 5,83 x 10^{-3} x 1000 x 152 = 886,160 mm^2

Tulangan terpasang = Ø 16 – 225 (As = 894 mm^2)

Cek terhadap rasio tulangan:

$\rho = As_{terpasang}/b.d$ = 894/(1000x152) = 5,882 x 10^{-3}

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

❖ Tulangan Tumpuan Arah Y

Mu = 1276,071 kgm = 1,276 x 10^7 Nmm

Mn = Mu/φ = 1,276 x 10^7/0,8 = 1,595 x 10^7 Nmm

h = 200 mm

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

fy = 2400 kg/cm^2 = 240 N/mm^2

Dicoba tulangan pokok Ø 16 mm

d' = tebal selimut beton = 40 mm

d = h4 – d' – Ø tulangan arah X - Ø/2 = 200 – 40 – 16 – 16/2 = 136 mm

k = Mn/(b.d^2.R_1) → $R_l = \beta_1$.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 1,595 x 10^7/(1000 x 136^2 x 19,125) = 0,045

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0{,}045)}$ = 0,046

Fmax = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < Fmax →tulangan tunggal *under reinforced*

As = F.b.d.R_l/fy = (0,046 x 1000 x 136 x 19,125)/240 = 498,525 mm^2

ρ = As/(b.d) = 498,525 /(1000 x 136) = 3,666 x 10^{-3}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1$.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)

= 0,03629

$\rho < \rho_{min} \rightarrow$ dipakai ρ_{min}

As = ρ.b.d = 5,83 x 10^{-3} x 1000 x 136 = 792,880 mm^2

Tulangan terpasang = Ø 16 – 250 (As = 804 mm^2)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d = 804/(1000x136) = 5{,}912 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

5.7.3.2 Perhitungan Balok Lantai

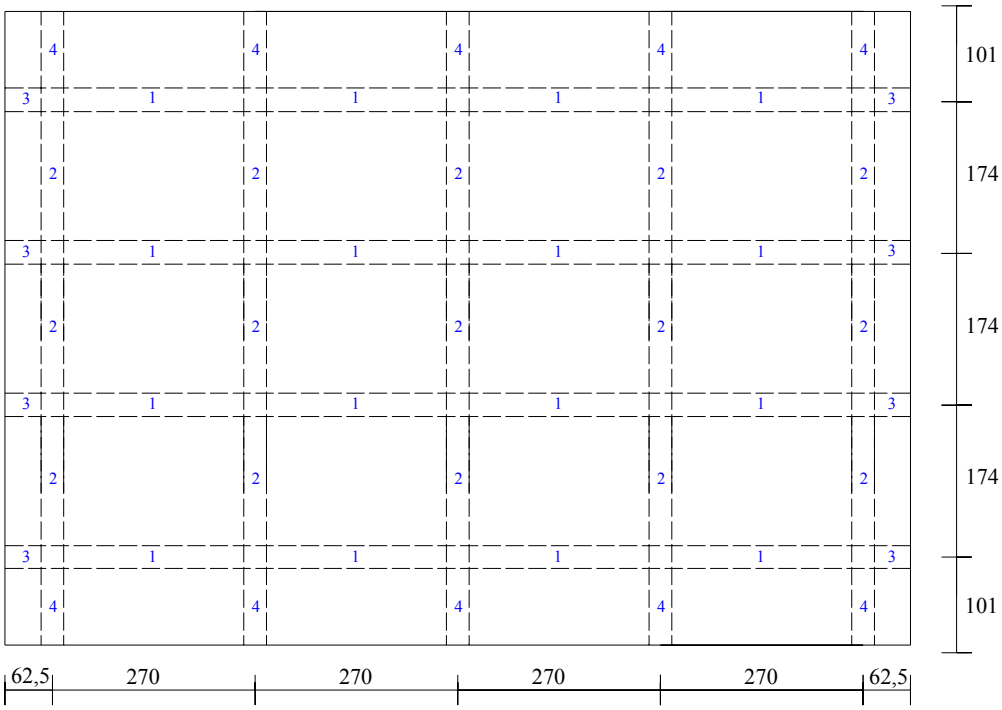
Diketahui panjang = 12,05 m

Direncanakan panjang balok memanjang = 2,700 (sama dengan jarak antar *counterfort*)

Ly = 2,700 m

Lx = 1,740 m

- Pendimensian Balok Lantai



Gambar 5.53 Denah Balok Lantai Kamar

Meskipun panjang atau bentang balok lantai berbeda-beda, yaitu ada 4 macam (lihat Gambar 5.53), akan tetapi di sini perhitungan dimensi hanya didasarkan pada balok lantai yang mempunyai bentang terpanjang, yaitu balok 1 (untuk balok memanjang) dan balok 2 (untuk balok melintang), dengan pertimbangan kemudahan pelaksanaan di lapangan. Dengan demikian, semua dimensi balok melintang maupun memanjang adalah sama.

- **Balok melintang**

  I = 174 cm = 1740 mm

  fy = 240 Mpa

  Faktor pengali (FP) $= (0{,}4 + \frac{fy}{700})$

  $= 0{,}743$

  $h_{min} = \frac{I}{28.FP} = \frac{1740}{28x0{,}743} = 83{,}638$ mm

  h = I/15 = 1740/15 = 116 mm

  Dipakai h = 450 mm

  b = 2/3.h = 2/3 x 450 = 300 mm

  Dimensi balok melintang = 450 x 300

- **Balok memanjang**

  I = 270 cm = 2700 mm

  fy = 240 Mpa

  Faktor pengali (FP) $= (0{,}4 + \frac{fy}{700})$

  $= 0{,}743$

  $h_{min} = \frac{I}{28.FP} = \frac{2700}{28x0{,}743} = 129{,}783$ mm

  h = I/15 = 2700/15 = 180 mm

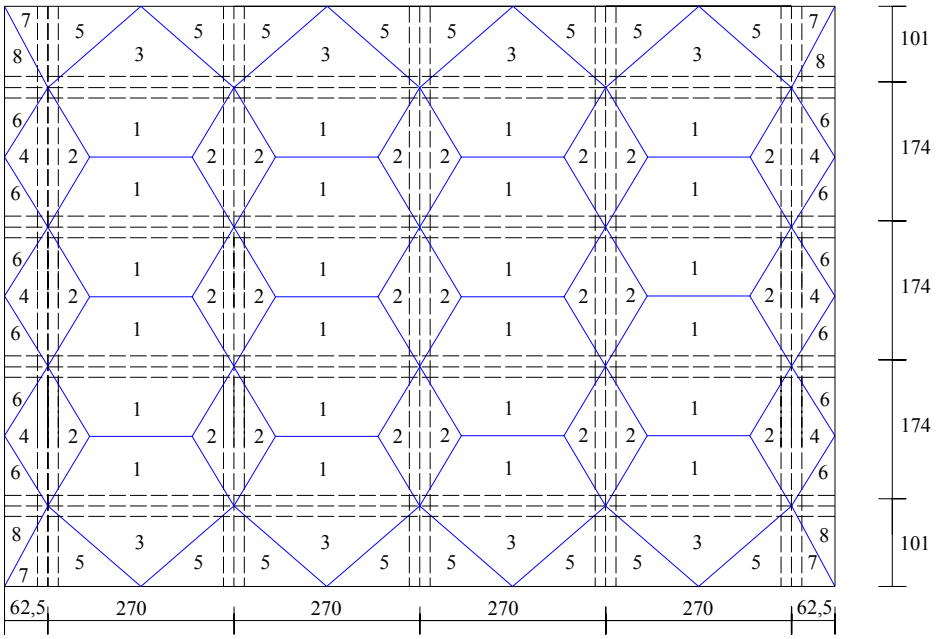
  Dipakai h = 450 mm

  b = 2/3.h = 2/3 x 450 = 300 mm

  Dimensi balok melintang = 450 x 300

➢ Pembebanan Balok Lantai

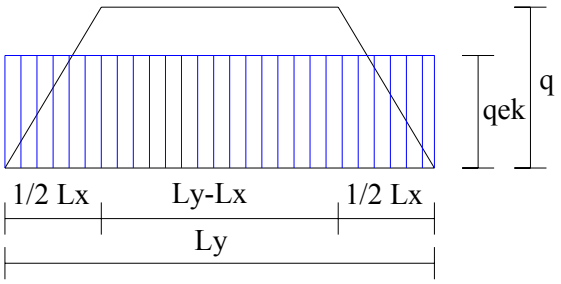
- Perataan beban



Gambar 5.54 Lay Out Balok Lantai Kamar serta Pola Pembebanan dengan Metode Amplop

Meskipun luasan segmen pelat berbeda-beda, yaitu ada 8 macam (lihat Gambar 5.54), akan tetapi untuk perataan beban hanya diambil segmen pelat dengan luasan terbesar, yaitu segmen 1 untuk perataan beban trapesium, dan segmen 2 serta segmen 3 untuk perataan beban segitiga.

<u>Beban Trapesium</u>



Gambar 5.55 Perataan Beban Trapesium

M_{max}Trapesium = 1/48.q.Lx.(3.Ly^2 – Lx^2)

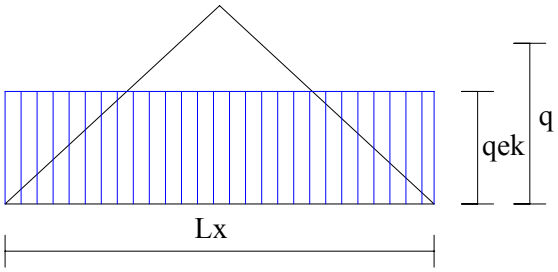
M_{max}Segi Empat = $1/8.q_{ek}.Ly^2$

M_{max}Trapesium = M_{max} Segi Empat

$1/48.q.Lx.(3.Ly^2 - Lx^2) = 1/8.\ q_{ek}.Ly^2$

$$q_{ek} = \frac{q.Lx.(3.Ly^2 - Lx^2)}{6.Ly^2}$$

<u>Beban Segitiga</u>



Gambar 5.56 Perataan Beban Segitiga

M_{max}Segitiga = $1/12.q.Lx^2$

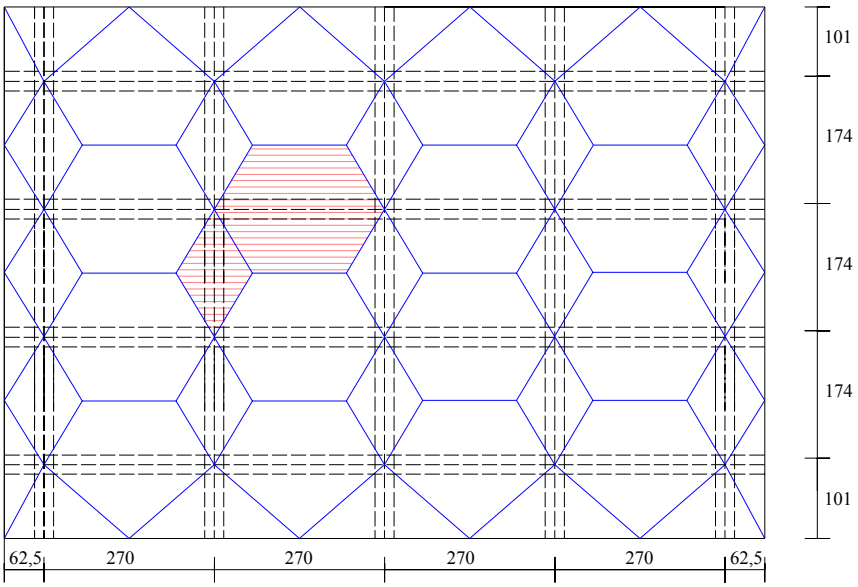
M_{max}Segi Empat = $1/8.q_{ek}.Ly^2$

M_{max}Segitiga = M_{max} Segi Empat

$1/12.q.Lx^2 = 1/8.q_{ek}.Ly^2$

$q_{ek} = 2/3.q$

- Pembebanan



Gambar 5.57 Pola Pembebanan Balok Lantai Kamar

q = 3480 kg/m^2 (lihat perhitungan pelat lantai)

- **Balok Melintang**

  Akibat beban pelat segitiga :

  q_{ek} = 2/3.q = 2/3 x 3480 = 2320 kg/m

  2 x q_{ek} = 2 x 2320 = 4640 kg/m

  Akibat berat sendiri balok :

  = 0,45 x 0,30 x 2400 = 324 kg/m

  $\mathbf{q_{total}}$ = 4640 + 324 = 4964 kg/m

- **Balok Memanjang**

  Akibat beban pelat trapesium :

  $$q_{ek} = \frac{q.Lx.(3.Ly^2 - Lx^2)}{6.Ly^2} = \frac{3480x1,74(3x2,700^2 - 1,74^2)}{6x2,700^2} = 2608,471 \text{ kg/m}$$

  2 x q_{ek} = 2 x 2608,471= 5216,942

  Akibat berat sendiri balok :

  = 0,45 x 0,30 x 2400 = 324 kg/m

  $\mathbf{q_{total}}$ = 5216,942 + 324 = 5540,942 kg/m

Asumsi : Balok menerus di atas dua tumpuan, sehingga :

M_{lap} = 1/16.q.L^2

M_{tump} = 1/11.q.L^2

Vu = ½ .q.L

➢ Pembebanan Balok Lantai

<u>Balok Melintang</u>

1. Tulangan Lentur Lapangan

   M_{lap} = 1/16.q.L^2 = 1/16 x 4964 x 1,74^2 =939,313 kgm= 9,393 x 10^6 Nmm

   Direncanakan menggunakan :

   Tulangan utama = Ø 16 mm

   Tulangan sengkang = Ø 8 mm

   fy = 240 Mpa = 240 N/mm^2 = 2400 kg/cm^2

   h = 450 mm

   b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

$Mn = Mu/\varphi = 9{,}393 \times 10^6 / 0{,}8 = 1{,}174 \times 10^7$ Nmm

$k = Mn/(b.d^2.R_l) \rightarrow R_l = \beta_1.f'c = 0{,}85 \times 22{,}5 = 19{,}125$ N/mm^2

$k = 1{,}174 \times 10^7/(300 \times 394^2 \times 19{,}125) = 0{,}013$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0{,}013)} = 0{,}013$

$F_{max} = \beta_1.450/(600 + fy) = (0{,}85 \times 450)/(600 + 240) = 0{,}455$

$k_{max} = (1 - ½.Fmax).Fmax = (1 - ½ \times 0{,}455) \times 0{,}455 = 0{,}351$

$F < F_{max}$ →tulangan tunggal *under reinforced*

$k < k_{max}$→ tidak perlu tulangan tekan, akan tetapi tetap direncanakan di daerah tekan terdapat tulangan praktis 2 buah diameter 16 mm dengan pertimbangan kemudahan pelaksanaan di lapangan. Maka perlu diperiksa apakah tulangan ini perlu diperhitungkan ataukah tidak.

$As' = 2 \times (1/4.\pi.\ Ø^2) = 2 \times (1/4 \times 3{,}14 \times 16^2) = 401{,}92$ mm^2

$As_2 = As' = 401{,}92$ mm^2

$M_2 = As'.fy.(d - d') = 401{,}92 \times 240 \times (394 - 40) = 34147123{,}2$ Nmm

$M_1 = Mn - M_2 = 1{,}174 \times 10^7 - 34147123{,}2 = -22407123{,}2$ Nmm

$M_1 \leq 0$ → maka As' dianggap 0, perhitungan tulangan tunggal :

$As = F.b.d.R_l/fy = (0{,}013 \times 300 \times 394 \times 19{,}125)/240 = 122{,}448$ mm^2

$\rho = As/(b.d) = 122{,}448/(300 \times 394) = 1{,}036 \times 10^{-4}$

$\rho_{min} = 1{,}4/ fy = 1{,}4/240 = 5{,}83 \times 10^{-3}$

$\rho_{maks} = 0{,}03635$ →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0{,}85 \times 450/(600+240) \times (19{,}125/240)$

$= 0{,}03629$

$\rho < \rho_{min}$ → dipakai ρ_{min}

$As = \rho.b.d = 5{,}83 \times 10^{-3} \times 300 \times 394 = 689{,}106$ mm^2

Tulangan terpasang = 4 Ø 16 (As = 804 mm^2)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d = 804/(300 \times 394) = 6{,}802 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks}$ → ok

2. Tulangan Lentur Tumpuan

M_{tump} =1/11.q.L^2 =1/11 x 4964 x $1,74^2$ =1366,273kgm=1,366 x 10^7 Nmm

Direncanakan menggunakan :

Tulangan utama = Ø 16 mm

Tulangan sengkang = Ø 8 mm

fy = 240 Mpa = 240 N/mm^2 = 2400 kg/cm^2

h = 450 mm

b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

Mn = Mu/φ = 1,366 x 10^7/0,8 = 1,708 x 10^7 Nmm

k = Mn/(b.d^2.R_1) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 1,708 x 10^7 /(300 x 394^2 x 19,125) = 0,019

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0,019)}$ = 0,019

F_{max} = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

k_{max} = (1 – ½.Fmax).Fmax = (1 – ½ x 0,455) x 0,455 = 0,351

F < F_{max} →tulangan tunggal *under reinforced*

k < k_{max}→ tidak perlu tulangan tekan, akan tetapi tetap direncanakan di daerah tekan terdapat tulangan praktis 2 buah diameter 16 mm dengan pertimbangan kemudahan pelaksanaan di lapangan. Maka perlu diperiksa apakah tulangan ini perlu diperhitungkan ataukah tidak.

As' = 2 x (1/4.π. $Ø^2$) = 2 x (1/4 x 3,14 x 16^2) = 401,92 mm^2

As_2 = As' = 401,92 mm^2

M_2 = As'.fy.(d – d') = 401,92 x 240 x (394 – 40) = 34147123,2 Nmm

M_1 = Mn – M_2 = 1,708 x 10^7 - 34147123,2 = -17067123,2 Nmm

$M_1 \leq 0$ → maka As' dianggap 0, perhitungan tulangan tunggal :

As = F.b.d.R_l/fy = (0,019 x 300 x 394 x 19,125)/240 = 178,962 mm^2

ρ = As/(b.d) = 178,962/(300 x 394) = 1,514 x 10^{-4}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)$

$= 0,03629$

$\rho < \rho_{min} \rightarrow$ dipakai ρ_{min}

$As = \rho.b.d = 5,83 \times 10^{-3} \times 300 \times 394 = 689,106\ mm^2$

Tulangan terpasang = 4 Ø 16 ($As = 804\ mm^2$)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d = 804/(300x394) = 6,802 \times 10^{-3}$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

3. Tulangan Sengkang

Vu = ½.q.L = ½ x 4964 x 1,74 = 4318,68 kg

Direncanakan menggunakan :

Tulangan sengkang = Ø 8 mm

$fy = 140\ Mpa = 140\ N/mm^2 = 1400\ kg/cm^2$

$f'c = 22,5\ Mpa = 22,5\ N/mm^2 = 225\ kg/cm^2$

h = 450 mm

b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

Di tumpuan Vu = 4318,68 kg

Pada jarak sejauh d dari tumpuan:

Vu = 4318,68 – q.d = 4318,68 – 4964 x 0,394 = 2362,864 kg

$Vn = Vu/\varphi = 2362,864/0,6 = 3938,107$ kg

$Vc = 0,17.b.d.\sqrt{f'c} = 0,17 \times 30 \times 39,4 \times \sqrt{225} = 3014,1$ kg

$(Vn – Vc) < 2/3.\sqrt{f'c}.b.d$

$(3938,107 – 3014,1) < 2/3 \times \sqrt{225} \times 30 \times 39,4$

924,007 < 11820 → penampang cukup ukurannya

$\varphi.(Vc/2) = 0,6 \times (3014,1/2) = 904,23$ kg

$Vu > \varphi.(Vc/2) \rightarrow$ perlu tulangan geser di sepanjang tumpuan sampai terjadinya $\varphi.(Vc/2)$

$\varphi.Vc = 0,6 \times 3014,1 = 1808,46$ kg

Vu > φ.Vc → perlu tulangan geser

Direncanakan menggunakan sengkang Ø 8 mm

Av = jumlah luas penampang dua kali sengkang

= 2 x (1/4 x 3,14 x 8^2) = 100,48 mm^2 = 1,0048 cm^2

S = jarak sengkang

= Av.d.fy/(Vn – Vc)

= 1,0048 x 39,4 x 1400/(3938,107 – 3014,1) = 59,983 cm

Syarat : S < d/2

59,983 < 39,4/2

59,983 > 19,7 → ditentukan S = 12 cm = 120 mm

Dipakai sengkang Ø 8 – 100 (tumpuan) dan Ø 8 – 140 (lapangan)

- Cek Terhadap Pengaruh Geser Lentur

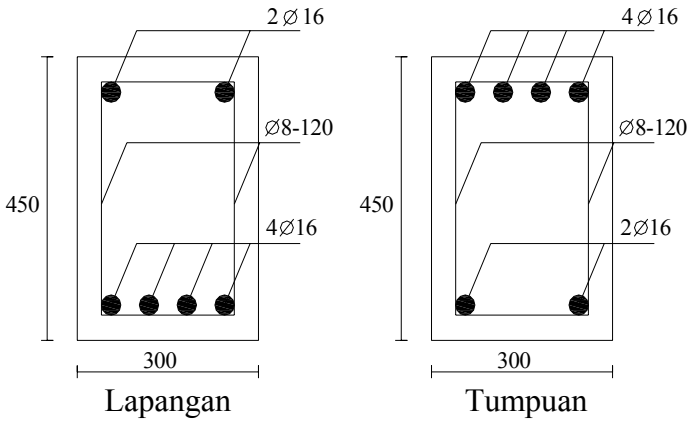
$$\tau = \frac{8.V}{7.b.h} = \frac{8x4318,68}{7x30x45} = 3,656 \text{ kg/cm}^2$$

Syarat :

$\tau \leq \tau_{bm}$

3,656 ≤ 1,35 $\sqrt{\sigma'_{bk}}$ = 1,35 x √225

3,656 ≤ 20,25 kg/cm^2 (aman)



Gambar 5.58 Penulangan Balok Melintang Lantai Kamar

<u>Balok Melintang</u>

1. Tulangan Lentur Lapangan

M_{lap} = 1/16.q.L^2 = 1/16 x 5540,942 x $2,700^2$ = 2524,592 kgm

= 2,525 x 10^7 Nmm

Direncanakan menggunakan :

Tulangan utama = Ø 16 mm

Tulangan sengkang = Ø 8 mm

fy = 240 Mpa = 240 N/mm^2 = 2400 kg/cm^2

h = 450 mm

b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

Mn = Mu/φ = 2,525 x 10^7/0,8 = 3,156 x 10^7 Nmm

k = Mn/(b.d^2.R$_1$) → R$_l$ = β$_1$.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 3,156 x 10^7/(300 x 394^2 x 19,125) = 0,035

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0{,}035)}$ = 0,036

F$_{max}$ = β$_1$.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

k$_{max}$ = (1 – ½.Fmax).Fmax = (1 – ½ x 0,455) x 0,455 = 0,351

F < F$_{max}$ →tulangan tunggal *under reinforced*

k < k$_{max}$→ tidak perlu tulangan tekan, akan tetapi tetap direncanakan di daerah tekan terdapat tulangan praktis 2 buah diameter 16 mm dengan pertimbangan kemudahan pelaksanaan di lapangan. Maka perlu diperiksa apakah tulangan ini perlu diperhitungkan ataukah tidak.

As' = 2 x (1/4.π. Ø^2) = 2 x (1/4 x 3,14 x 16^2) = 401,92 mm^2

As$_2$ = As' = 401,92 mm^2

M$_2$ = As'.fy.(d – d') = 401,92 x 240 x (394 – 40) = 34147123,2 Nmm

M$_1$ = Mn – M$_2$ = 3,156 x 10^7 - 34147123,2 = -2587123,2 Nmm

M$_1$ < 0 → maka As' dianggap 0, perhitungan tulangan tunggal :

As$_1$ = F.b.d.R$_l$/fy = (0,036x 300 x 394 x 19,125)/240 = 339,086 mm^2

ρ = As/(b.d) = 339,086/(300 x 394) = 2,869 x 10^{-3}

ρ$_{min}$ = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ$_{maks}$ = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ$_{maks}$ = β$_1$.450/(600 + fy).(R$_l$/fy) = 0,85x450/(600+240)x(19,125/240)

= 0,03629

$\rho < \rho_{min} \rightarrow$ dipakai ρ_{min}

As = ρ.b.d = 5,83 x 10^{-3} x 300 x 394 = 689,106 mm^2

Tulangan terpasang = 4 Ø 16 (As = 804 mm^2)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d$ = 804/(300x394) = 6,802 x 10^{-3}

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

2. Tulangan Lentur Tumpuan

M_{tump} = 1/11.q.L^2 = 1/11 x 5540,942 x $2,700^2$ = 3672,133 kgm

= 3,672 x 10^7 Nmm

Direncanakan menggunakan :

Tulangan utama = Ø 16 mm

Tulangan sengkang = Ø 8 mm

fy = 240 Mpa = 240 N/mm^2 = 2400 kg/cm^2

h = 450 mm

b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

Mn = Mu/φ = 3,672 x 10^7 /0,8 = 4,590 x 10^7 Nmm

k = Mn/(b.d^2.R_1) → $R_l = \beta_1$.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = 4,590 x 10^7/(300 x 394^2 x 19,125) = 0,052

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0,052)}$ = 0,053

$F_{max} = \beta_1$.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

k_{max} = (1 – ½.Fmax).Fmax = (1 – ½ x 0,455) x 0,455 = 0,351

F < F_{max} →tulangan tunggal *under reinforced*

k < k_{max}→ tidak perlu tulangan tekan, akan tetapi tetap direncanakan di daerah tekan terdapat tulangan praktis 2 buah diameter 16 mm dengan pertimbangan kemudahan pelaksanaan di lapangan. Maka perlu diperiksa apakah tulangan ini perlu diperhitungkan ataukah tidak.

As' = 2 x (1/4.π. $Ø^2$) = 2 x (1/4 x 3,14 x 16^2) = 401,92 mm^2

As_2 = As' = 401,92 mm^2

M_2 = As'.fy.(d – d') = 401,92 x 240 x (394 – 40) = 34147123,2 Nmm

M_1 = Mn – M_2 = 4,590 x 10^7 - 34147123,2 = 11752876,8 Nmm

M_1 > 0 → maka As' diperhitungkan

k = M_1/(b.d^2.R_1) = 11752876,8/(300 x 394^2 x 19,125) = 0,013

F = 1 - $\sqrt{1-2k}$ = 1 - $\sqrt{1-(2*0{,}013)}$ = 0,013

F_{max} = β_1.450/(600 + fy) = (0,85 x 450)/(600 + 240) = 0,455

F < F_{max} →tulangan tunggal *under reinforced*

As_1 = F.b.d.R_l/fy = (0,013x 300 x 394 x 19,125)/240 = 122448 mm^2

As = As_1 + As_2 = 122,448 + 401,92 = 524,368 mm^2

ρ = As/(b.d) = 524,368/(300 x 394) = 4,436 x 10^{-3}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ_{maks} = 0,03635 →(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau ρ_{maks} = β_1.450/(600 + fy).(R_l/fy) = 0,85x450/(600+240)x(19,125/240)

= 0,03629

ρ < ρ_{min} → dipakai ρ_{min}

As = ρ.b.d = 5,83 x 10^{-3} x 300 x 394 = 689,106 mm^2

Tulangan terpasang = 4 Ø 16 (As = 804 mm^2)

Cek terhadap rasio tulangan :

ρ = $As_{terpasang}$/b.d = 804/(300x394) = 6,802 x 10^{-3}

ρ_{min} < ρ < ρ_{maks} → ok

Periksa :

d'/d ≤ d'/d_{max}

40/394 ≤ 0,7.F_{max}

0,102 ≤ 0,7 x 0,455

0,102 ≤ 0,319 (ok)

3. Tulangan Sengkang

Vu = ½.q.L = ½ x 5540,942 x 2,700 = 7480,272 kg

Direncanakan menggunakan :

Tulangan sengkang = Ø 8 mm

fy = 140 Mpa = 140 N/mm^2 = 1400 kg/cm^2

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

h = 450 mm

b = 300 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 450 – 40 – ½ x 16 – 8 = 394 mm

Di tumpuan Vu = 7480,272 kg

Pada jarak sejauh d dari tumpuan:

Vu = 7480,272 – q.d = 7480,272 – 5540,942 x 0,394 = 5297,141 kg

Vn = Vu/φ = 5297,141/0,6 = 8828,568 kg

Vc = 0,17.b.d.√f'c = 0,17 x 30 x 39,4 x√225 = 3014,1 kg

(Vn – Vc) < 2/3.√f'c.b.d

(8828,568 – 3014,1) < 2/3 x √225 x 30 x 39,4

5814,468 < 11820 → penampang cukup ukurannya

φ.(Vc/2) = 0,6 x (3014,1/2) = 904,23 kg

Vu > φ.(Vc/2) → perlu tulangan geser di sepanjang tumpuan sampai terjadinya φ.(Vc/2)

φ.Vc = 0,6 x 3014,1 = 1808,46 kg

Vu > φ.Vc → perlu tulangan geser

Direncanakan menggunakan sengkang Ø 8 mm

Av = jumlah luas penampang dua kali sengkang

= 2 x (1/4 x 3,14 x 8^2) = 100,48 mm^2 = 1,0048 cm^2

S = jarak sengkang

= Av.d.fy/(Vn – Vc)

= 1,0048 x 39,4 x 1400/(8828,568 – 3014,1) = 9,532 cm

Syarat : S < d/2

9,532 < 39,4/2

9,532 < 19,7 (ok)

Dipakai sengkang Ø 8 – 100 (tumpuan) dan Ø 8 – 140 (lapangan)

- Cek Terhadap Pengaruh Geser Lentur

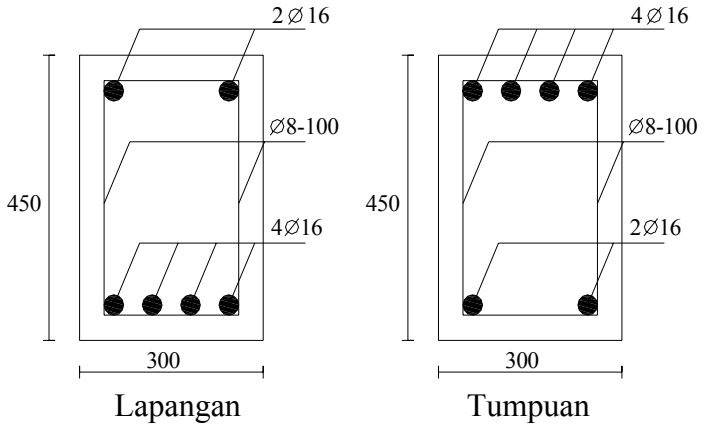
$$\tau = \frac{8.V}{7.b.h} = \frac{8x7480,272}{7x30x45} = 6,333 \text{ kg/cm}^2$$

Syarat :

$\tau \leq \tau_{bm}$

$6,333 \leq 1,35 \sqrt{\sigma'_{bk}} = 1,35 \text{ x } \sqrt{225}$

$6,333 \leq 20,25 \text{ kg/cm}^2$ (aman)

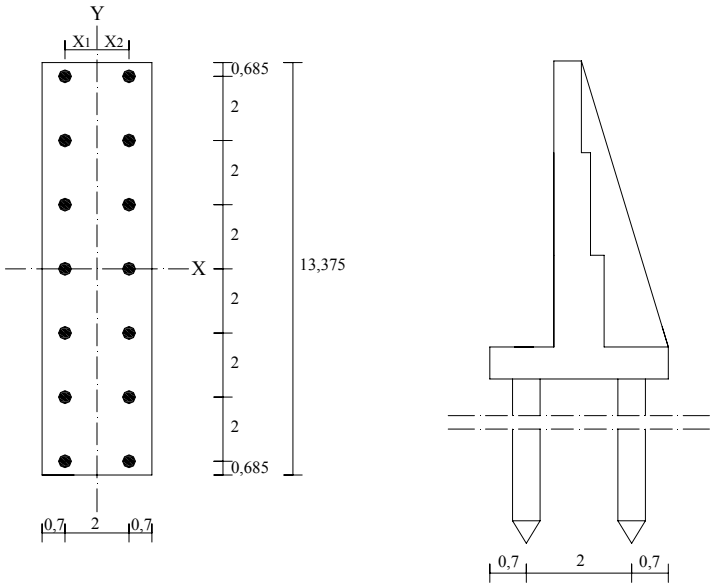


Gambar 5.59 Penulangan Balok Memanjang Kamar

5.8 Perhitungan Pondasi Tiang Pancang

5.8.1 Perhitungan Pondasi Gerbang A

5.8.1.1 Perhitungan Tiang Pancang Dinding Penahan Tanah



Gambar 5.60 Lay Out Tiang Pancang Dinding Gerbang A

S = jarak antar tiang pancang = 2,0 m

Lebar tapak (B) = 3,4 m

Panjang gerbang = 13,375 m

Tinggi dinding gerbang = 4,47 m

- Pembebanan Tiang Pancang
  - Momen terhadap tengah-tengah dasar konstruksi

    M = Σ G (x – B/2) – (Σ H.y)

    Di mana :

    $$x = \frac{\Sigma Mpasif}{\Sigma G} = \frac{42,991}{18,660} = 2,304$$

    $$y = \frac{\Sigma Maktif}{\Sigma H} = \frac{12,126}{6,803} = 1,782$$

    M = 18,66 x (2,304 – 3,4/2) – (6,803 x 1,782) = -0,853 tm
  - Besarnya momen dan beban yang bekerja sepanjang gerbang adalah :

    M_{tot} = M x (panjang gerbang)

    = 0,853 x 13,375 = 11,409 t

    G_{tot} = ΣG x (panjang gerbang)

    = 18,66 x 13,375 = 249,578

  Jumlah tiang (N) diambil = 7 baris (m) x 2 kolom (n) = 14 buah

  $X_1 = X_2 = 1,0$ m

  $\Sigma X^2 = 1,0^2 + 1,0^2 = 2,0$ m^2

  $$P_{max/min} = \frac{\Sigma G}{N} \pm \frac{M_{tot} \times X_{maks}}{ny\Sigma X^2} = \frac{249,578}{14} \pm \frac{11,409 \times 1}{7x2}$$

  P_{max} = 18,642 ton

  P_{min} = 17,012 ton

- Perhitungan Daya Dukung Tiang Pancang

Dimensi tiang pancang direncanakan = 40 x 40 cm

Kedalaman pemancangan direncanakan 10 m

$$P_{all} = \frac{q_c.A}{3} + \frac{f_c.O}{5}$$

Di mana :

A = luas penampang tiang = ¼ x π x 40^2 = 1256 cm^2 = 0,1256 m^2

O = keliling tiang = π x 40 = 125,6 cm = 1,256 m

Tabel 5.17 Nilai q_c dan f_c untuk Pondasi

Titik Sondir	Kedalaman (m)	q_c (t/m^2)	f_c (t/m)
S1	20	260	80
S2	20	90	65
S3	20	95	80

Sumber: Balai Dinas Pengelolaan Sumber Daya Air Seluna Kudus

Data yang dipakai adalah data sondir di titik S1 dengan qc dan fc yang terbesar.

$$P_{all} = \frac{260x0{,}1256}{3} + \frac{80x1{,}256}{5} = 30{,}981 \text{ t}$$

Berat sendiri tiang = 0,25 x π x $0{,}4^2$ x 10 x 2,4 = 3,014 t

Daya dukung tiang tunggal :

$P_{tiang\ tunggal}$ = P_{all} – berat sendiri = 30,981 – 3,014 = 27,967 t

$P_{tiang\ tunggal}$ > P_{max} = 27,967 t > 18,642 t (aman)

Daya dukung dalam kelompok tiang (*Pile Group*)

$$\text{Efisiensi } (\eta) = 1 - \frac{\theta}{90}\left\{\frac{(m-1)n + (n-1)m}{m.n}\right\}$$

Di mana :

θ = arc tan (d/s) = arc tan 40/200 = $11{,}31^0$

m = 7

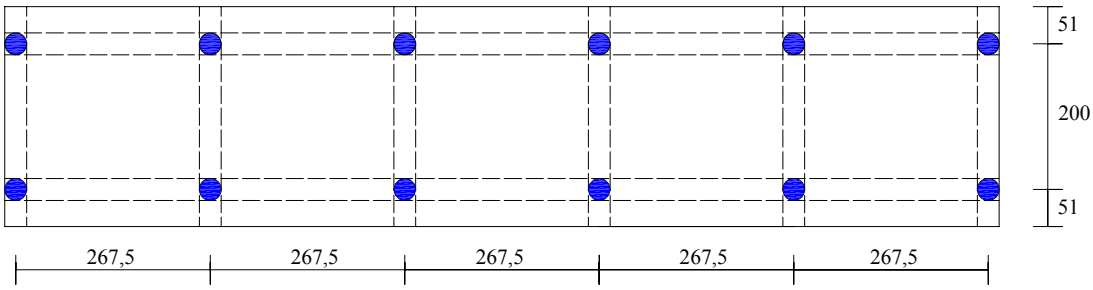
n = 2

$$\text{Efisiensi } (\eta) = 1 - \frac{11{,}31}{90}\left\{\frac{(7-1)2 + (2-1)7}{7x2}\right\} = 0{,}829$$

$P_{kelompok\ tiang}$ = $P_{tiang\ tunggal}$ x efisiensi = 27,967 x 0,829 = 23,185 t

$P_{kelompok\ tiang}$ > P_{max} = 23,185 t > 18,642 t (aman)

5.8.1.2 Perhitungan Tiang Pancang pada Lantai



Gambar 5.61 Penempatan Tiang Pancang Pelat Lantai Gerbang A

Lebar lantai = 3,02 m

Panjang lantai = 13,375 m

Direncanakan menggunakan pondasi tunggal sebanyak (N) = 12 buah

a. Pembebanan Tiang Pancang (Ditinjau Saat Kamar Kosong)

- Berat Pelat 1 = 0,12 x 3,02 x 6,715 x 2,4 = 5,840 t (↓)
- Berat Pelat 2 = 0,62 x 3,02 x 6,66 x 2,4 = 29,928 t (↓)
- Balok Melintang = 0,45 x 0,3 x 3,02 x 6 x 2,4 = 5,871 t (↓)
- Balok Memanjang = 0,45 x 0,3 x 13,375 x 2 x 2,4 = 8,667 t (↓)
- Gaya *Uplift* = 0,24 x 3,02 x 13,375 x 1 = 9,694 t (↑) +

ΣG = 40,612 t (↓)

$$P_{max} = \frac{\Sigma G}{N} = \frac{40{,}612}{12} = 3{,}384 \text{ t}$$

Dimensi tiang pancang direncanakan = 40 x 40 cm

Daya dukung tiang tunggal :

$P_{tiang\ tunggal} = P_{all}$ – berat sendiri = 30,981 – 3,014 = 27,967 t

$P_{tiang\ tunggal} > P_{max}$ = 27,967 t > 3,384 t (aman)

b. Pembebanan Tiang Pancang (Ditinjau Saat Kamar Berisi Air)

- Berat Pelat 1 = 0,12 x 3,02 x 6,715 x 2,4 = 5,840 t (↓)
- Berat Pelat 2 = 0,62 x 3,02 x 6,66 x 2,4 = 29,928 t (↓)
- Balok Melintang = 0,45 x 0,3 x 3,02 x 6 x 2,4 = 5,871 t (↓)
- Balok Memanjang = 0,45 x 0,3 x 13,375 x 2 x 2,4 = 8,667 t (↓)

- Berat Air = 3,12 x 3,02 x 13,375 x 1 = 126,025 t (↓)
- Gaya *Uplift* = 0,24 x 3,02 x 13,375 x 1 = 9,694 t (↑)+

ΣG =166,638 t (↓)

$$P_{max} = \frac{\Sigma G}{N} = \frac{166,638}{12} = 13,887 \text{ t}$$

Dimensi tiang pancang direncanakan = 40 x 40 cm

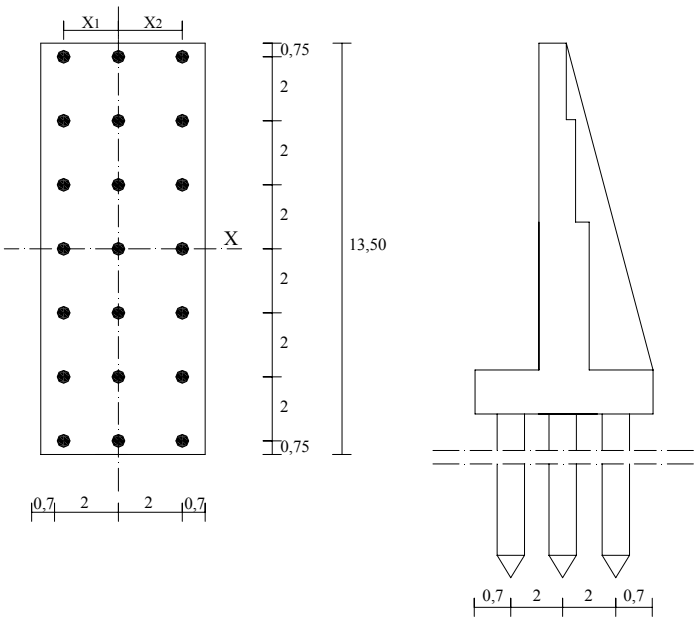
Daya dukung tiang tunggal :

$P_{tiang\ tunggal} = P_{all}$ – berat sendiri = 30,981 – 3,014 = 27,967 t

$P_{tiang\ tunggal} > P_{max}$ = 27,967 t > 13,887 t (aman)

5.8.2 Perhitungan Pondasi Gerbang B

5.8.2.1 Perhitungan Tiang Pancang Dinding Penahan Tanah



Gambar 5.62 Lay Out Tiang Pancang Dinding Gerbang B

S = jarak antar tiang pancang = 2,0 m

Lebar tapak (B) = 5,4 m

Panjang gerbang = 13,50 m

Tinggi dinding gerbang = 7,2 m

- Pembebanan Tiang Pancang
  - Momen terhadap tengah-tengah dasar konstruksi

    M = Σ G (x – B/2) – (Σ H.y)

    Di mana :

    $$x = \frac{\Sigma Mpasif}{\Sigma G} = \frac{167{,}035}{45{,}761} = 3{,}650$$

    $$y = \frac{\Sigma Maktif}{\Sigma H} = \frac{48{,}512}{21{,}643} = 2{,}241$$

    M = 45,761 x (3,65 – 5,4/2) – (21,643 x 2,241) = -5,029 tm
  - Besarnya momen dan beban yang bekerja sepanjang gerbang adalah :

    M_{tot} = M x (panjang gerbang)

    = 5,029 x 13,50 = 67,892 t

    G_{tot} = ΣG x (panjang gerbang)

    = 45,761 x 13,50 = 617,774 t

  Jumlah tiang (N) diambil = 7 baris (m) x 3 kolom (n) = 21 buah

  $X_1 = X_2 = 2{,}0$ m

  $\Sigma X^2 = 2{,}0^2 + 2{,}0^2 = 8{,}0$ m^2

  $$P_{max/min} = \frac{\Sigma G}{N} \pm \frac{M_{tot} \times X_{maks}}{ny\Sigma X^2} = \frac{617{,}774}{21} \pm \frac{67{,}892 \times 1}{7x8}$$

  P_{max} = 30,630 ton

  P_{min} = 28,205 ton

- Perhitungan Daya Dukung Tiang Pancang

Dimensi tiang pancang direncanakan = 50 x 50 cm

Kedalaman pemancangan direncanakan 10 m

$$P_{all} = \frac{q_c.A}{3} + \frac{f_c.O}{5}$$

Di mana :

A = luas penampang tiang = ¼ x π x 50^2 = 1962,5 cm^2 = 0,1963 m^2

O = keliling tiang = π x 50 = 157 cm = 1,57 m

q_c = 260 t/m^2

f_c = 80 t/m

$P_{all} = \frac{260x0{,}1963}{3} + \frac{80x1{,}57}{5} = 42{,}9333$ t

Berat sendiri tiang = 0,25 x π x $0{,}5^2$ x 10 x 2,4 = 4,710 t

Daya dukung tiang tunggal :

$P_{tiang\ tunggal} = P_{all}$ – berat sendiri = 42,9333 – 4,71 = 38,223 t

$P_{tiang\ tunggal} > P_{max}$ = 38,223 t > 30,630 t (aman)

Daya dukung dalam kelompok tiang (*Pile Group*)

Efisiensi (η) = $1 - \frac{\theta}{90}\left\{\frac{(m-1)n + (n-1)m}{m.n}\right\}$

Di mana :

θ = arc tan (d/s) = arc tan 40/200 = $11{,}31^0$

m = 7

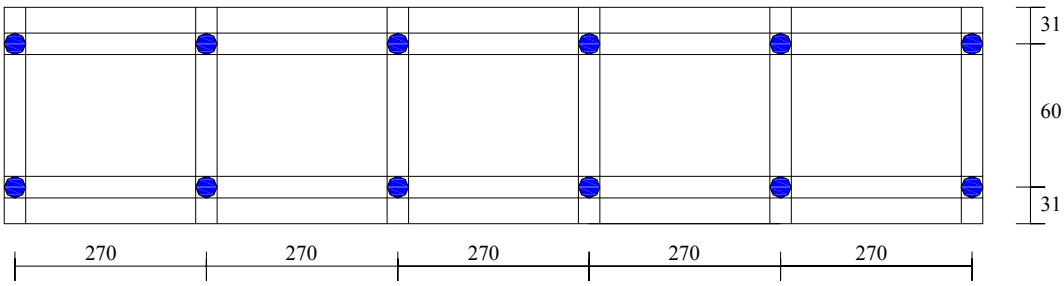
n = 3

Efisiensi (η) = $1 - \frac{11{,}31}{90}\left\{\frac{(7-1)3 + (3-1)7}{7x3}\right\}$=0,81

$P_{kelompok\ tiang} = P_{tiang\ tunggal}$ x efisiensi = 38,223 x 0,81 = 30,961 t

$P_{kelompok\ tiang} > P_{max}$ = 30,961 t > 30,630 t (aman)

5.8.2.2 Perhitungan Tiang Pancang pada Lantai



Gambar 5.63 Penempatan Tiang Pancang Pelat Lantai Gerbang B

Lebar lantai = 1,22 m

Panjang lantai = 13,50 m

Direncanakan menggunakan pondasi tunggal sebanyak (N) = 12 buah

a. Pembebanan Tiang Pancang (Ditinjau Saat Kamar Kosong)

- Berat Pelat 1 = 0,12 x 1,22 x 6,805 x 2,4 = 2,391 t (↓)
- Berat Pelat 2 = 0,62 x 1,22 x 6,695 x 2,4 = 12,154 t (↓)
- Balok Melintang = 0,45 x 0,3 x 1,22 x 6 x 2,4 = 2,372 t (↓)
- Balok Memanjang = 0,45 x 0,3 x 13,50 x 2 x 2,4 = 8,748 t (↓)
- Gaya *Uplift* = 2,74 x 1,22 x 13,50 x 1 = 45,128 t (↑) +

ΣG = -19,464 t (↑)

$$P_{max} = \frac{\Sigma G}{N} = \frac{19,464}{12} = 1,622 \text{ t}$$

Dimensi tiang pancang direncanakan = 40 x 40 cm

Daya dukung tiang tunggal :

$P_{tiang\ tunggal} = P_{all}$ – berat sendiri = 30,981 – 3,014 = 27,967 t

$P_{tiang\ tunggal} > P_{max}$ = 27,967 t > 1,622 t (aman)

b. Pembebanan Tiang Pancang (Ditinjau Saat Kamar Berisi Air)

- Berat Pelat 1 = 0,12 x 1,22 x 6,805 x 2,4 = 2,391 t (↓)
- Berat Pelat 2 = 0,62 x 1,22 x 6,695 x 2,4 = 12,154 t (↓)
- Balok Melintang = 0,45 x 0,3 x 1,22 x 6 x 2,4 = 2,372 t (↓)
- Balok Memanjang = 0,45 x 0,3 x 13,50 x 2 x 2,4 = 8,748 t (↓)
- Berat Air = 5,62 x 1,22 x 13,50 x 1 = 92,561 t (↓)
- Gaya *Uplift* = 2,74 x 1,22 x 13,50 x 1 = 45,128 t (↑) +

ΣG = 73,097 t (↓)

$$P_{max} = \frac{\Sigma G}{N} = \frac{73,097}{12} = 6,091 \text{ t}$$

Dimensi tiang pancang direncanakan = 40 x 40 cm

Daya dukung tiang tunggal :

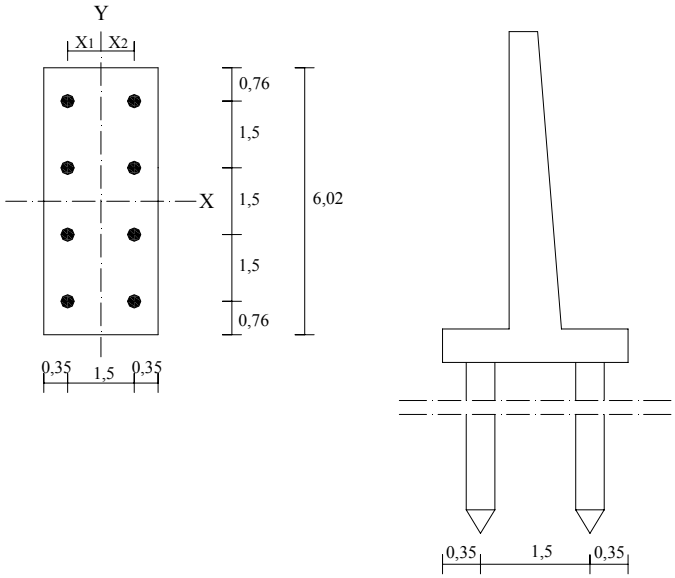
$P_{tiang\ tunggal} = P_{all}$ – berat sendiri = 30,981 – 3,014 = 27,967 t

$P_{tiang\ tunggal} > P_{max}$ = 27,967 t > 6,091 t (aman)

5.8.3 Perhitungan Pondasi Kamar

5.8.3.1 Perhitungan Pondasi Dinding Kamar

Perhitungan pondasi dinding kamar hanya dilakukan untuk bagian yang merupakan perbedaan ketinggian. Sedangkan dinding kamar yang lain pondasi yang dipakai diambil sama dengan pondasi pada gerbang B.



Gambar 5.64 Lay Out Tiang Pancang Dinding Beda Elevasi

S = jarak antar tiang pancang = 1,5 m

Lebar tapak (B) = 1,85 m

Panjang gerbang = 6,02 m

Tinggi dinding gerbang = 2,7 m

- Pembebanan Tiang Pancang
  - Momen terhadap tengah-tengah dasar konstruksi

    M = Σ G (x – B/2) – (Σ H.y)

    Di mana :

$$x = \frac{\Sigma Mpasif}{\Sigma G} = \frac{14{,}835}{11{,}125} = 1{,}333$$

$$y = \frac{\Sigma Maktif}{\Sigma H} = \frac{2{,}648}{3{,}713} = 0{,}713$$

M = 11,125 x (1,333 – 2,7/2) – (3,713 x 0,713) = -2,836 tm

- Besarnya momen dan beban yang bekerja sepanjang gerbang adalah :

M_{tot} = M x (panjang gerbang)

= 2,836 x 6,02 = 17,073 t

G_{tot} = ΣG x (panjang gerbang)

= 11,125 x 6,02 = 66,973 t

Jumlah tiang (N) diambil = 4 baris (m) x 2 kolom (n) = 8 buah

$X_1 = X_2 = 0,75$ m

$\Sigma X^2 = 0,75^2 + 0,75^2 = 1,125\ m^2$

$$P_{max/min} = \frac{\Sigma G}{N} \pm \frac{M_{tot} \times X_{maks}}{ny\Sigma X^2} = \frac{66,973}{8} \pm \frac{17,073 \times 1}{4x1,125}$$

P_{max} = 12,166 ton

P_{min} = 4,578 ton

- Perhitungan Daya Dukung Tiang Pancang

Dimensi tiang pancang direncanakan = 40 x 40 cm

Kedalaman pemancangan direncanakan 10 m

$$P_{all} = \frac{q_c.A}{3} + \frac{f_c.O}{5}$$

Di mana :

A = luas penampang tiang = ¼ x π x 40^2 = 1256 cm^2 = 0,1256 m^2

O = keliling tiang = π x 40 = 125,6 cm = 1,256 m

q_c = 260 t/m^2

f_c = 80 t/m

$$P_{all} = \frac{260x0,1256}{3} + \frac{80x1,256}{5} = 30,981 \text{ t}$$

Berat sendiri tiang = 0,25 x π x $0,4^2$ x 10 x 2,4 = 3,014 t

Daya dukung tiang tunggal :

$P_{tiang\ tunggal} = P_{all}$ – berat sendiri = 30,981 – 3,014 = 27,967 t

$P_{tiang\ tunggal} > P_{max}$ = 27,967 t > 12,166 t (aman)

Daya dukung dalam kelompok tiang (*Pile Group*)

Efisiensi (η) = $1 - \frac{\theta}{90}\left\{\frac{(m-1)n+(n-1)m}{m.n}\right\}$

Di mana :

θ = arc tan (d/s) = arc tan 40/150 = $14{,}93^0$

m = 4

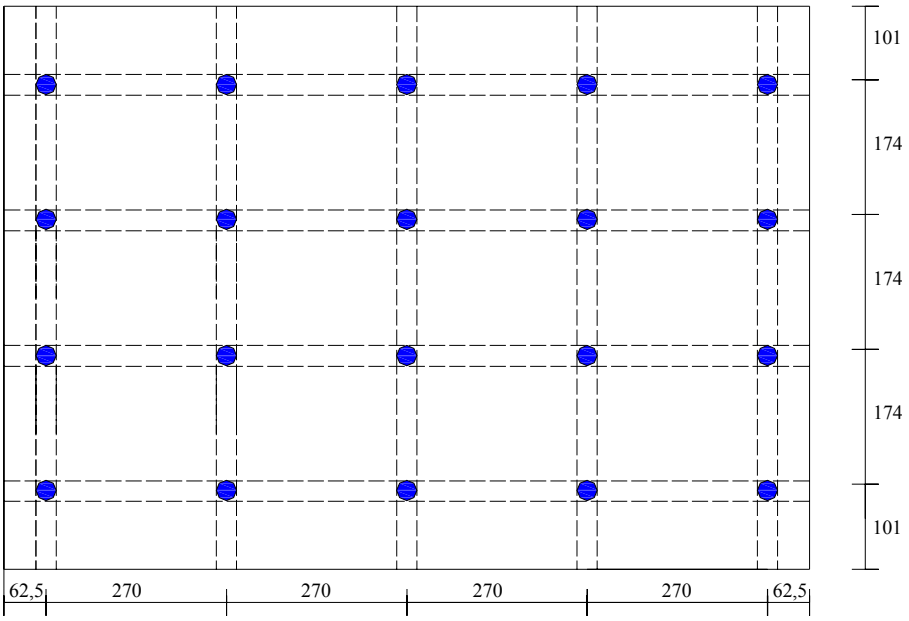
n = 2

Efisiensi (η) = $1 - \frac{14{,}93}{90}\left\{\frac{(4-1)2+(2-1)4}{4x2}\right\}$=0,793

$P_{kelompok\ tiang}$ = $P_{tiang\ tunggal}$ x efisiensi = 27,967 x 0,793 = 22,178 t

$P_{kelompok\ tiang}$ > P_{max} = 22,178 t > 12,116 t (aman)

5.8.3.2 Perhitungan Pondasi Pelat Lantai Kamar



Gambar 5.65 Penempatan Tiang Pancang Pelat Lantai Kamar

Lebar lantai = 7,24 m

Panjang lantai = 12,05 m

Direncanakan menggunakan pondasi tunggal sebanyak (N) = 24 buah

a. Pembebanan Tiang Pancang (Ditinjau Saat Kamar Kosong)

- Berat Pelat = 0,20 x 7,24 x 16,85 x 2,4 = 58,557 t (↓)
- Balok Melintang = 0,45 x 0,3 x 7,24 x 6 x 2,4 = 14,075 t (↓)
- Balok Memanjang = 0,45 x 0,3 x 16,85 x 4 x 2,4 = 21,838 t (↓)
- Gaya *Uplift* = 2,7 x 7,24 x 16,85 x 1 = 329,384 t (↑) +

ΣG =-234,914 t (↑)

$$P_{max} = \frac{\Sigma G}{N} = \frac{234{,}914}{24} = 9{,}788 \text{ t}$$

Dimensi tiang pancang direncanakan = 40 x 40 cm

Daya dukung tiang tunggal :

$P_{tiang\ tunggal} = P_{all}$ – berat sendiri = 30,981 – 3,014 = 27,967 t

$P_{tiang\ tunggal} > P_{max}$ = 27,967 t > 9,788 t (aman)

b. Pembebanan Tiang Pancang (Ditinjau Saat Kamar Berisi Air)

- Berat Pelat = 0,20 x 7,24 x 16,85 x 2,4 = 58,557 t (↓)
- Balok Melintang = 0,45 x 0,3 x 7,24 x 6 x 2,4 = 14,075 t (↓)
- Balok Memanjang = 0,45 x 0,3 x 16,85 x 4 x 2,4 = 21,838 t (↓)
- Berat Air = 5,62 x 16,85 x 7,24 x 1 = 685,606 t (↓)

- Gaya *Uplift* = 2,7 x 7,24 x 16,85 x 1 = 329,384 t (↑) +

ΣG = 450,692 t (↓)

$$P_{max} = \frac{\Sigma G}{N} = \frac{450{,}692}{24} = 18{,}779 \text{ t}$$

Dimensi tiang pancang direncanakan = 40 x 40 cm

Daya dukung tiang tunggal :

$P_{tiang\ tunggal} = P_{all}$ – berat sendiri = 30,981 – 3,014 = 27,967 t

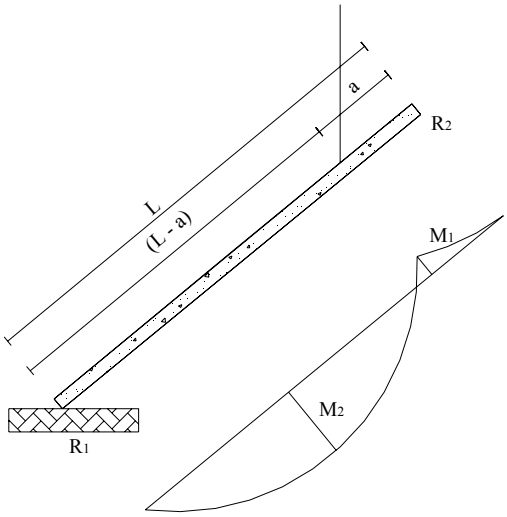
$P_{tiang\ tunggal} > P_{max}$ = 27,967 t > 18,779 t (aman)

5.8.4 Penulangan Tiang Pancang

- Dimensi 40 x 40 cm

Panjang tiang (L) = 10 m

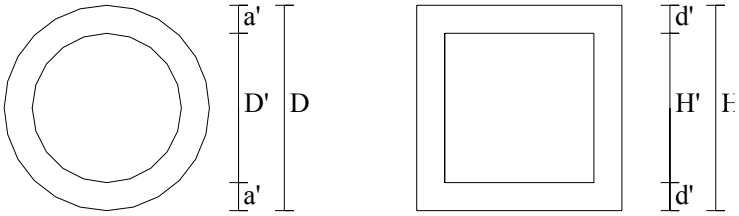
Penulangan tiang pancang dihitung berdasarkan kebutuhan saat pengangkatan pada satu titik tumpu (pendirian tiang pancang) karena menghasilkan momen yang lebih besar dari pada pengangkatan pada dua titik tumpu (lihat Bab II).



Gambar 5.66 Pendirian Tiang Pancang

- q = berat tiang pancang = ¼.π.(0,4^2).(2,4) = 0,3014 t/m
- a = 0,29 L = 0,29 x 10 = 2,9 m
- $M_1 = M_2$ = ½ x q x a^2 = ½ x 0,3014 x 2,9^2 = 1,267 tm

Perhitungan menggunakan penampang bujur sangkar yang telah dikorelasikan terhadap penampang lingkaran :



Gambar 5.67 Korelasi Penampang Lingkaran menjadi Persegi

D' = D – 2a' ⟶ H' = 0,88 (D – 2a')

Perhitungan tulangan :

H' = 0,88D – 0,88 x 2 a' = 0,88 x 40 – 0,88 x 2 x 5 = 26,4 cm

= H – 2d'

26,4 = H – 2 x 5

H = 36,4 cm

B = 364 mm

d = 364 – t selimut – Ø sengkang – ½ tul. Pokok

= 364 – 50 – 8 – ½ x 16 = 298 mm

Mu= 1,267 tm = 1,267 x 10^7 Nmm

Mn= 1,267 x 10^7/0,8 = 1,584 x 10^7 Nmm

k = $\frac{Mn}{(b.d^2.R_1)}$; $R_l = \beta_1.f'c$ = 0,85 x 22,5 = 19,125 N/mm^2

k = $\frac{1{,}584x10^7}{(364x298^2x19{,}125)}$ = 0,026

F = 1 - $\sqrt{1-2k}$ = 0,026

As = F.b.d.R_l/fy = (0,026 x 364 x 298 x 19,125)/240 = 224,740 mm^2

ρ = As/(b.d) = 224,740/(364 x 298) = 2,072 x 10^{-3}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ < ρ_{min} → dipakai ρ_{min}

As_{min} = ρ_{min}.b.d = 5,83 x 10^{-3} x 364 x 298 = 632,392 mm^2

Dipakai tulangan pokok 4 Ø 16 (As = 804 mm^2)

<u>Check terhadap geser</u>

$$Vu = \frac{q.(L^2 - 2La)}{2(L-a)} = \frac{0{,}3014(10^2 - 2x10x0{,}29x10)}{2(10-0{,}29x10)} = 0{,}891465 \text{ t} = 8914{,}65 \text{ N}$$

Vn = Vu/ Ø = 8914,65/0,6 = 14857,75 N

ØVc/2 = (0,6 x 0,17 √f'c x b x d)/2

= (0,6 x 0,17 √22,5 x 364 x 298)/2

= 26240,922 N

Vu < ØVc/2 → tidak perlu tulangan geser

Untuk keperluan pengikatan tulangan utama, dipakai tulangan spiral Ø 8 – 150

- Dimensi 50 x 50 cm

Panjang tiang (L) = 10 m

Penulangan tiang pancang dihitung berdasarkan kebutuhan saat pengangkatan pada satu titik tumpu (pendirian tiang pancang) karena menghasilkan momen yang lebih besar dari pada pengangkatan pada dua titik tumpu (lihat Bab II).

- q = berat tiang pancang = ¼.π.(0,5^2).(2,4) = 0,471 t/m
- a = 0,29 L = 0,29 x 10 = 2,9 m
- $M_1 = M_2$ = ½ x q x a^2 = ½ x 0,471 x 2,9^2 = 1,981 tm

Perhitungan menggunakan penampang bujur sangkar yang telah dikorelasikan terhadap penampang lingkaran :

D' = D – 2a' ⟶ H' = 0,88 (D – 2a')

Perhitungan tulangan :

H' = 0,88D – 0,88 x 2 a' = 0,88 x 50 – 0,88 x 2 x 5 = 35,2 cm

= H – 2d'

35,2 = H – 2 x 5

H = 45,2 cm

B = 452 mm

d = 452 – t selimut – Ø sengkang – ½ tul. Pokok

= 452 – 50 – 8 – ½ x 16 = 386 mm

Mu= 1,981 tm = 1,981 x 10^7 Nmm

Mn= 1,981 x 10^7/0,8 = 2,476 x 10^7 Nmm

k = $\frac{Mn}{(b.d^2.R_1)}$; $R_1 = \beta_1$.f'c = 0,85 x 22,5 = 19,125 N/mm^2

k = $\frac{2{,}476x10^7}{(452x386^2x19{,}125)}$ = 0,019

F = 1 - $\sqrt{1-2k}$ = 0,019

As = F.b.d.R_l/fy = (0,019 x 452 x 386 x 19,125)/240 = 264,162 mm^2

ρ = As/(b.d) = 264,162/(452 x 386) = 1,514 x 10^{-3}

ρ_{min} = 1,4/ fy = 1,4/240 = 5,83 x 10^{-3}

ρ < ρ_{min} → dipakai ρ_{min}

$As_{min} = \rho_{min}$.b.d = 5,83 x 10^{-3} x 452 x 386 = 1017,172 mm^2

Dipakai tulangan pokok 6 Ø 16 (As = 1206 mm^2)

<u>Check terhadap geser</u>

$$Vu = \frac{q.(L^2 - 2La)}{2(L-a)} = \frac{0{,}471(10^2 - 2x10x0{,}29x10)}{2(10-0{,}29x10)} = 1{,}393099 \text{ t} = 13930{,}99 \text{ N}$$

Vn = Vu/ Ø = 13930,99 /0,6 = 23218,3167 N

ØVc/2 = (0,6 x 0,17 √f'c x b x d)/2

= (0,6 x 0,17 √22,5 x 452 x 386)/2

= 42207,261 N

Vu < ØVc/2 → tidak perlu tulangan geser

Untuk keperluan pengikatan tulangan utama, dipakai tulangan spiral Ø 8 – 150

5.9 Perhitungan Pipa Pengisian/Pengosongan Kamar

Untuk pengisian dan pengosongan kamar tidak menggunakan pompa, tetapi menggunakan gaya gravitasi dengan memanfaatkan beda elevasi muka air. Pengisian/pengosongan dapat dilakukan melalui lubang yang dapat diletakkan pada pintu atau dinding gerbang tergantung dengan luasan kamar yang dilayani.

A. Ruang Pintu

- Dimensi

W gerbang = w + 2a = 6,02 m

L pintu gerbang A = 3,5 m

L pintu gerbang B = 3,57 m

- Perhitungan :

Luas ruang pintu gerbang A (F_1) = (2g + 2t + m + L sin 45^0) x W gerbang

= (2.0,205 + 2.1 + 2,5 + 3,5sin45^0) x 6,02

= 44,457 m^2

Luas ruang pintu gerbang B (F_2) = (2g + 2t + m + L – L sin 45^0) x W gerbang

= (2.0,26 + 2.1 + 2,5 + 3,57 – 3,57sin45^0) x 6,02

= 36,515 m^2

B. Luas Kamar

W kamar = 12,04 m

Panjang kamar (Lk) = 16,85 m

Luas kamar (F_3) = W kamar x Lk = 12,04 x 16,85 = 202,874 m^2

C. Perhitungan Pipa Pengosongan/Pengisian Kamar

Direncanakan waktu pengisian/pengosongan kamar ± 10 menit (T = 600 detik) untuk semua saluran, kemudian perhitungan selanjutnya adalah untuk menentukan diameter pipa saluran pengisian/pengosongan kamar untuk pelayanan kapal.

Beda tinggi muka air gerbang dengan kamar (Δ H) = (+16,00) – (+13,50)

= 2,50 m

- Perhitungan

Luas lubang pengisian/pengosongan (Fk) = $F_1 + F_2 + F_3$

= 44,457 + 36,515 + 202,874

= 283,846 m^2

Direncanakan akan dipindahkan dalam waktu T = 600 detik, sehingga :

a. Apabila Menggunakan Pipa dalam Dinding Gerbang

$$T = \frac{2.Fk.\sqrt{h}}{\mu.a.\sqrt{2g}}$$

$$600 = \frac{2x283{,}846\sqrt{2{,}5}}{0{,}62xax\sqrt{2x9{,}81}}$$

a = 0,545 m^2

Direncanakan menggunakan 2 pasang pipa (4 buah)

A = 4 (1/4.π.D^2)

0,545 = 4 x (1/4 x 3,14 x D^2)

D^2 = 0,174 m

D = 41,7 cm; diambil diameter pipa baja = 45 cm

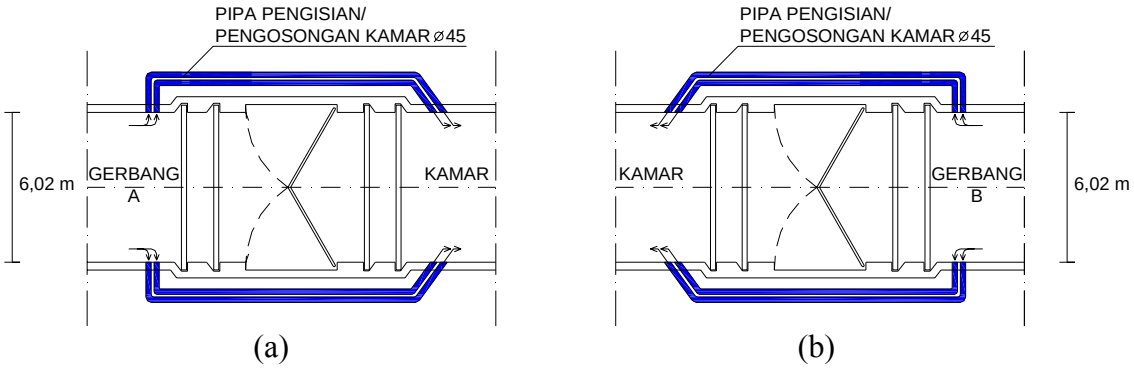
b. Apabila Lubang Pengisian/Pengosongan pada Pintu

$$T = \frac{2.Fk.\sqrt{h}}{\mu.a.\sqrt{2g}}$$

$$600 = \frac{2x283{,}846\sqrt{2{,}5}}{0{,}32xax\sqrt{2x9{,}81}}$$

a = 1,055 m^2 → Ukuran lubang = 1,03 x 1,03 m (terlalu besar dan membahayakan kapal).

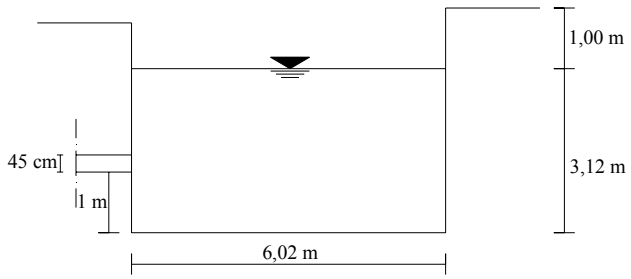
Sehingga digunakan lubang pengisian/pengosongan melalui pipa dalam dinding gerbang.



Gambar 5.68 Lubang Pengisian/Pengosongan (a) Gerbang A, (b) Gerbang B

D. Perhitungan Plat Penutup Lubang Pengisian/Pengosongan

- Pada Gerbang A



Gambar 5.69 Lay Out Lubang Pengisian Gerbang A

Perhitungan menggunakan rumus *Bach* dengan letak lubang pengisian setinggi 1 m dari dasar gerbang A.

Perhitungan :

$$\sigma_{maks} = 1/2.k.P.\frac{a^2.b^2}{\left(a^2+b^2\right)t^2}$$

a = lebar pelat = 130 cm

b = panjang pelat = 175 cm

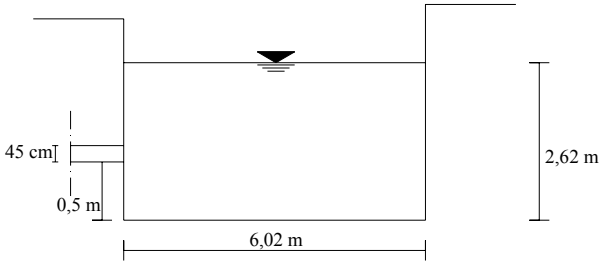
P = tekanan air = ½ . (1). (3,12 – 1) = 1,06 t/m^2 = 0,106 kg/cm^2

$$1400 = 1/2x0{,}8x0{,}106x\frac{130^2 x175^2}{\left(130^2+175^2\right)t^2}$$

$t^2 = 0,329$

t = 0,574 cm ; diambil tebal plat pintu 6 mm

- Pada Gerbang B



Gambar 5.70 Lay Out Lubang Pengosongan Gerbang B

Letak lubang pengosongan terletak setinggi 0,5 m dari dasar gerbang B.

P = tekanan air = ½ . (1). (2,62 – 0,5) = 1,06 t/m^2 = 0,106 kg/cm^2

$$1400 = 1/2x0,8x0,106x\frac{130^2 x175^2}{\left(130^2 + 175^2\right)t^2}$$

$t^2 = 0,329$

t = 0,574 cm ; diambil tebal plat pintu 6 mm

5.10 Perhitungan Rembesan

Rembesan yang diperhitungkan adalah rembesan air di bawah tanah yang dapat mengakibatkan penggerusan terhadap lantai, sedangkan untuk rembesan samping tidak diperhitungkan karena bangunan menggunakan dinding beton. Perhitungan rembesan ini adalah untuk memeriksa apakah panjang (L_H) konstruksi lantai mencukupi atau tidak dari pengaruh penggerusan dengan berdasarkan teori *Lane* seperti berikut :

$$C = \frac{1/3\,L_H + L_V}{H} > \hat{C}$$

$$C = \frac{1/3\,x13,375 + 0}{2} = 2,229 < \hat{C} = 8,5 \text{ (tidak aman)}$$

Sehingga dapat dilakukan alternatif penanganan :

1. Memperpanjang lintasan vertikal dengan konstruksi *sheet pile*/turap

   $1/3.L_H + L_V > C.H$

$1/3 \times 13{,}375 + L_V > 8{,}5 \times 2$

$L_V > 12{,}542$ m

Berarti panjang *sheet pile*/turap yang dibutuhkan minimal sepanjang 12,542 m.

2. Memperpanjang lintasan horisontal dengan menambah panjang pelat lantai

   $L_{H\ total} = 13{,}375 + 16{,}85 + 13{,}5 = 43{,}725$ m

   $L_V = 2{,}5$ m (akibat adanya dinding beda elevasi)

   $C = \dfrac{1/3\, x 43{,}725 + 2{,}5}{2} = 8{,}6 > \hat{C} = 8{,}5$

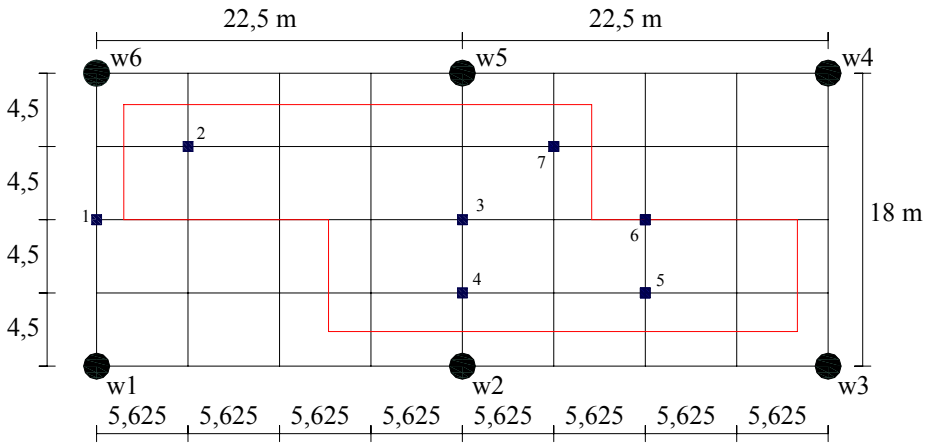
Dengan pertimbangan untuk kemudahan pelaksanaan, yaitu apabila menggunakan konstruksi *sheet pile* / turap yang terlalu panjang (> 12,542 m) maka akan sulit dalam pemasangannya. Oleh karena itu dipilih dengan menggunakan pelat lantai pada bagian kamar untuk memperpanjang lintasan horizontal yang dalam pelaksanaannya lebih mudah dilakukan.

5.11 Dewatering

Dewatering disini digunakan untuk menurunkan muka air tanah disekitar lokasi pekerjaan. Penurunan ini dilakukan untuk memudahkan proses pelaksanaan konstruksi dan juga untuk keamanan. Dalam perencanaan *dewatering* dilakukan dengan pompa air bertenaga diesel yang disesuaikan dengan muka air tanah yang harus dikurangi.

Perhitungan *dewatering* dilakukan hanya untuk menurunkan muka air di sekitar galian dengan memasang sumur-sumur pompa di sekeliling galian untuk memompa air keluar dari tanah, hingga muka air tanah berada di bawah galian. Ketinggian muka air tanah yang diinginkan ádalah -1,5 m di bawah galian.

- Perhitungan



Gambar 5.71 Rencana Lokasi Sumur Pompa Dan Titik Yang Ditinjau

Kedalaman galian = 6,9 m

Muka air tanah = 4 m di bawah pernukaan tanah

Muka air tanah diturunkan -1,5 m dari kedalaman galian, sehingga :

Tinggi penurunan muka air rencana (S) = (6,9 + 1,5) – 4 = 4,4 m

Permeabilitas tanah (k) = 10^{-4} m/dt

Re = R (jari-jari pengaruh) = 3000 x S x √k = 3000 x 4,4 x 10^{-4} = 132 m

T = k = 10^{-4} m/dt → aquifer bebas

Tabel 5.18 Pengaruh Sumur Pompa

Jarak sumur dengan titik	W1	W2	W3	W4	W5	W6	$\ln\left\{\frac{r1 \times r2 \times ..rn}{\mathrm{Re}^n}\right\}$
1	9	24,23	45,89	45,89	24,23	9	-10,9
2	14,63	21,61	41,63	39,63	17,46	7,2	-11,3
3	24,23	9	24,23	24,23	9	24,23	-12,2
4	22,95	4,5	22,95	26,24	13,5	26,24	-12,4
5	34,05	12,12	12,12	17,57	17,57	36,35	-11,5
6	34,93	14,41	14,41	14,41	14,41	34,93	-11,5
7	31,19	14,63	21,61	17,46	7,2	28,48	-11,9

Draw down terkecil pada titik 1 yaitu -10,9 m

Apabila penurunan air dipertahankan pada kedalaman 4 m dari muka air tanah semula, maka debit pompa dapat diperhitungkan sebagai berikut :

$$S = -\frac{Q_O}{2\pi.T}\ln\left(\frac{r}{R}\right)$$

Di mana nilai $\ln\left(\frac{r}{R}\right)$ = -10,9 m, sehingga :

$$4{,}4 = -\frac{Q_O}{2x3{,}14x10^{-4}}x(-10{,}9)$$

Q_O = 2,54 x 10^{-4} m^3/dt = 15,21 liter/menit

Jadi untuk keperluan *dewatering* digunakan pompa dengan kapasitas debit 15 liter/menit, sejumlah 6 sumur.

5.12 Perhitungan Bolder

Bolder yang digunakan pada perencanaan Saluran Pintu Air ini menggunakan bahan dari beton bertulang dengan bentuk silinder dan *casing* baja dengan ketebalan 3 mm. Besarnya gaya tarik kapal yang bekerja pada bolder direncanakan sebesar 7 ton.

Perhitungan tulangan sebagai balok persegi :

1. Tulangan Lentur

   Mu = F x 0,30 = 7 x 0,30 = 2,1 tm = 2,1 x 10^3 kgm = 2,1 x 10^7 Nmm

   Tulangan utama = Ø 16 mm

   Tulangan sengkang = Ø 8 mm

   fy = 240 Mpa = 240 N/mm^2 = 2400 kg/cm^2

   h = 200 mm

   b = 200 mm

   d' = tebal selimut beton = 40 mm

   d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

   = 200 – 40 – ½ x 16 – 8 = 144 mm

   Mn = Mu/φ = 2,1 x 10^7/0,8 = 2,625 x 10^7 Nmm

   k = Mn/(b.d^2.R_1) → R_l = β_1.f'c = 0,85 x 22,5 = 19,125 N/mm^2

   k = 2,625 x 10^7/(200 x 144^2 x 19,125) = 0,33

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0{,}33)} = 0{,}417$

$F_{max} = \beta_1.450/(600 + fy) = (0{,}85 \times 450)/(600 + 240) = 0{,}455$

$k_{max} = (1 - ½.Fmax).Fmax = (1 - ½ \times 0{,}455) \times 0{,}455 = 0{,}351$

$k < k_{max} \rightarrow$ tidak perlu tulangan tekan, akan tetapi tetap direncanakan di daerah tekan terdapat tulangan praktis 2 buah diameter 16 mm dengan pertimbangan kemudahan pelaksanaan di lapangan. Maka perlu diperiksa apakah tulangan ini perlu diperhitungkan ataukah tidak.

$As' = 2 \times (1/4.\pi.\ Ø^2) = 2 \times (1/4 \times 3{,}14 \times 16^2) = 401{,}92\ mm^2$

$As_2 = As' = 401{,}92\ mm^2$

$M_2 = As'.fy.(d - d') = 401{,}92 \times 240 \times (144 - 40) = 10031923{,}2$ Nmm

$M_1 = Mn - M_2 = 2{,}625 \times 10^7 - 10031923{,}2 = 16218076{,}8$ Nmm

$M_1 > 0 \rightarrow$ maka As' diperhitungkan

$k = M_1/(b.d^2.R_1) = 16218076{,}8/(200 \times 144^2 \times 19{,}125) = 0{,}204$

$F = 1 - \sqrt{1-2k} = 1 - \sqrt{1-(2*0{,}204)} = 0{,}23$

$F_{max} = \beta_1.450/(600 + fy) = (0{,}85 \times 450)/(600 + 240) = 0{,}455$

$F < F_{max} \rightarrow$tulangan tunggal *under reinforced*

$As_1 = F.b.d.R_l/fy = (0{,}23 \times 200 \times 144 \times 19{,}125)/240 = 527{,}85\ mm^2$

$As = As_1 + As_2 = 527{,}85 + 401{,}92 = 929{,}77\ mm^2$

$\rho = As/(b.d) = 929{,}77 /(200 \times 144) = 0{,}032$

$\rho_{min} = 1{,}4/ fy = 1{,}4/240 = 5{,}83 \times 10^{-3}$

$\rho_{maks} = 0{,}03635 \rightarrow$(Tabel-8 *Dasar-Dasar Perencanaan Beton Bertulang, Seri Beton*-1, W.C. Vis, Gideon Kusuma)

atau $\rho_{maks} = \beta_1.450/(600 + fy).(R_l/fy) = 0{,}85x450/(600+240)x(19{,}125/240)$

$= 0{,}03629$

$\rho > \rho_{min} \rightarrow$ dipakai ρ

$As = \rho.b.d = 0{,}032 \times 200 \times 144 = 921{,}6\ mm^2$

Tulangan terpasang = 5 Ø 16 ($As = 1005\ mm^2$)

Cek terhadap rasio tulangan :

$\rho = As_{terpasang}/b.d = 1005/(200x144) = 0{,}035$

$\rho_{min} < \rho < \rho_{maks} \rightarrow$ ok

$Asb = As + As_{terpasang} = 921{,}6 + 1005 = 1926{,}6\ mm^2$

$\rho b = Asb/(b.d) = 1926{,}6/(200 \times 144) = 0{,}067$

$\rho b + \rho_{terpasang} \geq \rho$

$0{,}067 + 0{,}035 \geq 0{,}032$

$0{,}102 \geq 0{,}032 \rightarrow$ ok

Periksa :

$d'/d \leq d'/d_{max}$

$40/144 \leq 0{,}7.F_{max}$

$0{,}28 \leq 0{,}7 \times 0{,}455$

$0{,}28 \leq 0{,}319$

2. Tulangan Sengkang

Vu = F =7 ton = 7000 kg

Direncanakan menggunakan :

Tulangan sengkang = Ø 8 mm

fy = 140 Mpa = 140 N/mm^2 = 1400 kg/cm^2

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

h = 200 mm

b = 200 mm

d' = tebal selimut beton = 40 mm

d = h – d' – ½ Ø tulangan utama – Ø tulangan sengkang

= 200 – 40 – ½ x 16 – 8 = 144 mm

Vn = Vu/φ = 7000/0,6 = 11666,67 kg

Vc = 0,17.b.d.√f'c = 0,17 x 20 x 14,4 x√225 = 734,4 kg

(Vn – Vc) < 2/3.√f'c.b.d

(11666,67 – 734,4) < 2/3 x √225 x 20 x 14,4

10932,27 > 2880 → penampang tidak cukup ukurannya

Solusinya adalah penampang diperbesar. Akan tetapi, karena digunakan *casing* baja, maka penampang dianggap kuat, sehingga penampang tidak perlu diperbesar.

φ.(Vc/2) = 0,6 x (734,4/2) = 220,32 kg

Vu > φ.(Vc/2) → perlu tulangan geser di sepanjang tumpuan sampai terjadinya φ.(Vc/2)

φ.Vc = 0,6 x 734,4 = 440,64 kg

Vu > φ.Vc → perlu tulangan geser

Direncanakan menggunakan sengkang Ø 8 mm

Av = jumlah luas penampang dua kali sengkang

= 2 x (1/4 x 3,14 x 8^2) = 100,48 mm^2 = 1,0048 cm^2

S = jarak sengkang

= Av.d.fy/(Vn – Vc)

= 1,0048 x 14,4 x 1400/(11666,67 – 734,4) = 1,853 cm

Syarat : S < d/2

1,853 < 14,4/2

1,853 < 7,2 (ok)

Dipakai sengkang Ø 8 – 70 (tumpuan) dan Ø 8 – 100 (lapangan)

- Cek Terhadap Pengaruh Geser Lentur

$$\tau = \frac{8.V}{7.b.h} = \frac{8x7000}{7x20x20} = 20 \text{ kg/cm}^2$$

Syarat :

$\tau \leq \tau_{bm}$

20 ≤ 1,35 $\sqrt{\sigma'_{bk}}$ = 1,35 x √225

20 ≤ 20,25 kg/cm^2 (aman)

Untuk keperluan praktis di lapangan, tulangan sengkang ini berupa tulangan melingkar atau spiral.