

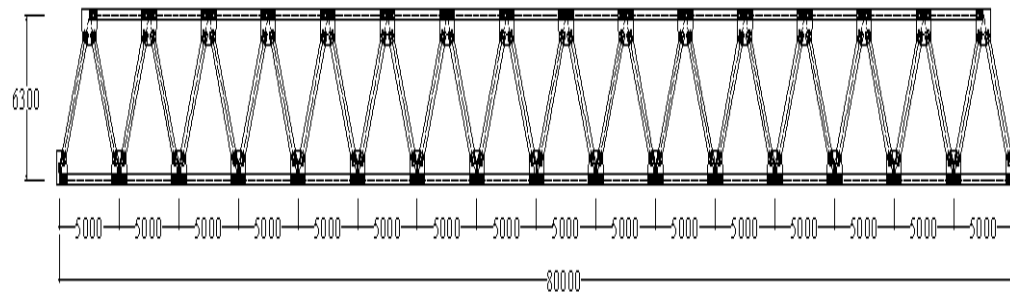
## BAB V

### PERHITUNGAN KONSTRUKSI

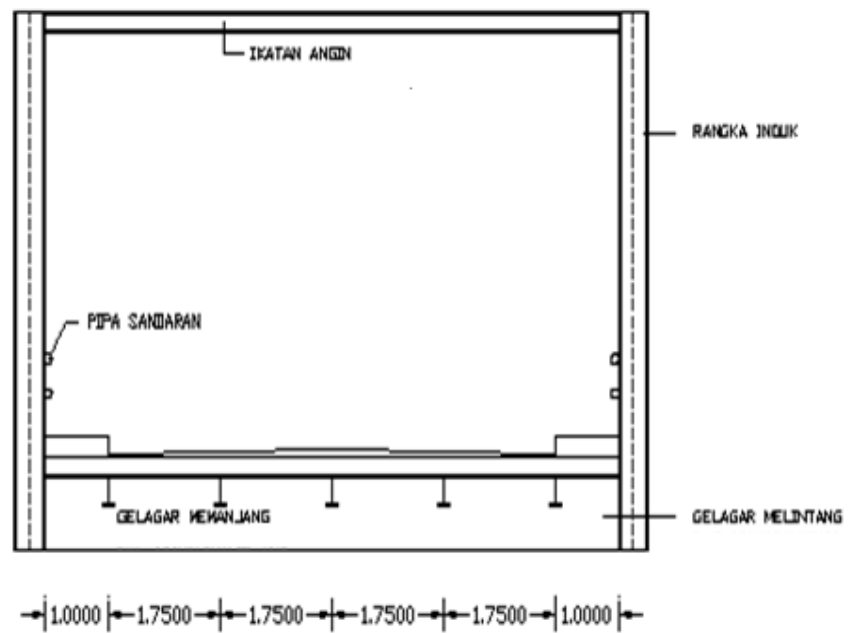
#### 5.1 DATA PERENCANAAN BANGUNAN

Direncanakan :

- Bentang Jembatan : 80 meter
  - Lebar Jembatan :  $9 ( 1 + 7 + 1 )$  meter
  - Jenis Jembatan : Struktur Rangka Baja
  - Bangunan Atas
    - a. Lantai Jembatan
      - Lebar Lantai Jembatan :  $2 \times 3,5$  meter
      - Mutu Beton : 25 Mpa
      - Tinggi Plat : 20 cm
    - b. Lantai Trotoar
      - Lebar Lantai Trotoar :  $2 \times 1$  meter
      - Mutu Beton : 25 Mpa
      - Tinggi Plat : 20 cm
  - Bangunan Bawah
    - a. Abutment
      - Mutu beton : 35 MPa
      - Mutu tulangan : 240 MPa
      - Jenis : Kontraport
    - b. Pelat injak
      - Mutu beton : 35 MPa
      - Mutu tulangan : 240 MPa
    - c. Bangunan pondasi
      - Mutu beton : 40 MPa
      - Mutu tulangan : 240 MPa
      - Jenis : Tiang pancang
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Gambar 5.1 Penampang Memanjang Jembatan



Gambar 5.2 Penampang Melintang Jembatan

## 5.2 PERHITUNGAN BANGUNAN ATAS

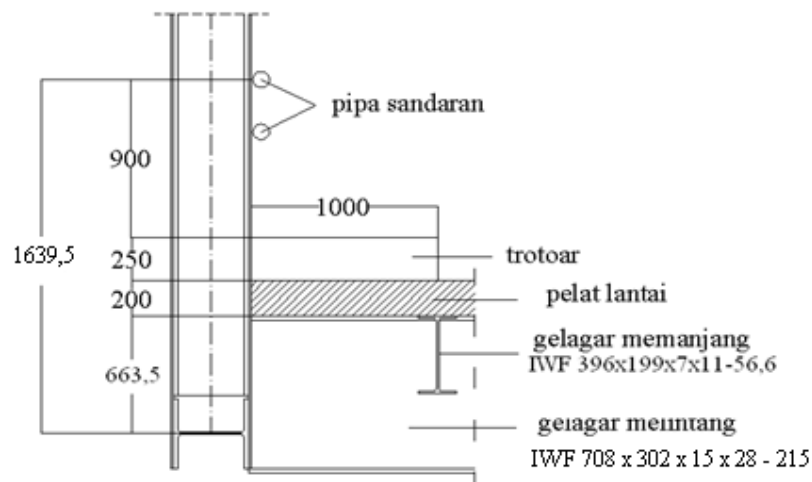
### 5.2.1 Perhitungan Sandaran

Railing atau sandaran merupakan pagar untuk pengamanan pengguna jembatan khususnya pejalan kaki. Menurut Pedoman Perencanaan Pembebanan Jembatan Jalan Raya halaman 10 :

*Tiang-tiang sandaran pada setiap tepi trotoar harus diperhitungkan untuk dapat menahan beban horizontal sebesar 100 kg/m' yang bekerja pada tinggi 90 cm diatas lantai trotoir.*

Jika gelagar melintang diasumsikan menggunakan IWF 708x302x15x28-215 dan rangka induk diasumsikan menggunakan IWF 428x407x20x35-283 maka tinggi sandaran dari sumbu bawah rangka induk dihitung sebagai berikut :

- h1 = tinggi sandaran dari trotoar = 900 mm
- h2 = tinggi trotoar = 250 mm
- h3 = tinggi plat lantai kendaraan = 200 mm
- h4 = tinggi gelagar melintang = 890 mm (IWF 708x302x15x28-215)
- h5 = tebal sayap gelagar melintang = 23 mm
- h6 = lebar profil rangka induk = 407 mm (IWF 428x407x20x35-283)



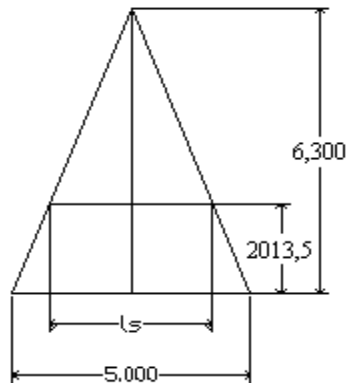
Gambar 5.3 Tinggi Tiang Sandaran

$$\begin{aligned} h_s &= h_1 + h_2 + h_3 + (h_4 - h_5 - (1/2 \times h_6)) \\ &= 708 + 250 + 200 + (708 - 23 - (1/2 \times 407)) \\ &= 1639,5 \text{ mm} \end{aligned}$$

sedangkan tinggi total rangka adalah 6.3 meter

Sandaran diasumsikan mempunyai sendi pada rangka utama dengan panjang sandaran yang menumpu pada rangka utama sebesar (pada tengah bentang) :

Dengan menggunakan rumus segitiga :



$$\frac{5000}{6300} = \frac{Ls}{(6300 - 1639,5)}$$

$$\begin{aligned} Ls &= \frac{(5000 \times 4660,5)}{6300} \\ &= 3698,809 \text{ mm} = 369,880 \text{ cm} \end{aligned}$$

Pembebanan pada pipa sandaran :

- Beban horizontal (H) = 100 kg/m
- Beban vertikal (V) = 7,13 kg/m (berat sendiri pipa sandaran)

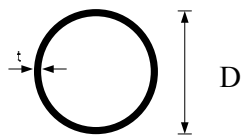
Sandaran direncanakan menggunakan pipa  $\phi$  76,3 mm (3 inchi).

**a. Data Perencanaan**

$$\sigma_{\text{ijin}} = 160 \text{ MPa}$$

$$E_{\text{baja}} = 2,1 \times 10^5 \text{ MPa}$$

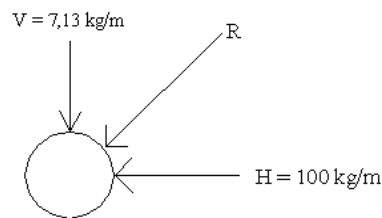
**b. Data Teknis Profil**



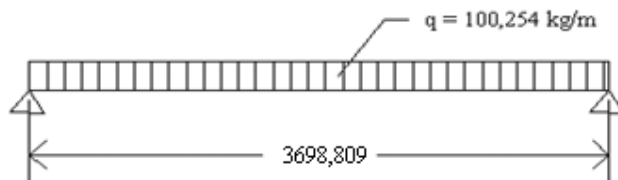


$$\begin{aligned} D &= 7,63 \text{ cm} \\ t &= 0,4 \text{ cm} \\ F &= 9,085 \text{ cm}^2 \\ G &= 7,13 \text{ kg/m} \end{aligned}$$

$$\begin{aligned} I &= 59,5 \text{ cm}^4 \\ i &= 2,60 \text{ cm} \\ W &= 15,6 \text{ cm} \end{aligned}$$



$$\begin{aligned} R &= \sqrt{V^2 + H^2} \\ &= \sqrt{7,13^2 + 100^2} \\ &= 100,254 \text{ kg/m} \end{aligned}$$



$$\begin{aligned} R_{AV} &= \frac{1}{2} \times q \times Ls \\ &= \frac{1}{2} \times 100,254 \times 3,698 = 185,369 \text{ kg} \end{aligned}$$

Momen yang terjadi pada pipa sandaran :

$$\begin{aligned} Mu &= \frac{1}{8} \times q \times Ls^2 \\ &= \frac{1}{8} \times 100,254 \times 3,698^2 = 171,374 \text{ kgm} \end{aligned}$$

Geser yang terjadi pada pipa sandaran :

$$\begin{aligned} D &= \frac{1}{2} \times q \times Ls \\ &= \frac{1}{2} \times 100,254 \times 3,698 = 185,369 \text{ kg} \end{aligned}$$

### c. Kontrol terhadap Bahan dan Tegangan yang Ada

#### 1) Terhadap lendutan

$$\frac{5 \times qh \times l^4}{384 EI} < \frac{l}{180}$$

$$\frac{5 \times 1,003 \times 369,8^4}{384 \times 2,1 \times 10^6 \times 59,5} = 1,95 \text{ cm} < \frac{l}{180} = \frac{369,8}{180} = 2,054 \text{ cm...OK}$$

2) Terhadap momen

$$\sigma_u < \sigma_{\text{ijin}}$$

$$\frac{Mu}{W} = \sigma_{\text{ijin}}$$

$$\frac{17137,4}{15,6} = 1098,55 \text{ kg/cm}^2 < 1600 \text{ kg/cm}^2 \text{....OK}$$

3) Terhadap geser

$$\tau = \frac{D \times S}{l} = \frac{185,369 \times 15,6}{59,5} = 48,600 \text{ kg/cm}^2$$

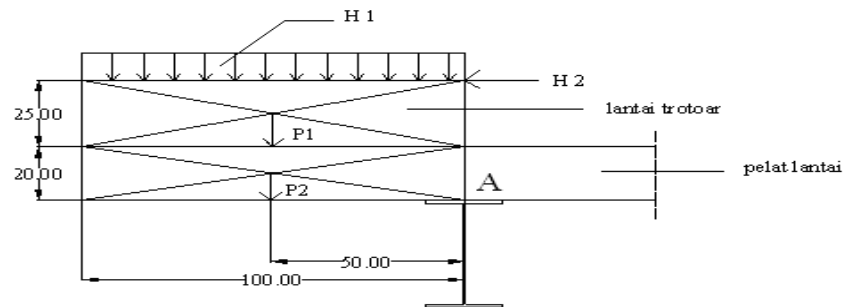
$$\tau_{\text{ijin}} = 0,58 \times \sigma_{\text{ijin}} = 0,58 \times 1600 = 928 \text{ kg/cm}^2$$

$$\tau < \tau_{\text{ijin}} \text{ .....OK}$$

Jadi pipa  $\phi$  76,3 ( 3 inchi ) dapat dipakai untuk sandaran.

### 5.2.2 Perhitungan Lantai Trotoar

Fungsi utama trotoar adalah memberikan layanan yang optimal bagi pejalan kaki baik dari segi keamanan maupun kenyamanan. Berdasar PPJJR 1987 : Kontruksi trotoar harus diperhitungkan terhadap beban hidup ( q ) = 500 kg/m<sup>2</sup>, Kerb yang terdapat pada tepi – tepi lantai kendaraan diperhitungkan untuk dapat menahan beban satu horisontal ke arah melintang jembatan sebesar ( P ) = 500 kg/m<sup>2</sup> yang bekerja pada puncak kerb yang bersangkutan atau pada tinggi 25 cm diatas permukaan lantai kendaraan apabila kerb lebih tinggi dari 25 cm.



Gambar 5.4 Pembebanan pada Trotoar

**a. Data Perencanaan**

- $f'_c = 25 \text{ MPa}$
- $\gamma_c = 2500 \text{ kg/m}^3$
- $f_y = 240 \text{ MPa}$
- $\phi = 16 \text{ mm}$
- $d = h - p - \frac{1}{2} \phi \text{ tulangan}$   
 $= 250 - 40 - 8 = 202 \text{ mm}$

**b. Pembebanan**

1) Akibat Beban Mati

- $P_1$  (berat trotoar)  $= 0,25 \times 1,00 \times 1,00 \times 2500 = 625 \text{ kg}$
- $P_2$  (berat pelat jembatan)  $= 0,20 \times 1,00 \times 1,00 \times 2500 = 500 \text{ kg}$

2) Akibat Beban Hidup

- $H_1$  (beban pejalan kaki)  $= 1,00 \times 500 = 500 \text{ kg}$
- $H_2$  (beban tumbukan (pada trotoar))  $= 1,00 \times 500 = 500 \text{ kg}$

3) Akibat Momen yang terjadi di titik A

- $MP_1 = 625 \times 0,5 = 312,5 \text{ kgm}$
- $MP_2 = 500 \times 0,5 = 250 \text{ kgm}$
- $MH_1 = 500 \times 0,5 = 250 \text{ kgm}$
- $MH_2 = 500 \times 0,45 = 225 \text{ kgm} +$   
 $M \text{ total (Mu)} = 1037,5 \text{ kgm}$

**c. Perhitungan Tulangan**

$$\frac{Mu}{bd^2} \times 10^{-2} = \rho \times 0,8 \times f_y \left( 1 - 0,588 \times \rho \times \frac{f_y}{f'_c} \right)$$

$$\frac{1037,5}{1 \times 0,202^2} \times 10^{-2} = \rho \times 0,8 \times 2400 \left( 1 - 0,588 \times \rho \times \frac{2400}{250} \right)$$

$$9031 \rho^2 - 1920 \rho + 2,543 = 0$$

$$\rho = 0,0013$$

$$\rho_{\min} = \frac{1,4}{f_y} = \frac{1,4}{240} = 0,0058$$

$$\rho_{\max} = 0,75 \times \beta_1 \left( \frac{0,85 f'_c}{f_y} \times \frac{600}{600 + f_y} \right) \text{ dan } \beta_1 = 0,85$$

$$\rho_{\max} = 0,75 \times 0,85 \left( \frac{0,85 \times 250}{2400} \times \frac{600}{600 + 2400} \right) \text{ dan } \beta_1 = 0,85$$

$$\rho_{\max} = 0,013$$

Karena  $\rho_{\min} > \rho \rightarrow$  dipakai  $\rho_{\min} = 0,0058$

$$A = \rho \times b \times d = 0,0058 \times 1000 \times 202 = 1171,6 \text{ mm}^2$$

Dipakai tulangan  $\phi$  16 - 150 ( $A_s = 1340 \text{ mm}^2$ )

Checking :

$$\rho = \frac{A_s \text{ terpasang}}{(b \times d)}$$

$$\frac{1340}{(1000 \times 202)} = 0,0087 < \rho_{\max} \dots \text{OK}$$

Menurut SKSNI T15-1991-03 pasal 3.16.12, dalam arah tegak lurus terhadap tulangan utama harus disediakan tulangan pembagi (untuk tegangan susut dan suhu)

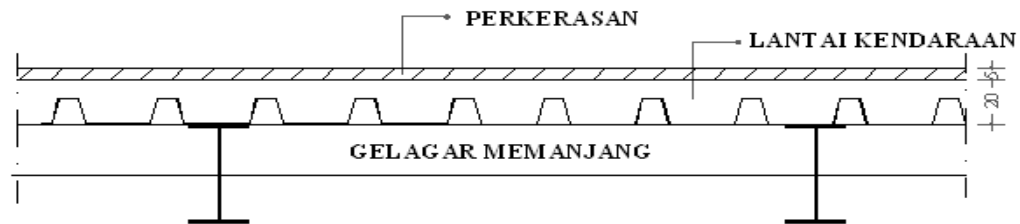
$\rightarrow$  untuk  $f_y = 240 \text{ MPa}$

$$A_s = 0,0025 \times b \times d$$

$$A_s = 0,0025 \times 1000 \times 202 = 505 \text{ mm}^2$$

Digunakan tulangan bagi D12-200 ( $A = 565 \text{ mm}^2$ )

### 5.2.3 Perencanaan Pelat Lantai Kendaraan



Gambar 5.5 Pelat Lantai Kendaraan

#### a. Data Perencanaan

- Mutu Beton ( $f^c$ ) = 25 MPa
- Mutu Tulangan ( $f_y$ ) = 240 MPa
- Tebal Pelat Lantai = 20 cm
- Tebal Perkerasan = 5 cm
- $\phi$  tulangan rencana = 14 mm
- Tebal Selimut Beton ( $p$ ) = 40 mm ( untuk konstruksi lantai yang langsung berhubungan dengan cuaca )
- Berat jenis beton ( $\gamma^c$ ) =  $25 \text{ kN/m}^3 = 2500 \text{ kg/m}^3$
- Berat jenis aspal ( $\gamma^a$ ) =  $22 \text{ kN/m}^2 = 2200 \text{ kg/m}^3$

#### b. Perhitungan Momen Lentur Pada Pelat Lantai Kendaraan

##### 1) Akibat Beban Mati :

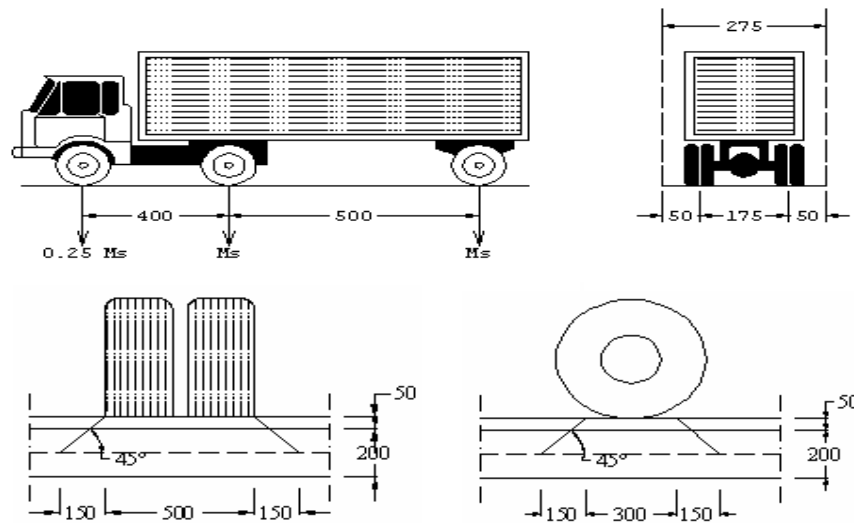
- Berat sendiri pelat =  $0,20 \times 1,00 \times 2500 = 500 \text{ kg/m}$
- Berat aspal =  $0,05 \times 1,00 \times 2200 = 110 \text{ kg/m}$
- Berat air hujan =  $0,05 \times 1,00 \times 1000 = 50 \text{ kg/m} +$

$$\sum qD_L = 660 \text{ kg/m}$$

$$\begin{aligned} \text{Momen Tumpuan} = \text{Momen Lapangan} &= \frac{1}{10} \times q \times L^2 \\ &= \frac{1}{10} \times 660 \times 1,75^2 \\ &= 202,125 \text{ kgm} \end{aligned}$$

## 2) Akibat Beban Hidup ( T ) :

Untuk perhitungan kekuatan lantai kendaraan atau sistem lantai kendaraan jembatan harus digunakan beban " T " yaitu beban yang merupakan kendaraan truck yang mempunyai beban roda ganda ( *Dual Wheel Load* ) sebesar 10 ton.

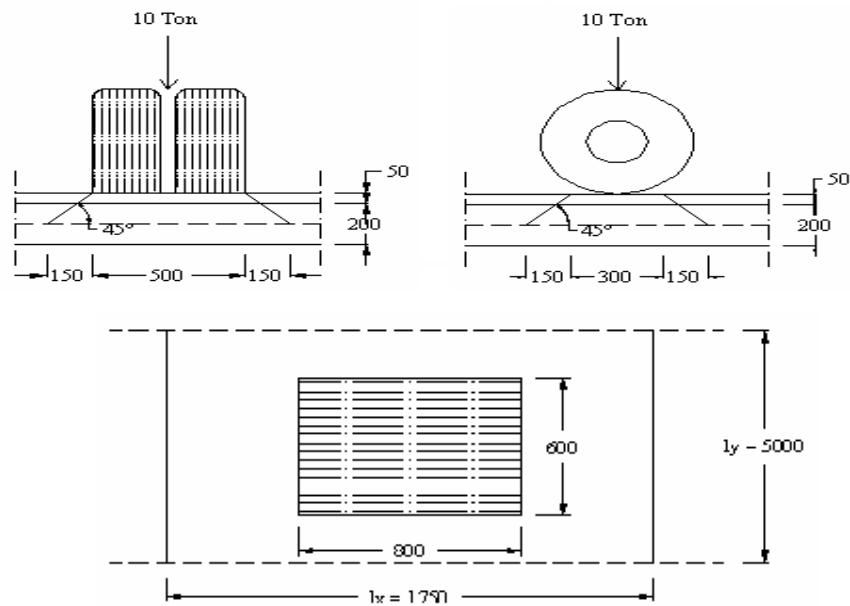


Gambar 5.6 Beban " T "

- o Beban " T " = 10 ton
- o Bidang kontak pada sumbu plat
  - $t_x = ( 50 + ( 2 \times 15 ) ) = 80 \text{ cm} = 0,8 \text{ m}$
  - $t_y = ( 30 + ( 2 \times 15 ) ) = 60 \text{ cm} = 0,6 \text{ m}$
- o Penyebaran Beban " T "

$$T' = \frac{10000}{0,8 \times 0,6} = 20833,333 \text{ kg/m}^2$$

- Kondisi 1 ( satu roda ditengah pelat )



Gambar 5.7 Penyebaran Beban " T " pada Kondisi 1

$$\begin{aligned} \circ \quad t_x &= 0,80 \text{ m} & \frac{t_x}{L_x} &= \frac{0,8}{1,75} = 0,457 \\ \circ \quad t_y &= 0,60 \text{ m} \\ \circ \quad L_x &= 1,75 \text{ m} & \frac{t_y}{L_y} &= \frac{0,6}{1,75} = 0,343 \\ \circ \quad L_y &= 5,00 \text{ m} \end{aligned}$$

Dari tabel Bittner :

$$\begin{aligned} F_{xm} &= 0,1529 \\ F_{ym} &= 0,0865 \end{aligned}$$

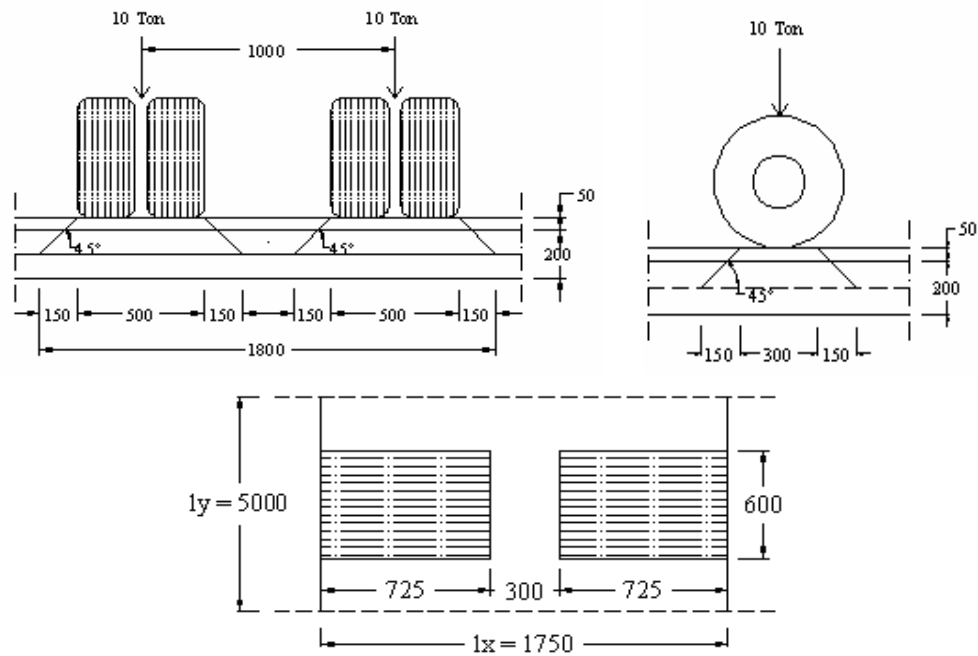
Momen maksimum pada kondisi 1 ( satu roda ditengah pelat ) :

$$\begin{aligned} M_{xm} &= f_{xm} \times T' \times t_x \times t_y \\ &= 0,1529 \times 20833,333 \times 0,8 \times 0,6 \\ &= 1529,000 \text{ kgm} \\ M_{ym} &= F_{ym} \times T' \times t_x \times t_y \end{aligned}$$

$$= 0,0865 \times 20833,333 \times 0,8 \times 0,6$$

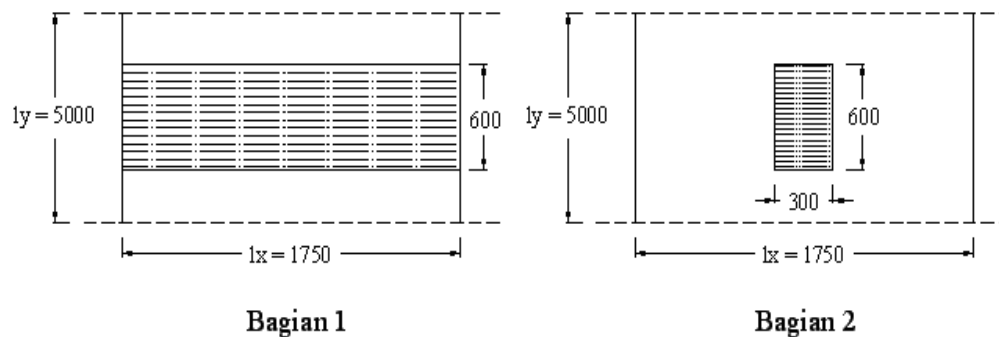
$$= 865,000 \text{ kg}$$

- Kondisi 2 ( dua roda berdekatan )



Gambar 5.8 Penyebaran Beban " T " pada Kondisi 2

Luas bidang kontak diatas dapat dihitung menjadi 2 bagian, yaitu :



#### ▪ Bagian 1

○  $t_x = 1,75 \text{ m}$

○  $t_y = 0,6 \text{ m}$

$$\frac{t_x}{l_x} = \frac{1,75}{1,75} = 1,0$$



$$\circ \quad l_x = 1,75 \text{ m} \quad \frac{t_y}{l_x} = \frac{0,6}{1,75} = 0,343$$

$$\circ \quad l_y = 5 \text{ m}$$

Dari tabel Bittner diperoleh :

$$f_{xm} = 0,0904$$

$$f_{ym} = 0,0572$$

Momen yang terjadi :

$$\begin{aligned} M_{xm1} &= f_{xm} \times T' \times t_x \times t_y \\ &= 0,0904 \times 20833,333 \times 1,75 \times 0,6 \\ &= 1977,500 \text{ kgm} \end{aligned}$$

$$\begin{aligned} M_{ym1} &= f_{ym} \times T' \times t_x \times t_y \\ &= 0,0572 \times 20833,333 \times 1,75 \times 0,6 \\ &= 1251,250 \text{ kgm} \end{aligned}$$

#### ▪ Bagian 2

$$\circ \quad t_x = 0,3 \text{ m} \quad \Rightarrow \quad \frac{t_x}{l_x} = \frac{0,3}{1,75} = 0,171$$

$$\circ \quad t_y = 0,6 \text{ m}$$

$$\circ \quad l_x = 1,75 \text{ m} \quad \frac{t_y}{l_x} = \frac{0,6}{1,75} = 0,343$$

$$\circ \quad l_y = 5 \text{ m}$$

Dari tabel Bittner diperoleh :

$$f_{xm} = 0,2106$$

$$f_{ym} = 0,1043$$

Momen yang terjadi :

$$\begin{aligned} M_{xm2} &= f_{xm} \times T' \times t_x \times t_y \\ &= 0,2106 \times 20833,333 \times 0,3 \times 0,6 \\ &= 789,750 \text{ kgm} \end{aligned}$$

$$M_{ym2} = f_{ym} \times T' \times t_x \times t_y$$

$$\begin{aligned}
 &= 0,1043 \times 20833,333 \times 0,3 \times 0,6 \\
 &= 391,125 \text{ kgm}
 \end{aligned}$$

Momen maksimum pada kondisi 2 :

$$\begin{aligned}
 M_{xm} &= M_{xm1} - M_{xm2} \\
 &= 1977,5 - 789,75 \\
 &= 1187,750 \text{ kgm} \\
 M_{ym} &= M_{ym1} - M_{ym2} \\
 &= 1251,25 - 391,125 \\
 &= 860,125 \text{ kgm}
 \end{aligned}$$

Momen maksimum akibat beban hidup “T” diambil dari momen terbesar pada kondisi 1 dan kondisi 2, yaitu :

- Momen maksimum pada kondisi 1 (satu roda ditengah pelat) :

$$\begin{aligned}
 M_{xm} &= 1529,000 \text{ kgm} \\
 M_{ym} &= 865,000 \text{ kgm}
 \end{aligned}$$

- Momen maksimum pada kondisi 2 (dua roda berdekatan) :

$$\begin{aligned}
 M_{xm} &= 1187,750 \text{ kgm} \\
 M_{ym} &= 860,125 \text{ kgm}
 \end{aligned}$$

Dipilih momen pada kondisi 1 (satu roda ditengah pelat), karena menghasilkan nilai momen yang terbesar.

Momen total yang terjadi pada pelat tengah akibat beban mati dan beban hidup adalah :

$$\begin{aligned}
 M_X &= M_{xDL} + M_{xLL} \\
 &= 202,125 + 1529,000 \\
 &= 1731,125 \text{ kgm} \\
 M_Y &= M_{yDL} + M_{yLL} \\
 &= 202,125 + 865,000 \\
 &= 1067,125 \text{ kgm}
 \end{aligned}$$

### c. Perhitungan Tulangan Pelat Lantai Kendaraan

- Tulangan pada arah melintang jembatan (lx)

$$M_x = \frac{M_x}{\phi}, \phi = 0,8 \text{ ( factor reduksi untuk menahan momen lentur )}$$

$$M_x = \frac{1731,125}{0,8} = 2163,906 \text{ kgm} = 21,639 \text{ Nm}$$

$$b = 1,00 \text{ m}$$

$$d = h - p - \left( \frac{1}{2} \phi \right)$$

$$= 200 - 40 - 8 = 152 \text{ mm} = 0,152 \text{ m}$$

$$\frac{M_x}{bd^2} = \frac{21,639}{1,00 \times 0,152^2} = 936,753 \text{ kN/m}^2 = 0,936753 \text{ Mpa}$$

$$\frac{M_x}{bd^2} = \rho \times 0,8 \times f_y \times \left( 1 - 0,588 \times \rho \times \frac{f_y}{f'_c} \right)$$

$$0,947 = \rho \times 0,8 \times 240 \times \left( 1 - 0,588 \times \rho \times \frac{240}{25} \right)$$

$$0,947 = 192 \rho (1 - 5,645 \rho)$$

$$1083,84 \rho^2 - 192 \rho + 0,947 = 0$$

$$\rho_1 = 0,0051$$

$$\rho_2 = 0,172$$

$$\rho_{balance} = \left( \frac{0,85 \times f'_c \times \beta_1}{f_y} \right) \times \left( \frac{600}{600 + f_y} \right)$$

$$= \left( \frac{0,85 \times 25 \times 0,85}{240} \right) \times \left( \frac{600}{600 + 240} \right)$$

$$= 0,0645$$

$$\rho_{max} = 0,75 \times \rho_{balance}$$

$$= 0,75 \times 0,0645 = 0,0483$$

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

Syarat,  $\rho_{\min} < \rho < \rho_{\max}$ , karena  $\rho < \rho_{\min}$  maka digunakan  $\rho = \rho_{\min} = 0,00583$

$$\begin{aligned} A_s &= \rho \times b \times d \times 10^6 \\ &= 0,00583 \times 1,00 \times 0,152 \times 10^6 \\ &= 886,16 \text{ mm}^2 \end{aligned}$$

Digunakan tulangan Ø 14 – 150 ( $A_s = 1026 \text{ mm}^2$ )

- Tulangan pada arah memanjang jembatan (ly)

$$M_y = \frac{My}{\phi}, \phi = 0,8 \text{ ( factor reduksi untuk menahan momen lentur )}$$

$$M_y = \frac{1067,125}{0,8} = 1333,906 \text{ kgm} = 13,33906 \text{ kNm}$$

$$b = 1,00 \text{ m}$$

$$\begin{aligned} d &= h - p - \phi_{tul} \times \left( \frac{1}{2} \phi \right) \\ &= 200 - 40 - 16 - 8 = 136 \text{ mm} = 0,136 \text{ m} \end{aligned}$$

$$\frac{My}{bd^2} = \frac{13,33906}{1,00 \times 0,136^2} = 724,948 \text{ kN/m}^2 = 0,742948 \text{ Mpa}$$

$$\frac{My}{bd^2} = \rho \times 0,8 \times f_y \times \left( 1 - 0,588 \times \rho \times \frac{f_y}{f'_c} \right)$$

$$0,743 = \rho \times 0,8 \times 240 \times \left( 1 - 0,588 \times \rho \times \frac{240}{25} \right)$$

$$0,743 = 192\rho(1 - 5,645\rho)$$

$$1083,84\rho^2 - 192\rho + 0,743 = 0$$

$$\rho_1 = 0,00325, \quad \rho_2 = 0,174$$

$$\begin{aligned} \rho_{balance} &= \left( \frac{0,85 \times f'_c \times \beta_1}{f_y} \right) \times \left( \frac{600}{600 + f_y} \right) \\ &= \left( \frac{0,85 \times 25 \times 0,85}{240} \right) \times \left( \frac{600}{600 + 240} \right) = 0,0645 \end{aligned}$$

$$\begin{aligned}\rho_{\max} &= 0,75 \times \rho_{\text{balance}} \\ &= 0,75 \times 0,0645 = 0,0483\end{aligned}$$

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

Syarat,  $\rho_{\min} < \rho < \rho_{\max}$ , karena  $\rho < \rho_{\min}$  maka digunakan

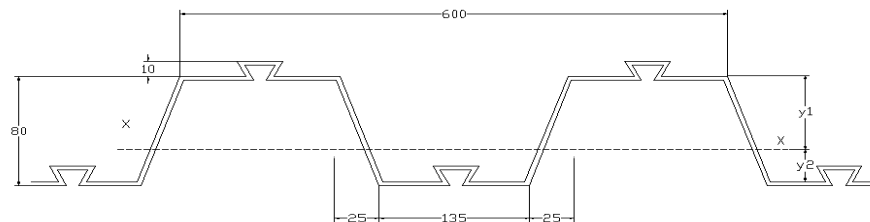
$$\rho = \rho_{\min} = 0,00583$$

$$\begin{aligned}A_s &= \rho \times b \times d \times 10^6 \\ &= 0,00583 \times 1,00 \times 0,136 \times 10^6 \\ &= 792,88 \text{ mm}^2\end{aligned}$$

Digunakan tulangan Ø 12 – 125 ( $A_s = 905 \text{ mm}^2$ )

#### d. Cek Deck Slab

Direncanakan menggunakan dek baja type Ribdeck 80 dengan dimensi sebagai berikut :



$$t = 1,2 \text{ mm}$$

$$W = 14,8 \text{ kg/m}^2$$

$$A = 1,848 \text{ mm}^2$$

$$I = 237,6 \text{ cm}^4$$

$$Y_{NA} = 42,5 \text{ mm} = 4,25 \text{ cm}$$

Mencari momen lawan ( $W_x$ )

$$\begin{aligned}y_2 &= \frac{80^2 \times 185 - 1,2(80 - 1,2)^2}{2 \times (80^2 \times 185 - 1,2(80 - 1,2))} \times 3 \\ &= \frac{80^2 \times 185 - 1,2(80 - 1,2)^2}{2 \times (80^2 \times 185 - 1,2(80 - 1,2))} \times 3 = 1,488 \text{ mm} = 0,148 \text{ cm}\end{aligned}$$

$$y_1 = 80 - 1,488 = 78,512 \text{ mm} = 7,851 \text{ cm}$$

$$W_1 = \frac{237,6}{0,148} = 1605,405 \text{ cm}^3$$

$$W_2 = \frac{237,6}{7,851} = 30,263 \text{ cm}^3$$

Untuk  $W_x$  dipakai  $W_2 = 30,263 \text{ cm}^3$

Cek tegangan yang terjadi :

$$\begin{aligned} \sigma_{\text{terjadi}} &= \frac{M}{W_x} < \bar{\sigma} \\ &= \frac{2163,906}{30,263} < 1867 \text{ kg/cm} \\ &= 71,503 \text{ kg/cm}^2 < 1867 \text{ kg/cm}^2 \dots \dots \text{OK} \end{aligned}$$

#### 5.2.4 Perencanaan Gelagar Memanjang

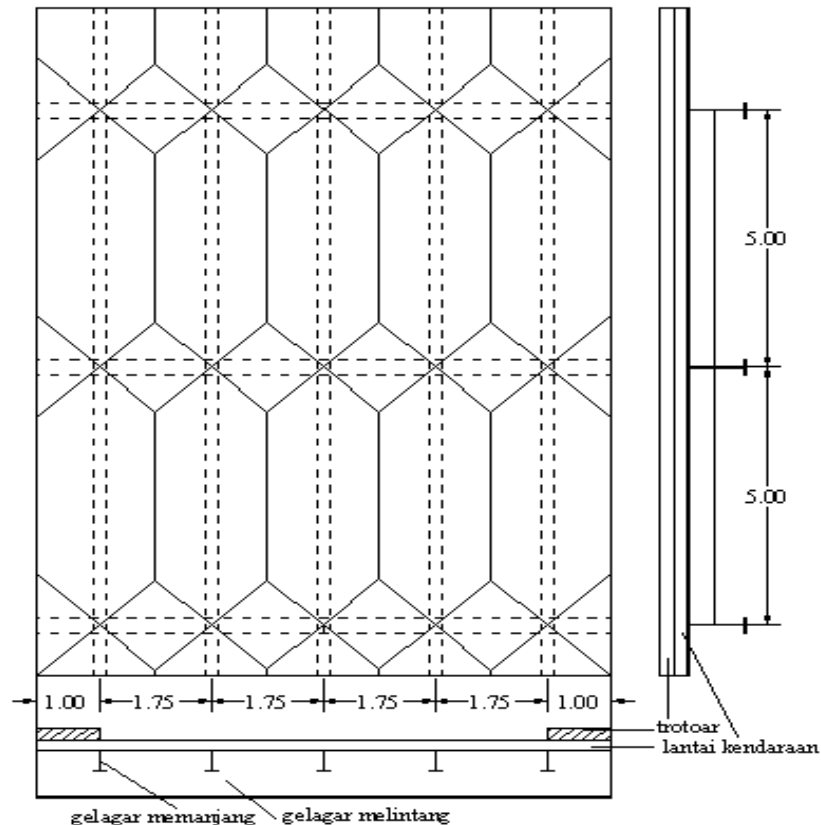
Gelagar jembatan berfungsi untuk menerima beban-beban yang bekerja diatasnya dan menyalurkannya ke bangunan dibawahnya. Pembebanan pada gelagar memanjang meliputi :

- Beban mati

Beban mati terdiri dari berat sendiri gelagar dan beban-beban yang bekerja diatasnya (pelat lantai jembatan, perkerasan, dan air hujan)

- Beban hidup

Beban hidup pada gelagar jembatan dinyatakan dengan beban “D” atau beban jalur, yang terdiri dari beban terbagi rata “q” ton per meter panjang per jalur, dan beban garis “P” ton per jalur lalu lintas tersebut.

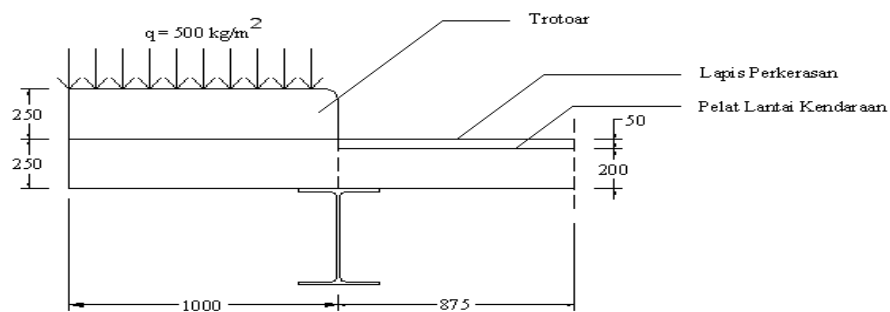


Gambar 5.9 Pemodelan Beban Gelagar Memanjang

**Data teknis perencanaan gelagar memanjang :**

- Mutu beton ( $f'c$ ) = 25 Mpa
- Mutu baja ( $f_y$ ) = 240 Mpa
- Berat isi beton bertulang = 2500 kg/m<sup>3</sup>
- Berat isi beton biasa = 2200 kg/m<sup>3</sup>
- Berat isi aspal = 2200 kg/m<sup>3</sup>
- Tebal pelat lantai kendaraan = 20 cm
- Tebal lapis perkerasan = 5 cm
- Tinggi trotoar = 25 cm
- Jarak antar gelagar melintang = 500 cm

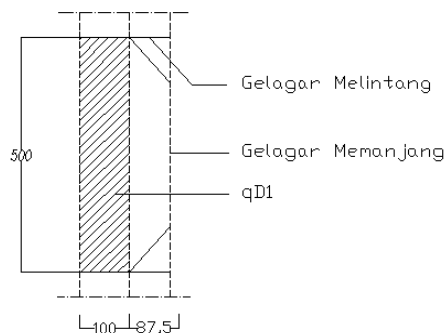
#### 5.2.4.1 Gelagar tepi



Gambar 5.10 Pembebanan Pada Gelagar Tepi

### 1. Perhitungan momen lentur pada gelagar tepi

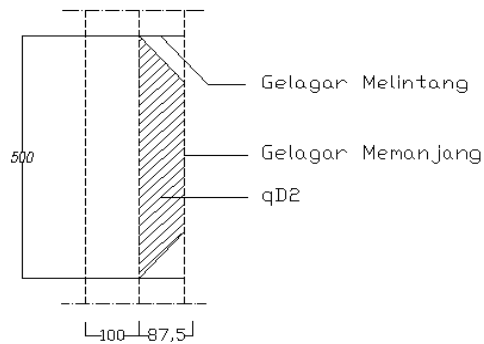
#### a. Beban mati



- Beban mati ( $q_{D1}$ ) akibat pelat lantai trotoar dan beban di atasnya :
  - Berat Trotoar =  $0,25 \times 1,00 \times 2500 = 625 \text{ kg/m}$

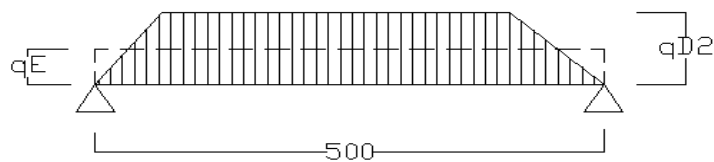


- Berat Pelat lantai =  $0,20 \times 1,00 \times 2500$  = 500 kg/m
  - Berat air hujan =  $0,05 \times 1,00 \times 1000$  = 50 kg/m
  - Berat Dek Baja =  $1,00 \times 11,35$  = 11,35 kg/m +
- $qD1 = 1311,35 \text{ kg/m}$



- Beban mati akibat (  $qD2$  ) pelat lantai trotoar dan beban diatasnya :

- Berat Perkerasan =  $0,05 \times 0,875 \times 2200$  = 96,25 kg/m
  - Berat Pelat lantai =  $0,20 \times 0,875 \times 2500$  = 437,5 kg/m
  - Berat air hujan =  $0,05 \times 0,875 \times 1000$  = 50 kg/m
  - Berat Dek Baja =  $0,875 \times 11,35$  = 9,931 kg/m +
- $qD2 = 593,681 \text{ kg/m}$



Beban Trapezium diubah menjadi beban Ekuivalen :

$$\frac{qD2}{24} \times (3L^2 - 4a^2) = \frac{qE}{8} \times L^2$$

$$\frac{593,681}{24} \times (3 \times 5^2 - 4 \times 0,875^2) = \frac{qE}{8} \times 5^2$$

$$q_E = 569,439 \text{ kg/m}$$

- Berat Sendiri Profil Gelagar Memanjang (  $q_{D3}$  ) = 49,6 kg/m  
( Diasumsikan menggunakan profil IWF 350 x 175 x 7 x 11 – 49,6 )

$$\begin{aligned} \text{Jadi beban Mati Total ( } q_{DL} \text{ )} &= q_{D1} + q_E + q_{D3} \\ &= 1311,35 + 569,439 + 49,6 \\ &= 1930,389 \text{ kg/m} \end{aligned}$$

**Gaya geser maksimum akibat beban mati (  $D_{mak DL}$  ) :**

$$\begin{aligned} D_{mak DL} &= \frac{1}{2} \times q \times L \\ &= \frac{1}{2} \times 1930,389 \times 5 \\ &= 4825,973 \text{ kg} \end{aligned}$$

**Momen maksimum akibat beban mati (  $M_{mak DL}$  ) :**

$$\begin{aligned} M_{max DL} &= \frac{1}{8} \times q_{DL} \times L^2 \\ &= \frac{1}{8} \times 1930,389 \times 5^2 \\ &= 6032,465 \text{ kgm} \end{aligned}$$

#### **b. Beban Hidup**

- **Beban terbagi rata (“q”)**

Bentang jembatan = 80 m, maka :

$$\begin{aligned} q &= 1.1 (1 + 30/L) \text{ t/m'} \quad \text{untuk } L > 60 \text{ m} \\ &= 1.1 (1 + 30/80) \text{ t/m'} = 1,65 \text{ t/m} \end{aligned}$$

Untuk perhitungan momen dan gaya lintang :

$$\text{Beban terbagi rata (q')} = \frac{q}{2,75} \times \alpha \times s' \quad , \quad \text{dimana :}$$

$\alpha$  = faktor distribusi,  $\alpha = 0,75$  bila kekuatan gelagar melintang diperhitungkan,  $\alpha = 1,00$  bila kekuatan gelagar melintang tidak diperhitungkan

$s'$  = lebar pengaruh beban hidup pada gelagar tepi

$$s' = \frac{1,750 \times 1}{2} = 0,875$$

$$\begin{aligned} q' &= \frac{q}{2,75} \times \alpha \times s' \\ &= \frac{1,65}{2,75} \times 0,75 \times 0,875 = 0,525 \text{ t/m} = 525 \text{ kg/m} \end{aligned}$$

Ketentuan penggunaan beban “D” dalam arah melintang jembatan :

- o Untuk jembatan dengan lebar lantai kendaraan lebih besar dari 5,50 meter, beban “D” sepenuhnya (100%) dibebankan pada lebar jalur 5,50 meter sedang lebar selebihnya dibebani hanya separuh beban “D” (50%).

$$q' = 50 \% \times 525 \text{ kg/m} = 262,5 \text{ kg/m}$$

- o Untuk perhitungan kekuatan gelagar karena pengaruh beban hidup pada trotoar, diperhitungkan beban sebesar 60% beban hidup trotoar.

$$\text{Beban hidup pada trotoar} = 500 \text{ kg/m}^2$$

Pengaruh beban hidup pada trotar (q)

$$q = 60\% \times (1,00 \times 500) = 300 \text{ kg/m}$$

Beban Hidup terbagi rata pada gelagar tepi :

$$q' = 262,25 + 300 = 562,25 \text{ kg/m}$$

#### • Beban garis “P”

$P = 12 \text{ ton}$ , Untuk perhitungan momen dan gaya lintang :

$$\text{Beban garis (P')} = \frac{P}{2,75} \times \alpha \times s' \times K \quad \text{dimana :}$$

$K$  = koefisien kejut, yang ditentukan dengan rumus :

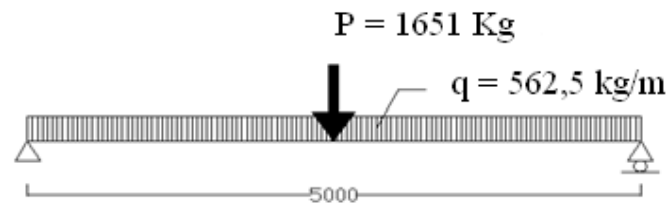
$$K = 1 + \left( \frac{20}{(50 + L)} \right) = 1 + \left( \frac{20}{(50 + 80)} \right) = 1,153$$

$$P' = \frac{P}{2,75} \times \alpha \times s' \times K = \frac{12}{2,75} \times 0,75 \times 0,875 \times 1,153$$

$$= 3,302 \text{ T} = 3302 \text{ kg}$$

Untuk jembatan dengan lebar lantai kendaraan lebih besar dari 5,50 meter, beban “D” sepenuhnya (100%) dibebankan pada lebar jalur 5,50 meter sedang lebar selebihnya dibebani hanya separuh beban “D” (50%).

$$P' = 50 \% \times 3302 = 1651 \text{ kg}$$



**Gaya geser maksimum akibat beban hidup ( Dmak LL ) :**

$$D_{makLL} = \frac{1}{2} p' + \frac{1}{2} q' L$$

$$= \left( \frac{1}{2} \times 1651 \right) + \left( \frac{1}{2} \times 562,25 \times 5 \right)$$

$$= 2231,125 \text{ kg}$$

**Momen maksimum akibat beban hidup (Mmak LL) :**

$$M_{maxLL} = \left( \frac{1}{8} \times q' \times l^2 \right) + \left( \frac{1}{4} \times P \times l \right)$$

$$= \left( \frac{1}{8} \times 562,25 \times 5^2 \right) + \left( \frac{1}{4} \times 1651 \times 5 \right)$$

$$= 3820,781 \text{ kgm}$$

**Gaya geser total pada gelagar tepi :**

$$D_{tot} = D_{makDL} + D_{makLL}$$

$$= 4825,973 \text{ kg} + 2231,125 \text{ kg}$$

$$= 7057,098 \text{ kg}$$

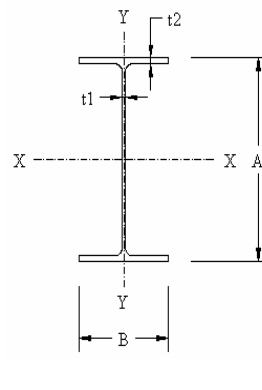
**Momen total pada gelagar tepi :**

$$\begin{aligned}
 M_{tot} &= M_{max\ DL} + M_{max\ LL} \\
 &= 6032,465\text{ kgm} + 3820,781\text{ kgm} \\
 &= 9853,246\text{ kgm}
 \end{aligned}$$

## 2. Pendimensian profil gelagar tepi

$$\begin{aligned}
 M_{tot} &= 9853,246\text{ kgm} = 985324,6\text{ kgcm} \\
 \bar{\sigma}_{Bj\ 44} &= 1867\text{ kg/cm}^2 \\
 W_x &= \frac{M_{tot}}{\bar{\sigma}} = \frac{985324,6}{1867} = 527,758\text{ cm}^3
 \end{aligned}$$

Digunakan profil baja IWF 350 × 175 x 7 x 11 – 49,6

	Profil	Berat	Ukuran (mm)				
	WF	(kg/m)	A	B	t1	t2	r
	350 x 175	49,6	350	175	7	11	14

Luas tampang	Momen Inersia		Jari-jari Inersia		Momen Lawan	
	Ix	Iy	ix	iy	Wx	Wy
63,14	13600	984	14,7	3,95	775	112

## 3. Kontrol terhadap bahan dan tegangan

- Kontrol terhadap lendutan ( $\delta$ )

$$\begin{aligned}
 \delta_{max} &= \frac{5 \times q_{tot} \times L^4}{384 E I_x} + \frac{P \times L^3}{48 E I_x} < \delta_{ijin} \\
 &= \frac{5 \times (19,303 + 5,623) \times 500^4}{384 \times (2,1 \times 10^6) \times 13600} + \frac{1651 \times 500^3}{48 \times (2,1 \times 10^6) \times 13600} < \frac{L}{500} \\
 &= 0,712 + 0,150 < 1,00\text{ cm} \\
 &= 0,862\text{ cm} < 1,00\text{ cm} \dots \dots \dots \text{OK}
 \end{aligned}$$

- Kontrol terhadap tegangan lentur yang terjadi ( $\sigma$ ) :

$$\sigma_{terjadi} = \frac{M_{tot}}{W_x} < \bar{\sigma}$$

$$\begin{aligned}
 &= \frac{985324,6}{775} < 1867 \text{ kg/cm} \\
 &= 1271,386 \text{ kg/cm}^2 < 1867 \text{ kg/cm}^2 \dots\dots\dots \mathbf{OK}
 \end{aligned}$$

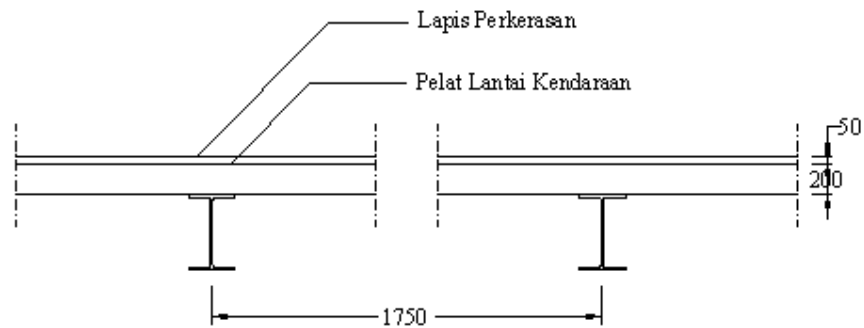
- Kontrol terhadap tegangan geser yang terjadi ( $\tau$ )

$$\begin{aligned}
 D_{\max} &= \left( \frac{1}{2} \times q_{\text{tot}} \times L \right) + \left( \frac{1}{2} \times P \right) \\
 &= \left( \frac{1}{2} \times (19,303 + 5,623) \times 500 \right) + \left( \frac{1}{2} \times 1651 \right) \\
 &= 7057 \text{ kg}
 \end{aligned}$$

$$\begin{aligned}
 A_{\text{web}} &= A_{\text{profil}} - A_{\text{flens}} \\
 &= 63,14 - (2 \times (17,5 \times 1,1)) \\
 &= 24,64 \text{ cm}^2
 \end{aligned}$$

$$\begin{aligned}
 \tau_{\text{terjadi}} &= \frac{D_{\max}}{A_{\text{web}}} < \bar{\tau} \\
 &= \frac{7057}{24,64} < 0,58 \times \bar{\sigma} \\
 &= 286,404 \text{ kg/cm}^2 < 1082,86 \text{ kg/cm}^2 \dots\dots\dots \mathbf{OK}
 \end{aligned}$$

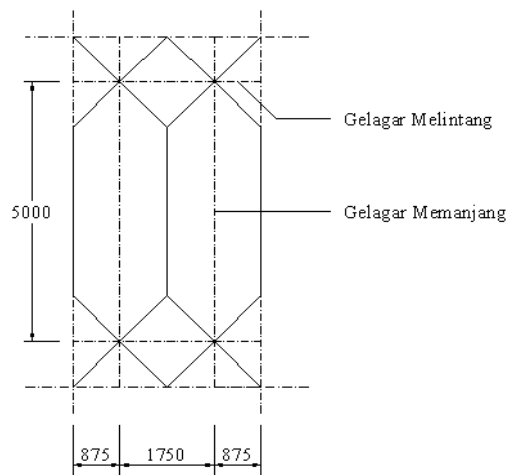
### 5.2.4.2 Gelagar tengah



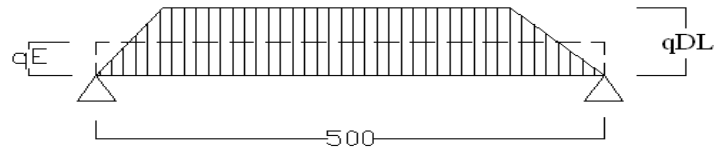
Gambar 5.11 Penampang Melintang Gelagar Tengah

#### 1) Perhitungan momen lentur pada gelagar tengah

##### a. Beban mati



- Berat lapis perkerasan =  $0,05 \times 0,875 \times 2200 = 96,25 \text{ kg/m}$
- Berat pelat lantai kendaraan =  $0,20 \times 0,875 \times 2500 = 437,50 \text{ kg/m}$
- Berat air hujan =  $0,05 \times 0,875 \times 1000 = 43,75 \text{ kg/m}$
- Berat profil dan dek baja =  $0,875 \times 11,35 = 9,931 \text{ kg/m} +$   
 $q_{DL} = 587,431 \text{ kg/m}$



Beban Trapezium diubah menjadi beban Ekuivalen :

$$\frac{qDL}{24} \times (3L^2 - 4a^2) = \frac{qE}{8} \times L^2$$

$$\frac{587,431}{24} \times (3 \times 5^2 - 4 \times 0,875^2) = \frac{qE}{8} \times 5^2$$

$$qE = 563,444 \text{ kg/m}$$

$$\begin{aligned} \text{Beban mati yang bekerja pada gelagar Tengah} &= 2 \times qE \\ &= 2 \times 563,444 \\ &= 1126,888 \text{ kg/m} \end{aligned}$$

$$\text{Berat Sendiri Profil Gelagar Memanjang} = 49,6 \text{ kg/m}$$

( Diasumsikan menggunakan profil IWF 350 x 175 x 7 x 11 – 49,6 )

$$\begin{aligned} \text{Beban Mati Total ( } qDL \text{ )} &= 1126,888 + 49,6 \\ &= 1176,480 \text{ kg/m} \end{aligned}$$

**Gaya Geser maksimum akibat beban mati ( Dmak DL ) :**

$$\begin{aligned} D_{\text{mak DL}} &= \frac{1}{2} \times q \times L \\ &= \frac{1}{2} \times 1176,480 \times 5 \\ &= 2941,2 \text{ kg} \end{aligned}$$

**Momen maksimum akibat beban mati ( Mmax DL ) :**

$$\begin{aligned} M_{\text{max DL}} &= \frac{1}{8} \times q_{DL} \times l^2 \\ &= \frac{1}{8} \times 1176,480 \times 5^2 \\ &= 3676,5 \text{ kgm} \end{aligned}$$



### b. Beban Hidup

#### • Beban terbagi rata (“q”)

Bentang jembatan = 80 m, maka :

$$q = 1.1 (1 + 30/L) \text{ t/m'} \quad \text{untuk } L > 60 \text{ m}$$

$$= 1.1 (1 + 30/80) \text{ t/m'} = 1,65 \text{ t/m}$$

Untuk perhitungan momen dan gaya lintang :

$$\text{Beban terbagi rata (q')} = \frac{q}{2,75} \times \alpha \times s'$$

dimana :

$\alpha$  = faktor distribusi,  $\alpha = 0,75$  bila kekuatan gelagar melintang diperhitungkan,  $\alpha = 1,00$  bila kekuatan gelagar melintang tidak diperhitungkan

$s'$  = lebar pengaruh beban hidup pada gelagar tepi = 1,75 m

$$q' = \frac{1,65}{2,75} \times 0,75 \times 1,75 = 0,7875 \text{ t/m} = 787,5 \text{ kg/m}$$

Ketentuan penggunaan beban “D” dalam arah melintang jembatan :

- Untuk jembatan dengan lebar lantai kendaraan lebih besar dari 5,50 meter, beban “D” sepenuhnya (100%) dibebankan pada lebar jalur 5,50 meter sedang lebar selebihnya dibebani hanya separuh beban “D” (50%).

Beban Hidup terbagi rata pada gelagar tengah :

$$q' = 100 \% \times 787,5 \text{ kg/m} = 787,5 \text{ kg/m}$$

#### • Beban garis “P”

$P = 12 \text{ ton}$

Untuk perhitungan momen dan gaya lintang :

$$\text{Beban garis (P')} = \frac{P}{2,75} \times \alpha \times s' \times K, \quad \text{dimana :}$$

$K$  = koefisien kejut, yang ditentukan dengan rumus :

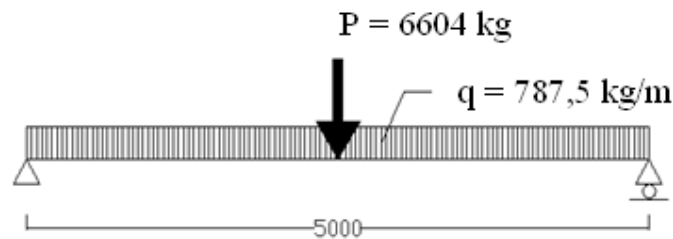
$$K = 1 + \left( \frac{20}{(50 + 80)} \right) = 1 + \left( \frac{20}{(50 + 80)} \right) = 1,153$$

$$\begin{aligned}
 P' &= \frac{P}{2,75} \times \alpha \times s' \times K = \frac{12}{2,75} \times 0,75 \times 1,75 \times 1,153 \\
 &= 6,604 \text{ T} = 6604 \text{ kg}
 \end{aligned}$$

Ketentuan penggunaan beban “D” dalam arah melintang jembatan :

Untuk jembatan dengan lebar lantai kendaraan lebih besar dari 5,50 meter, beban “D” sepenuhnya (100%) dibebankan pada lebar jalur 5,50 meter sedang lebar selebihnya dibebani hanya separuh beban “D” (50%).

$$P' = 100 \% \times 6604 = 6604 \text{ kg}$$



**Gaya geser maksimum akibat beban hidup ( Dmax LL ) :**

$$\begin{aligned}
 D_{makLL} &= \frac{1}{2} p' + \frac{1}{2} q' L \\
 &= \left( \frac{1}{2} \times 6604 \right) + \left( \frac{1}{2} \times 787,5 \times 5 \right) \\
 &= 5270,75 \text{ kg}
 \end{aligned}$$

**Momen maksimum akibat beban hidup (Mmax LL) :**

$$\begin{aligned}
 M_{max} &= \left( \frac{1}{8} \times q' \times l^2 \right) + \left( \frac{1}{4} \times P \times l \right) \\
 &= \left( \frac{1}{8} \times 787,5 \times 5^2 \right) + \left( \frac{1}{4} \times 6604 \times 5 \right) \\
 &= 10715,938 \text{ kgm}
 \end{aligned}$$

**Gaya geser total pada gelagar tengah :**

$$\begin{aligned}
 D_{tot} &= D_{makDL} + D_{makLL} \\
 &= 2941,20 \text{ kg} + 5270,75 \text{ kg} \\
 &= 8211,95 \text{ kg}
 \end{aligned}$$

**Momen total pada gelagar tengah :**

$$\begin{aligned} M_{tot} &= M_{max DL} + M_{max LL} \\ &= 3676,50 + 10715,938 \\ &= 14392,437 \text{ kgm} \end{aligned}$$

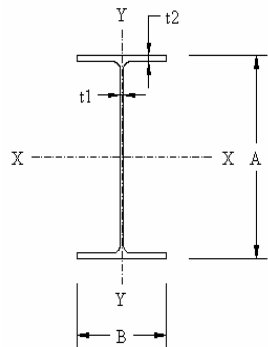
**2) Pendimensionian Profil gelagar tengah**

$$M_{tot} = 14392,437 \text{ kgm} = 1439243,7 \text{ kgcm}$$

$$\bar{\sigma}_{Bj 44} = 1867 \text{ kg/cm}^2$$

$$W_x = \frac{M_{tot}}{\bar{\sigma}} = \frac{1439243,7}{1867} = 770,885 \text{ cm}^3$$

Digunakan profil baja IWF 350 × 175 x 7 x 11 – 49,6



Profil WF	Berat (kg/m)	Ukuran (mm)				
		A	B	t1	t2	r
350 x 175	49,6	350	175	7	11	14

Luas tampang	Momen Inersia		Jari-jari Inersia		Momen Lawan	
	Ix	Iy	ix	iy	Wx	Wy
63,14	13600	984	14,7	3,95	775	112

**3) Kontrol terhadap bahan dan tegangan**

- Kontrol terhadap lendutan ( $\delta$ )

$$\begin{aligned} \delta_{\max} &= \frac{5 \times q_{tot} \times L^4}{384 EI_x} + \frac{P \times L^3}{48 EI_x} < \delta_{ijin} \\ &= \frac{5 \times (11,764 + 7,875) \times 500^4}{384 \times (2,1 \times 10^6) \times 13600} + \frac{6604 \times 500^3}{48 \times (2,1 \times 10^6) \times 13600} < \frac{L}{500} \\ &= 0,559 + 0,409 < 1,00 \text{ cm} \\ &= 0,968 \text{ cm} < 1,00 \text{ cm} \dots \dots \dots \text{OK} \end{aligned}$$

- Kontrol terhadap tegangan lentur yang terjadi ( $\sigma$ ) :

$$\begin{aligned}
 \sigma_{terjadi} &= \frac{M_{tot}}{W_x} < \bar{\sigma} \\
 &= \frac{1439243,7}{775} < 1867 \text{ kg/cm} \\
 &= 1857,088 \text{ kg/cm}^2 < 1867 \text{ kg/cm}^2 \dots\dots\dots \text{OK}
 \end{aligned}$$

- Kontrol terhadap tegangan geser yang terjadi ( $\tau$ )

$$\begin{aligned}
 D_{\max} &= \left( \frac{1}{2} \times q_{tot} \times L \right) + \left( \frac{1}{2} \times P \right) \\
 &= \left( \frac{1}{2} \times (11,764 + 7,875) \times 500 \right) + \left( \frac{1}{2} \times 6604 \right) \\
 &= 8211,75 \text{ kg}
 \end{aligned}$$

$$\begin{aligned}
 A_{web} &= A_{profil} - A_{flens} \\
 &= 72,16 - (2 \times (19,9 \times 1,1)) \\
 &= 28,38 \text{ cm}^2
 \end{aligned}$$

$$\begin{aligned}
 \tau_{terjadi} &= \frac{D_{\max}}{A_{web}} < \bar{\tau} \\
 &= \frac{8211,75}{28,38} < 0,58 \times \bar{\sigma} \\
 &= 289,349 \text{ kg/cm}^2 < 1082,86 \text{ kg/cm}^2 \dots\dots\dots \text{OK}
 \end{aligned}$$

### 5.2.5 Perencanaan Gelagar Melintang

Pembebanan pada gelagar melintang meliputi :

a. Beban Mati

Terdiri dari berat sendiri gelagar dan beban yang bekerja di atasnya (gelagar memanjang, pelat lantai jembatan, perkerasan, dan air hujan).

b. Beban Hidup

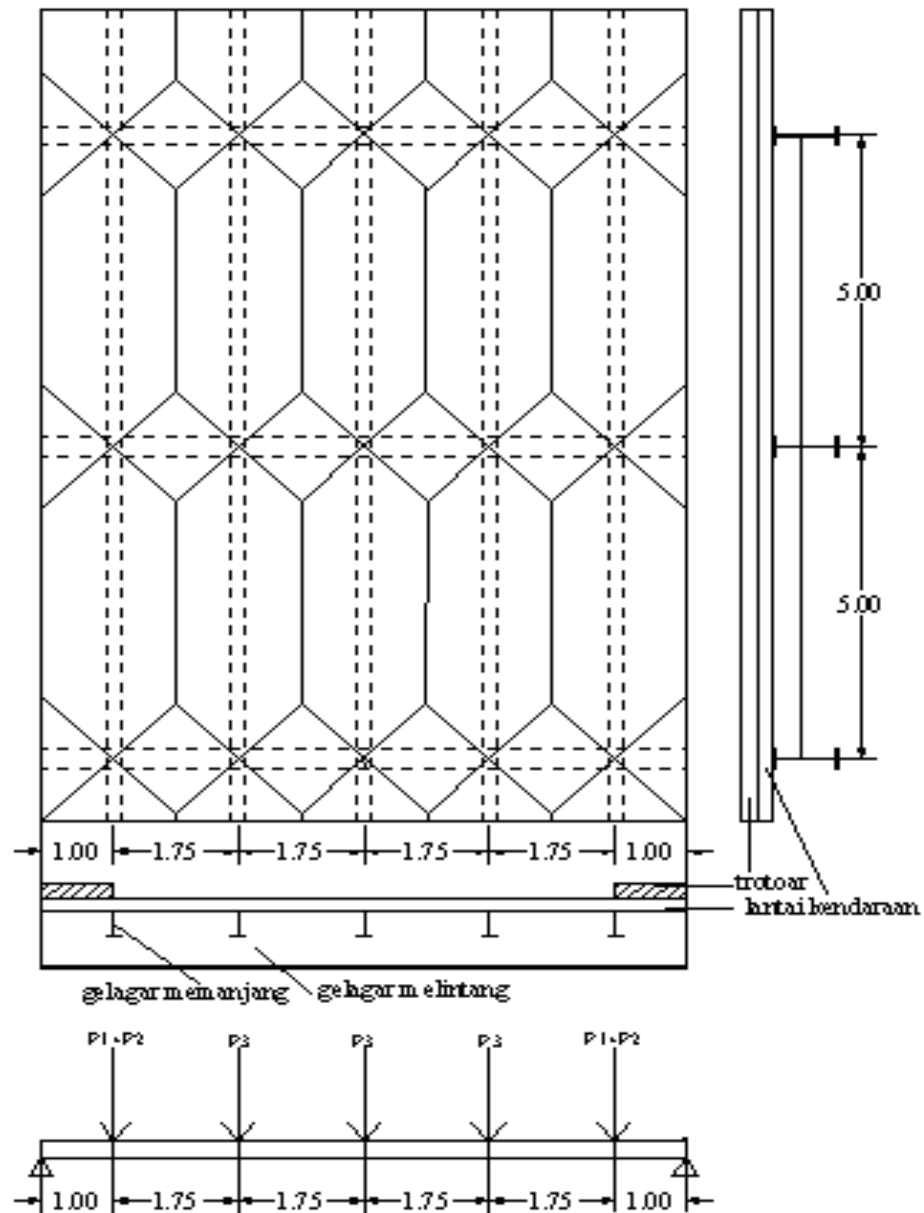
Beban hidup pada gelagar jembatan dinyatakan dengan beban “D” atau beban jalur, yang terdiri dari beban terbagi rata “q” ton per meter panjang per jalur lalu lintas tersebut.

Pada jembatan rangka baja, elemen struktur komposit terbentuk melalui kerjasama antara gelagar melintang dengan pelat beton. Factor penting dalam struktur komposit adalah lekatan antara gelagar melintang dengan pelat beton harus tetap ada. Untuk menjaga agar lekatan ini tetap ada, perlu adanya penghubung geser (shear connector) yang berfungsi untuk menahan gaya geser yang terjadi pada bidang pertemuan antara pelat beton dengan gelagar melintang. Pemakaian dek baja dibawah pelat beton berfungsi sebagai cetakan tetap dan untuk menahan momen positif yang terjadi pada pelat beton. Pemasangan dek baja sejajar dengan gelagar melintang.

#### 5.2.5.1 Kondisi Pre Komposit

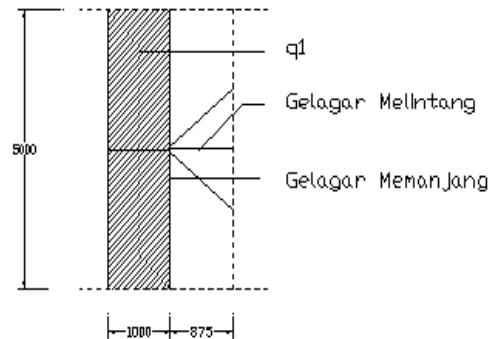
Kondisi pre komposit adalah kondisi dimana pelat beton belum mengeras dan beban hidup belum bekerja

## 1. Perhitungan Momen Lentur Gelagar Melintang



Gambar 5.12 Beban Mati Pada Kondisi Pre Komposit

### Beban P1



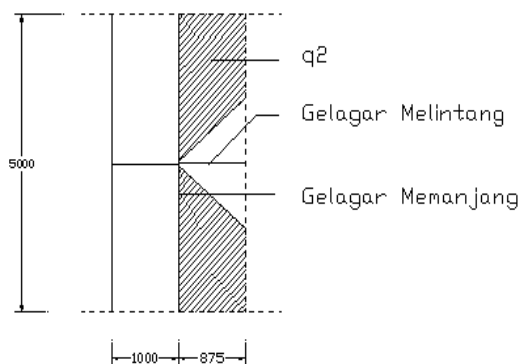
Berat trotoar	$= 0,25 \times 1,00 \times 2500$	$= 625$	kg/m
Berat plat lantai	$= 0,20 \times 1,00 \times 2500$	$= 500$	kg/m
Berat air hujan	$= 0,05 \times 1,00 \times 1000$	$= 50$	kg/m
Berat dek baja	$= 1,00 \times 11,35$	$= 11,35$	kg/m +
		$= 1311,35$	kg/m

Beban mati tersebut merupakan gaya terpusat ( P1 ) yang bekerja pada titik tumpu gelagar melintang :

$$P1 = q1 \times L$$

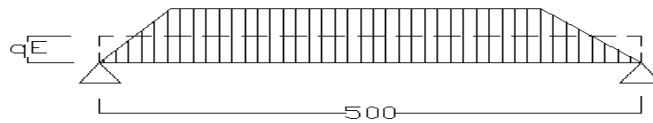
$$= 1311,35 \times 5,00 = 6556,75 \text{ kg}$$

### Beban P2



Berat plat lantai kendaraan	$= 0,20 \times 0,875 \times 2500$	$= 437,5$	kg/m
Berat air hujan	$= 0,05 \times 0,875 \times 1000$	$= 43,75$	kg/m

$$\begin{aligned}\text{Berat dek baja} &= 0,875 \times 11,35 &= 9,93 \text{ kg/m} + \\ & &= 491,18 \text{ kg/m}\end{aligned}$$



Beban Trapezium diubah menjadi beban Ekuivalen :

$$\frac{qDL}{24} \times (3L^2 - 4a^2) = \frac{qE}{8} \times L^2$$

$$\frac{491,18}{24} \times (3 \times 5^2 - 4 \times 0,875^2) = \frac{qE}{8} \times 5^2$$

$$qE = 471,123 \text{ kg/m}$$

Beban mati tersebut merupakan gaya terpusat ( P2 ) yang bekerja pada titik tumpu gelagar melintang :

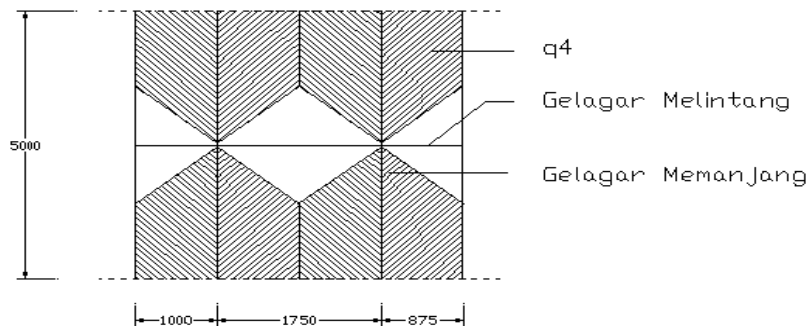
$$\begin{aligned}P2 &= qE \times L \\ &= 471,123 \times 5,00 = 2355,620 \text{ kg}\end{aligned}$$

### Beban P3

Berat gelagar memanjang IWF 350 x 175 x 7 x 11 – 49,6 = 49,6 kg/m

$$P3 = 49,6 \times 5,00 = 248 \text{ kg}$$

### Beban P4

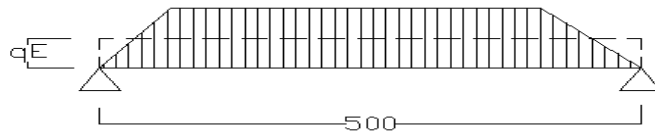


$$\text{Berat plat lantai kendaraan} = 0,20 \times 0,875 \times 2500 = 437,5 \text{ kg/m}$$

$$\text{Berat air hujan} = 0,05 \times 0,875 \times 1000 = 43,75 \text{ kg/m}$$



$$\begin{aligned}\text{Berat dek baja} &= 0,875 \times 11,35 &= 9,93 \text{ kg/m} + \\ & &= 491,18 \text{ kg/m}\end{aligned}$$



Beban Trapezium diubah menjadi beban Ekuivalen :

$$\frac{qDL}{24} \times (3L^2 - 4a^2) = \frac{qE}{8} \times L^2$$

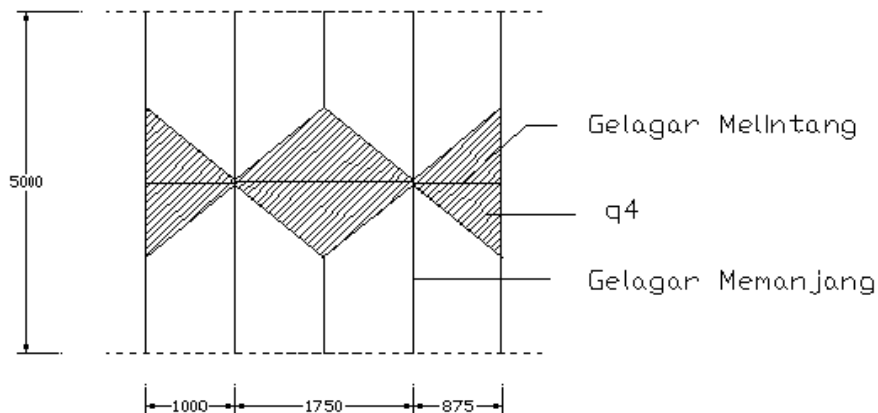
$$\frac{491,18}{24} \times (3 \times 5^2 - 4 \times 0,875^2) = \frac{qE}{8} \times 5^2$$

$$qE = 471,123 \text{ kg/m}$$

Beban mati tersebut merupakan gaya terpusat ( P4 ) yang bekerja pada titik tumpu gelagar melintang :

$$\begin{aligned}P4 &= ( 2 \times qE \times L ) + ( \text{berat gelagar memanjang} \times 5 ) \\ &= ( 2 \times 471,123 \times 5,00 ) + ( 49,6 \times 5 ) \\ &= 4994,23 \text{ kg}\end{aligned}$$

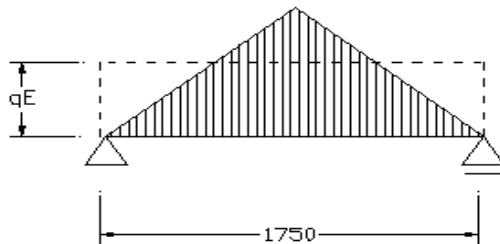
#### Beban q4



$$\text{Berat plat lantai kendaraan} = 0,20 \times 0,875 \times 2500 = 437,5 \text{ kg/m}$$

$$\text{Berat air hujan} = 0,05 \times 0,875 \times 1000 = 43,75 \text{ kg/m}$$

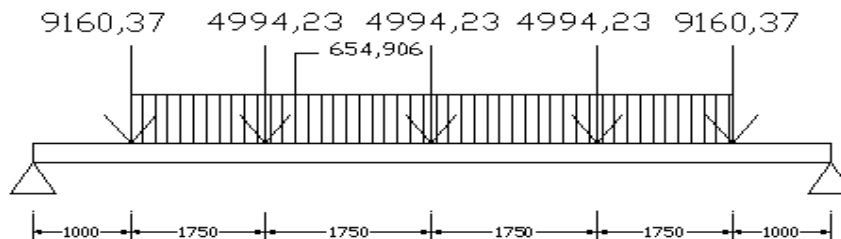
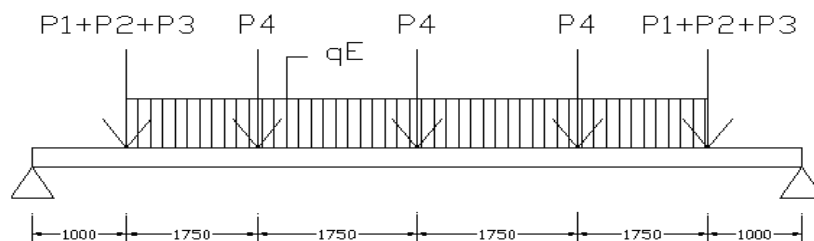
$$\begin{aligned}\text{Berat dek baja} &= 0,875 \times 11,35 &= 9,93 \text{ kg/m} + \\ & &= 491,18 \text{ kg/m}\end{aligned}$$



Beban segitiga diubah menjadi beban merata ekivalen :

$$\begin{aligned}\frac{qE}{12} \times L^2 &= \frac{qE}{8} \times L^2 \\ \frac{491,18}{12} \times 1,75^2 &= \frac{491,18}{8} \times 1,75^2 \\ qE &= 327,453 \text{ kg/m}\end{aligned}$$

$$\text{beban merata ekivalen yang bekerja} = 2 \times qE = 654,906 \text{ kg/m}$$



Reaksi Perletakan :

$$R_A = R_B = \frac{(3 \times P4) + (2 \times (P1 + P2 + P3)) + (qE \times L)}{2}$$

$$= \frac{(3 \times 4994,23) + (2 \times 9160,37) + (654,906 \times 5))}{2}$$

$$= 18288,98 \text{ kg}$$

Momen maksimum akibat beban mati :

$$= (R_{AV} \times 4,5) - ((P_1 + P_2 + P_3) \times 3,5) - (P_4 \times 1,75) - (qE \times 3,5 \times 1,75)$$

$$= (18288,98 \times 4,5) - (9160,37 \times 3,5) - (4994,23 \times 1,75) - (654,906 \times 3,5 \times 1,75)$$

$$= 37487,913 \text{ kgm}$$

Berat sendiri gelagar melintang = 215 kg/m

Asumsi gelagar melintang memakai profil IWF 708x302x15x28-215

$$R_p = \frac{1}{2} \times q \times L$$

$$= \frac{1}{2} \times 215 \times 9$$

$$= 967,5 \text{ kg}$$

$$M_p = \frac{1}{8} \times q \times L^2$$

$$= \frac{1}{8} \times 215 \times 9^2$$

$$= 2176,875 \text{ kgm}$$

Perhitungan geser dan momen yang bekerja pada kondisi Pra-Komposit :

$$D_{PRA} = 18288,98 + 967,5$$

$$= 19256,48 \text{ kg}$$

$$M_{PRA} = 37487,913 + 2176,875$$

$$= 39664,788 \text{ kgm}$$

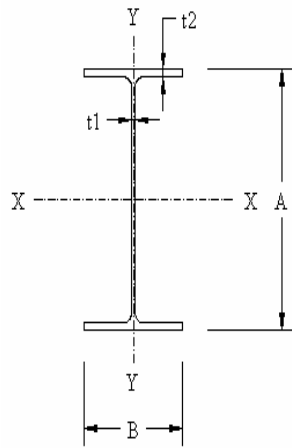
## 2. Pendimensian Gelagar Melintang

$$M_{PRA} = 39664,788 \text{ kgm} = 3966478,8 \text{ kgcm}$$

$$\bar{\sigma}_{Bj 44} = 1867 \text{ kg/cm}^2$$

$$W_x = \frac{M_{tot}}{\bar{\sigma}} = \frac{3966478,8}{1867} = 2124,519 \text{ cm}^3$$

Digunakan profil baja IWF 708x302x15x28-215

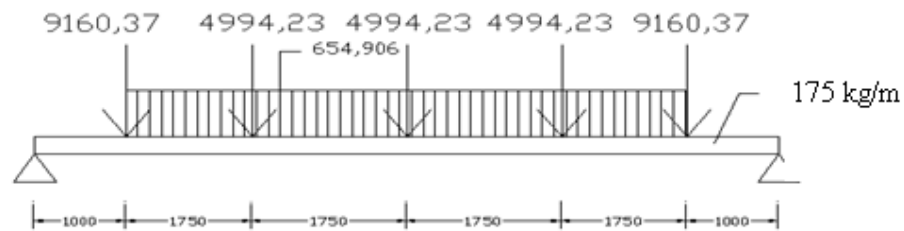


Profil WF	Berat (kg/m)	Ukuran (mm)				
		A	B	t1	t2	r
708 x 302	215	708	302	15	28	28

Luas tampang	Momen Inersia		Jari-jari Inersia		Momen Lawan	
	Ix	Iy	ix	iy	Wx	Wy
273,6	237000	12900	29,4	6,86	6700	853

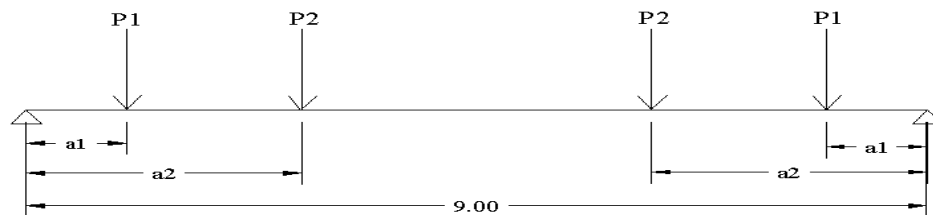
### 3. Kontrol Terhadap Bahan Dan Tegangan

- o Kontrol terhadap lendutan ( $\delta$ )



$$q = \frac{(215 \times 9) + (654,906 \times 7)}{9} = 724,371 \text{ kg/m} = 7,243 \text{ kg/cm}$$

- o Akibat beban terpusat di tepi



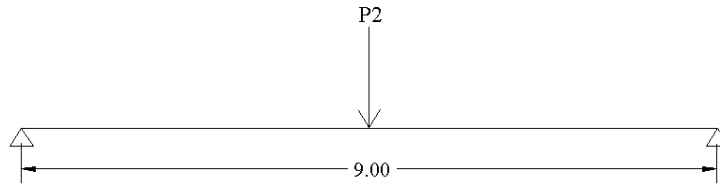
$$P1 = 9160,37 \text{ kg dan } P2 = 4994,23 \text{ kg}$$

$$\delta_1 = \frac{P1 \times a1}{24 EI} (3L^2 - 4a1^2) + \frac{P1 \times a2}{24 EI} (3L^2 - 4a2^2) =$$

$$\frac{9160,37 \times 100 (3 \times 900^2 - 4 \times 100^2)}{24 \times 2,1 \times 10^6 \times 237000} + \frac{4994,23 \times 275 (3 \times 900^2 - 4 \times 275^2)}{24 \times 2,1 \times 10^6 \times 237000}$$

$$= 0,183 + 0,244 = 0,427 \text{ cm}$$

- Akibat beban terpusat di tengah

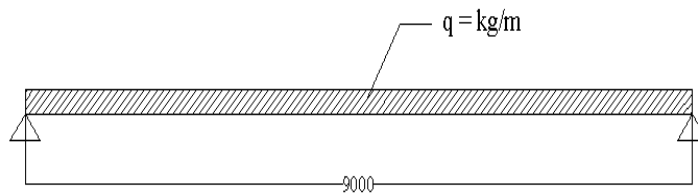


$$P2 = 4994,23 \text{ kg}$$

$$\delta_2 = \frac{P2 \times L^3}{48 EI}$$

$$= \frac{4994,23 \times 900^3}{48 \times 2,1 \times 10^6 \times 237000} = 0,152 \text{ cm}$$

- Akibat berat sendiri gelagar melintang



Gelagar melintang adalah IWF 594x302x14x23-175 dengan berat 175 kg/m

$$\delta_3 = \frac{5qxL^4}{384EI}$$

$$= \frac{5 \times 1,75 \times 900^4}{384 \times 2,1 \times 10^6 \times 237000} = 0,052 \text{ cm}$$

Lendutan total pada kondisi pra komposit adalah :

$$\delta_{\text{total}} = \delta_1 + \delta_2 + \delta_3$$

$$= 0,427 + 0,152 + 0,052$$

$$= 0,631 \text{ cm}$$

Lendutan Ijin ( $\delta_{ijin}$ )

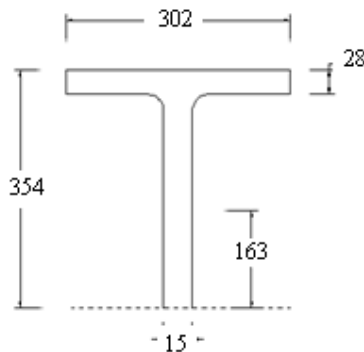
$$\delta_{ijin} = \frac{L}{500} = \frac{900}{500} = 1,800 \text{ cm}$$

$$\delta_{PRA-KOMP} = 0,631 < \delta_{ijin} = 1,800 \text{ cm} \dots\dots\text{OK}$$

- Kontrol terhadap tegangan lentur yang terjadi ( $\sigma$ ) :

$$\begin{aligned}\sigma_{terjadi} &= \frac{M_{tot}}{W_x} < \bar{\sigma} \\ &= \frac{3966478,8}{6700} < 1867 \text{ kg/cm} \\ &= 592,011 \text{ kg/cm}^2 < 1867 \text{ kg/cm}^2 \dots\dots\text{OK}\end{aligned}$$

- Kontrol terhadap tegangan geser yang terjadi ( $\tau$ ) :



$$\begin{aligned}S_x &= (30,2 \times 2,8 \times 34) + (1,5 \times 26,9 \times 16,3) \\ &= 3532,745 \text{ cm}^3\end{aligned}$$

$$\begin{aligned}\tau_{terjadi} &= \frac{D_{pra} \times S_x}{b \times I_x} < \tau \\ &= \frac{19256,48 \times 3532,745}{1,5 \times 237000} < 0,58 \times \sigma \\ &= 191,359 < 1082,86 \text{ kg/cm}^2 \dots\dots\text{OK}\end{aligned}$$

- Kontrol terhadap tegangan idiil ditengah bentang ( $\tau_i$ ) :

$$\begin{aligned} P &= P_3 + (q_E \times 7) + (q_D \times 9) \\ &= 4994,23 + (654,906 \times 7) + (215 \times 9) \\ &= 11513,572 \text{ kg} \end{aligned}$$

$$M_{\text{pra}} = 39664,788 \text{ kgm} = 3966478,8 \text{ kgcm}$$

$$\begin{aligned} \tau_{\text{terjadi}} &= \frac{P \times S}{b \times I_x} \\ &= \frac{11513,572 \times 3532,745}{1,5 \times 237000} \\ &= 114,149 \text{ kg/cm}^2 \end{aligned}$$

$$\begin{aligned} \sigma_{\text{terjadi}} &= \frac{M}{W_x} \\ &= \frac{3966478,8}{6700} \\ &= 592,011 \text{ kg/cm}^2 \end{aligned}$$

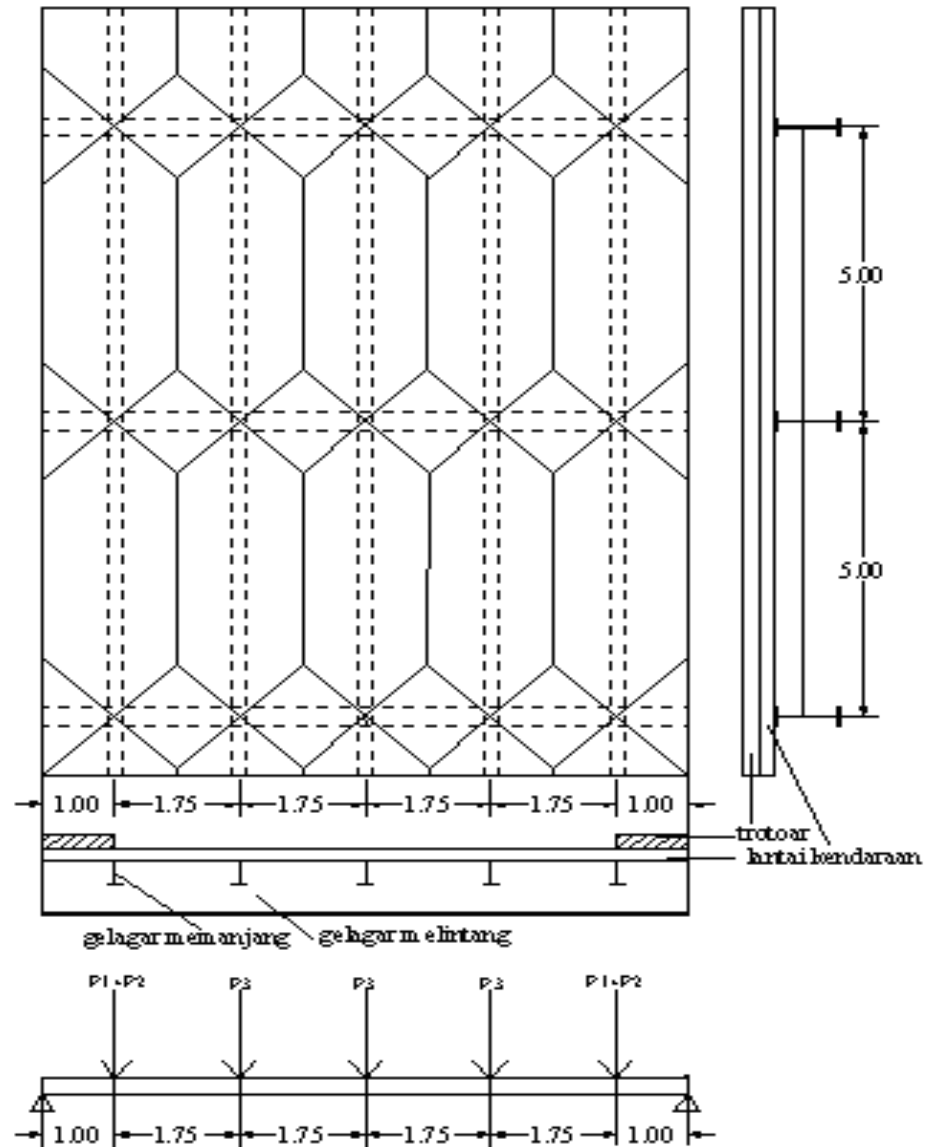
$$\begin{aligned} \tau_i &= \sqrt{\sigma^2 + (3 \times \tau^2)} &< \sigma \\ &= \sqrt{592,011^2 + (3 \times 114,149^2)} &< 1867 \text{ kg/cm}^2 \\ &= 624,153 \text{ kg/cm}^2 &< 1867 \text{ kg/cm}^2 \dots\dots \text{OK} \end{aligned}$$

### 5.2.5.2 Kondisi Post Komposit

Kondisi pre komposit adalah kondisi dimana pelat beton telah mengeras dan beban hidup telah bekerja

#### 1. Perhitungan Momen Lentur Gelagar Melintang

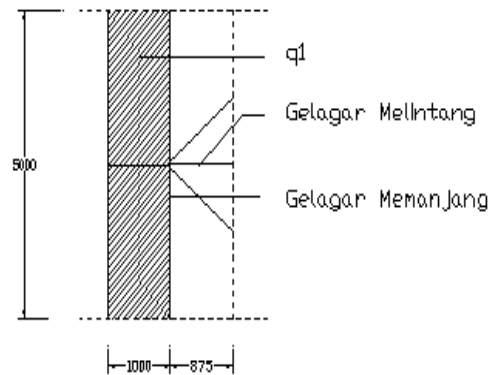
##### Beban Mati



Gambar 5.13 Beban Mati Pada Kondisi Post Komposit



### Beban P1

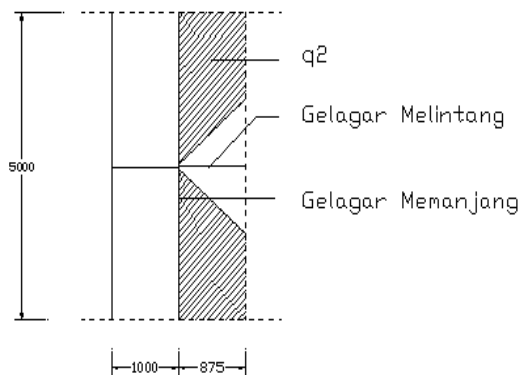


$$\text{Berat trotoar} = 0,25 \times 1,00 \times 2500 = 625 \text{ kg/m}$$

Beban mati tersebut merupakan gaya terpusat ( P1 ) yang bekerja pada titik tumpu gelagar melintang :

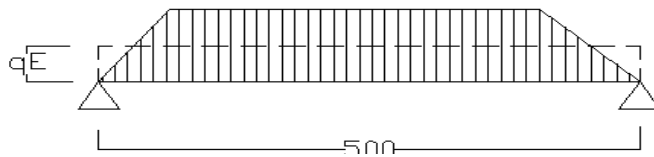
$$\begin{aligned} P1 &= q1 \times L \\ &= 625 \times 5,00 = 3125 \text{ kg} \end{aligned}$$

### Beban P2



$$\text{Berat air hujan} = 0,05 \times 0,875 \times 1000 = 43,75 \text{ kg/m}$$

$$\begin{aligned} \text{Berat lapis perkerasan} &= 0,05 \times 0,875 \times 2200 = 96,25 \text{ kg/m} + \\ &= 140 \text{ kg/m} \end{aligned}$$



Beban Trapezium diubah menjadi beban Ekuivalen :

$$\frac{qDL}{24} \times (3L^2 - 4a^2) = \frac{qE}{8} \times L^2$$

$$\frac{140}{24} \times (3 \times 5^2 - 4 \times 0,875^2) = \frac{qE}{8} \times 5^2$$

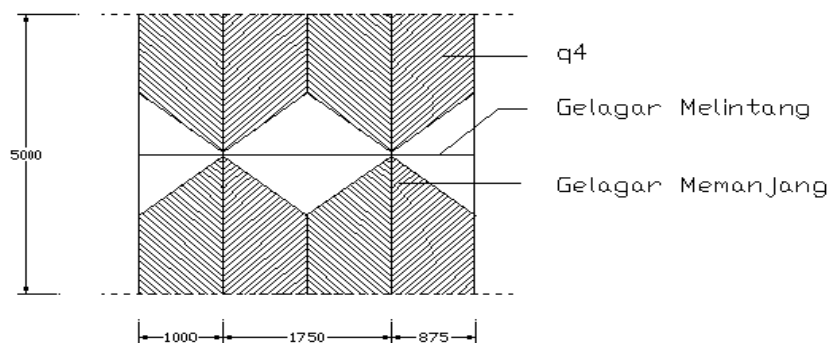
$$qE = 134,283 \text{ kg/m}$$

Beban mati tersebut merupakan gaya terpusat ( P2 ) yang bekerja pada titik tumpu gelagar melintang :

$$P2 = qE \times L$$

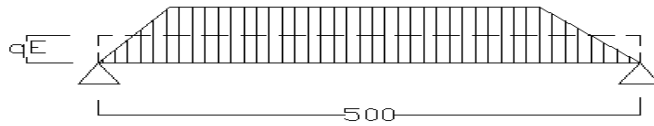
$$= 134,283 \times 5,00 = 671,415 \text{ kg}$$

### Beban P3



$$\text{Berat air hujan} = 0,05 \times 0,875 \times 1000 = 43,75 \text{ kg/m}$$

$$\begin{aligned} \text{Berat lapis perkerasan} &= 0,05 \times 0,875 \times 2200 = 96,25 \text{ kg/m} + \\ &= 140 \text{ kg/m} \end{aligned}$$



Beban Trapezium diubah menjadi beban Ekuivalen :

$$\frac{qDL}{24} \times (3L^2 - 4a^2) = \frac{qE}{8} \times L^2$$

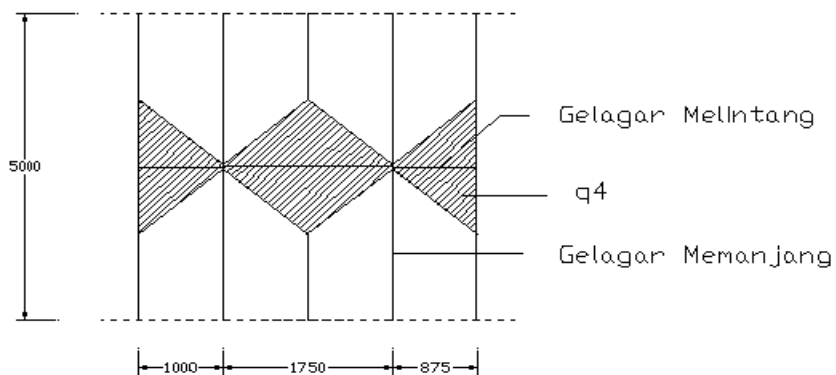
$$\frac{140}{24} \times (3 \times 5^2 - 4 \times 0,875^2) = \frac{qE}{8} \times 5^2$$

$$qE = 134,283 \text{ kg/m}$$

Beban mati tersebut merupakan gaya terpusat ( P4 ) yang bekerja pada titik tumpu gelagar melintang :

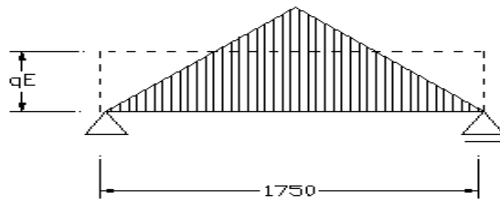
$$\begin{aligned} P3 &= ( 2 \times qE \times L ) \\ &= ( 2 \times 134,283 \times 5,00 ) \\ &= 1342,83 \text{ kg} \end{aligned}$$

#### Beban q4



$$\text{Berat air hujan} = 0,05 \times 0,875 \times 1000 = 43,75 \text{ kg/m}$$

$$\begin{aligned} \text{Berat lapis perkerasan} &= 0,05 \times 0,875 \times 2200 = 96,25 \text{ kg/m} + \\ &= 140 \text{ kg/m} \end{aligned}$$



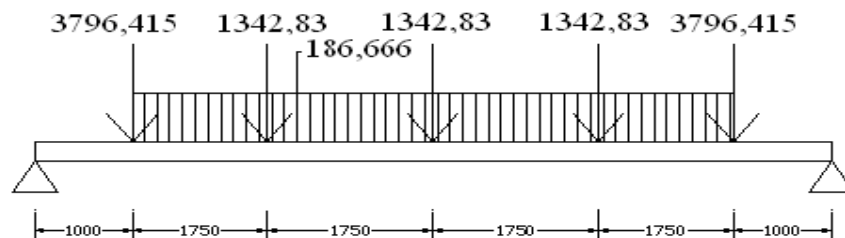
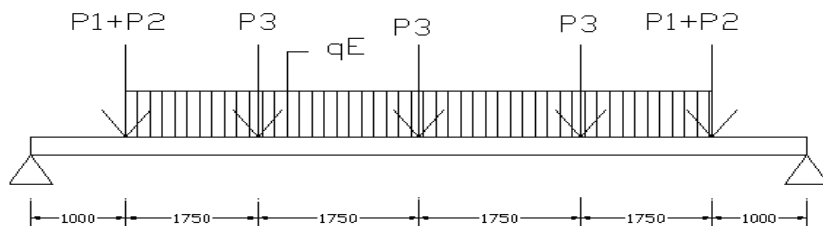
Beban segitiga diubah menjadi beban merata ekivalen :

$$\frac{qE}{12} \times L^2 = \frac{qE}{8} \times L^2$$

$$\frac{491,18}{12} \times 1,75^2 = \frac{qE}{8} \times 1,75^2$$

$$qE = 93,333 \text{ kg/m}$$

beban merata ekivalen yang bekerja =  $2 \times qE = 186,666 \text{ kg/m}$



Reaksi Perletakan :

$$R_A = R_B = \frac{(3 \times P3) + (2 \times ((P1 + P2))) + (qE \times L))}{2}$$

$$= \frac{(3 \times 1342,83) + (2 \times 3796,415) + (186,666 \times 5))}{2}$$

$$= 6277,31 \text{ kg (D1)}$$

Momen maksimum akibat beban mati :

$$= (R_{AV} \times 4,5) - ((P1 + P2) \times 3,5) - (P3 \times 1,75) - (qE \times 3,5 \times 1,75)$$

$$= (6277,31 \times 4,5) - (3796,415 \times 3,5) - (1342,83 \times 1,75) - (186,666 \times 3,5 \times 1,75)$$

$$= 11468,613 \text{ kgm (M1)}$$

## b. Beban Hidup

- Beban terbagi rata (“q”)

Bentang jembatan = 80 m, maka :

$$q = 1.1 (1 + 30/L) \text{ t/m'} \quad \text{untuk } L > 60 \text{ m}$$

$$= 1.1 (1 + 30/80) \text{ t/m'}$$

$$= 1,65 \text{ t/m}$$

- Beban terbagi rata sepanjang gelagar melintang untuk lebar 5,5 m

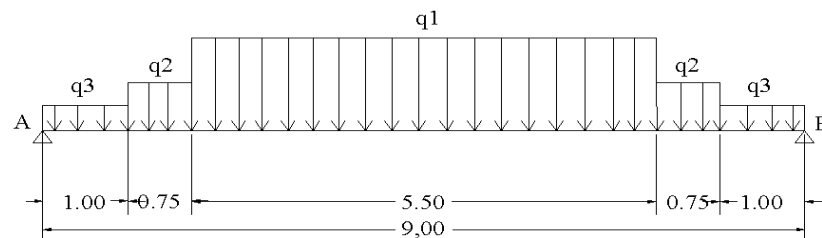
$$q_1 = \frac{q \times 5,5}{2,75} = \frac{1,65 \times 5,5}{2,75} = 3 \text{ t/m} = 3000 \text{ kg/m}$$

- Beban terbagi rata untuk lebar sisanya

$$q_2 = 50 \% \times 3000 \text{ kg/m} = 1500 \text{ kg/m}$$

- Beban terbagi rata pada trotoar

$$q_3 = 60\% \times (500 \times 500) = 1,5 \text{ ton/m} = 1500 \text{ kg/m}$$



Reaksi peletakan :

$$R_A = R_B = \frac{(q_1 \times 5,5) + (2 \times q_2 \times 0,75) + (2 \times q_3 \times 1,00)}{2}$$

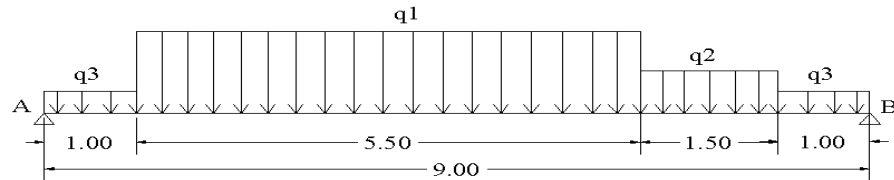
$$= \frac{(3000 \times 5,5) + (2 \times 1500 \times 0,75) + (2 \times 1500 \times 1,00)}{2}$$

$$= 10875 \text{ kg}$$

Momen maksimum yang terjadi akibat beban q :

$$\begin{aligned}
 &= (R_A \times 4,5) - (1500 \times 1,0 \times 4,0) - (q_2 \times 0,75 \times 3,125) - (q_1 \times 2,75 \times 1,375) \\
 &= (10875 \times 4,5) - (1500 \times 1,0 \times 4,0) - (1500 \times 0,75 \times 3,125) - (3000 \times 2,75 \times 1,375) \\
 &= 28078,125 \text{ kgm (M2)}
 \end{aligned}$$

Menentukan Geser Maksimum ( Dmak ) akibat beban q



Reaksi Peletakan

$$\Sigma M_A = 0$$

$$(R_B \times 9,0) - (q_3 \times 1,0 \times 8,5) - (q_2 \times 1,5 \times 7,25) - (q_1 \times 5,5 \times 3,75) - (q_3 \times 1,0 \times 0,5) = 0$$

$$(R_B \times 9,0) - (1,5 \times 1,0 \times 8,5) - (1,5 \times 1,5 \times 7,25) - (3 \times 5,5 \times 3,75) - (1,5 \times 1,0 \times 0,5) = 0$$

$$R_B = \frac{66,9375}{9} = 7,4375 \text{ t} = 7437,5 \text{ kg}$$

$$\Sigma M_B = 0$$

$$(R_A \times 9,0) - (q_3 \times 1,0 \times 8,5) - (q_1 \times 5,5 \times 5,25) - (q_2 \times 1,5 \times 1,75) - (q_3 \times 1,0 \times 0,5) = 0$$

$$(R_A \times 9,0) - (1,5 \times 1,0 \times 8,5) - (3 \times 5,5 \times 5,25) - (1,5 \times 1,5 \times 1,75) - (1,5 \times 1,0 \times 0,5) = 0$$

$$R_A = \frac{78,561}{9} = 8,729167 \text{ t} = 8729,167 \text{ kg (D2)}$$

- Beban “P”

$$P = 12 \text{ ton}$$

$$\text{Koefesien kejut (K)} = 1 + \left( \frac{20}{(50 + L)} \right)$$

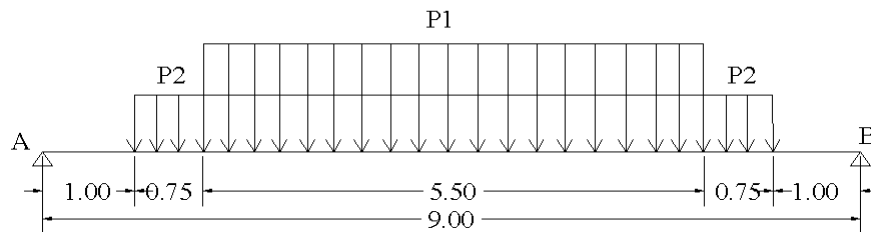
$$K = 1 + \left( \frac{20}{(50 + 80)} \right) = 1,182$$

- Beban P bekerjasepanjang gelagar melintang untuk lebar 5,5 m

$$P_1 = \frac{P}{2,75} \times K = \frac{12}{2,75} \times 1,182 = 5,158 \text{ t/m} = 5158 \text{ kg/m}$$

o Beban P untuk lebar sisanya ( 50% dari P1 )

$$P_2 = 50 \% \times 5158 \text{ kg/m} = 2579 \text{ kg/m}$$



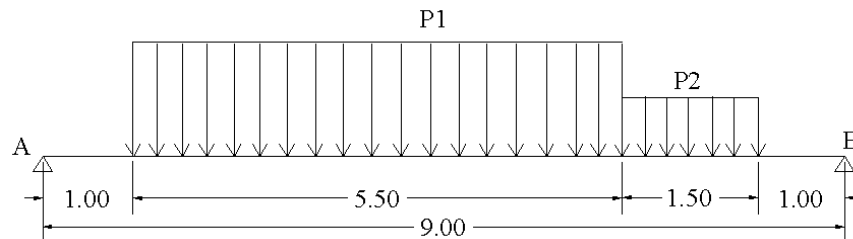
Reaksi Perletakan:

$$\begin{aligned} R_A &= \frac{(P_2 \times 0,75) + (P_1 \times 5,5) + (P_2 \times 0,75)}{2} \\ &= \frac{(2579 \times 0,75) + (5158 \times 5,5) + (2579 \times 0,75)}{2} \\ &= 16118,75 \text{ kg} \end{aligned}$$

Momen maksimum yang terjadi akibat beban garis "P"

$$\begin{aligned} M_{\max} &= (R_A \times 4,5) - (P_2 \times 0,75 \times 3,125) - (P_1 \times 2,75 \times 1,375) \\ &= (16118,75 \times 4,5) - (2579 \times 0,75 \times 3,125) - (5158 \times 2,75 \times 1,375) \\ &= 46986,156 \text{ kgm (M3)} \end{aligned}$$

Menentukan Geser Maksimum ( Dmak ) akibat beban P :



Reaksi Peletakan :

$$\Sigma M_A = 0$$

$$(R_B \times 9,0) - (P_1 \times 5,5 \times 3,75) - (P_2 \times 1,50 \times 7,25) = 0$$

$$(R_B \times 9,0) - (5158 \times 5,5 \times 3,75) - (2579 \times 1,50 \times 7,25) = 0$$

$$R_B = \frac{134430,375}{9} = 14936,708 \text{ kg}$$

$$\Sigma M_B = 0$$

$$(R_A \times 9,0) - (P_1 \times 5,5 \times 5,25) - (P_2 \times 1,50 \times 1,75) = 0$$

$$(R_A \times 9,0) - (5158 \times 5,5 \times 5,25) - (2579 \times 1,50 \times 1,75) = 0$$

$$R_A = \frac{155707,125}{9} = 17300,792 \text{ kg (D3)}$$

Perhitungan Momen dan Geser yang Bekerja

- Momen

$$\begin{aligned} M_{\text{post}} &= M_{\text{pra}} + M_1 + M_2 + M_3 \\ &= 39664,788 + 11468,613 + 28078,125 + 46986,156 \\ &= 126197,682 \text{ kgm} \end{aligned}$$

- Geser

$$\begin{aligned} D_{\text{post}} &= D_{\text{pra}} + D_1 + D_2 + D_3 \\ &= 19256,48 + 6277,31 + 8729,167 + 17300,792 \\ &= 51563,749 \text{ kg} \end{aligned}$$

## 2. Perhitungan Gelagar Komposit

Perhitungan lebar efektif :

$$\text{Syarat : } b_{\text{eff}} \leq \frac{1}{4} \text{ Bentang} = \frac{1}{4} \times 9,0 = 2,25 \text{ m}$$

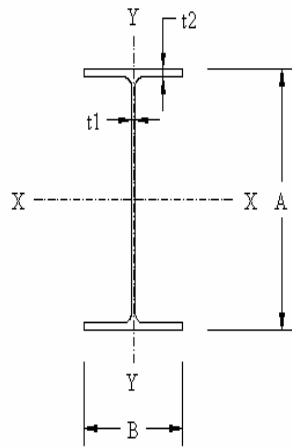
$$b_{\text{eff}} \leq \text{Jarak antar gelagar} = 5,0 \text{ m}$$

$$b_{\text{eff}} \leq 12 \times \text{Tebal pelat} = 12 \times 0,2 = 2,4 \text{ m}$$

Diambil nilai beff yang terkecil,  $b_{\text{eff}} = 2,25 \text{ m}$

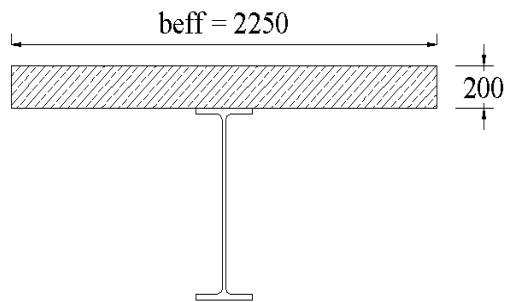
Data teknis profil IWF 708 × 302 x 15 x 28 – 215





Profil WF	Berat (kg/m)	Ukuran (mm)				
		A	B	t1	t2	r
708 x 302	215	708	302	15	28	28

Luas tampang	Momen Inersia		Jari-jari Inersia		Momen Lawan	
	Ix	Iy	ix	iy	Wx	Wy
273,6	237000	12900	29,4	6,90	6700	853



Angka Ekuivalen ( n ) :

$$E_s = 2,1 \times 10^5 \text{ MPa}$$

$$E_c = 4700 \sqrt{f'c} = 4700 \sqrt{25} = 23500 \text{ Mpa}$$

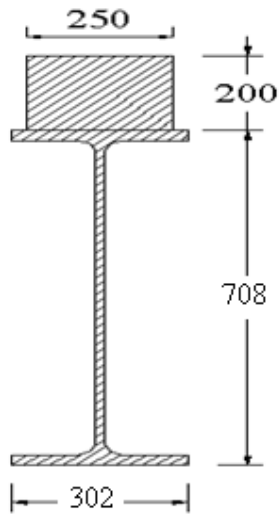
$$n = \frac{E_s}{E_c} = \frac{2,1 \times 10^5}{23500} = 8,9 \sim 9$$

Luas baja ekuivalen (  $A_{EKIVALEN}$  ) :

$$b' = \frac{b_{eff}}{n} = \frac{2250}{9} = 250 \text{ mm} = 25 \text{ cm}$$

$$\begin{aligned} A_{EKIVALEN} &= b' \times t \\ &= 25 \times 20 = 500 \text{ cm}^2 \end{aligned}$$

$$A_{PROFIL} = 273,6 \text{ cm}^2$$



Luas penampang komposit (  $A_{\text{KOMPOSIT}}$  ) :

$$\begin{aligned} A_{\text{KOMPOSIT}} &= A_{\text{EKIVALEN}} + A_{\text{PROFIL}} \\ &= 500 + 273,6 \\ &= 773,6 \text{ cm}^2 \end{aligned}$$

Titik berat penampang komposit (  $Y_{\text{komp}}$  ) :

$$\begin{aligned} y &= \frac{\left( A_1 \times \left( \frac{h}{2} \right) \right) + \left( \left( \frac{b_e}{n} \right) \times t \times \left( h + \frac{t}{2} \right) \right)}{\left( A_1 + \left( \frac{b_e}{n} \times t \right) \right)} \\ &= \frac{\left( 273,6 \times \left( \frac{70,8}{2} \right) \right) + \left( \left( \frac{225}{9} \right) \times 20 \times \left( 70,8 + \frac{20}{2} \right) \right)}{\left( 273,6 + \left( \frac{225}{9} \times 20 \right) \right)} \\ &= 64,743 \text{ cm} \end{aligned}$$

Momen inersia penampang komposit (  $I_k$  ) :

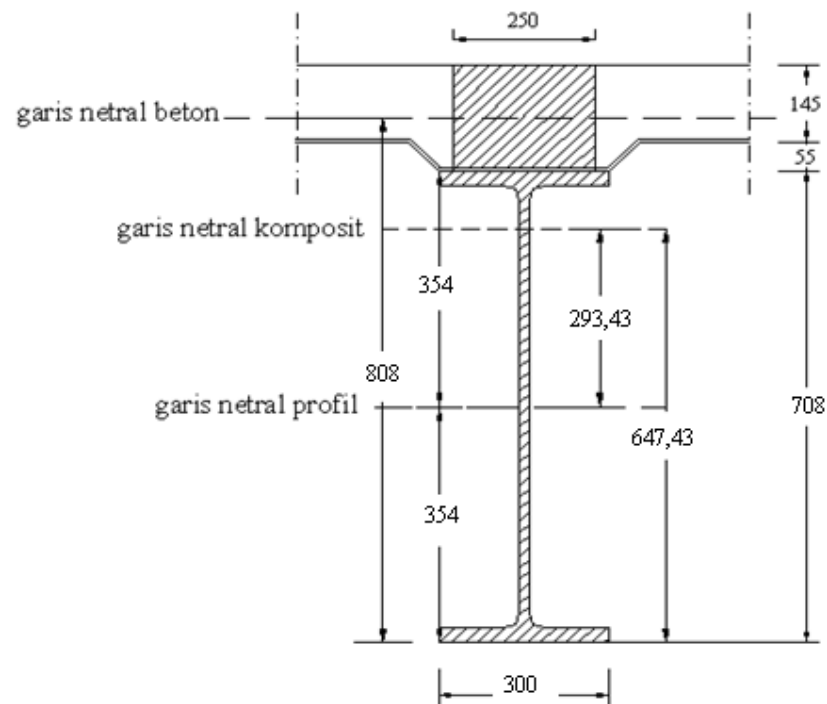
$$= \left( I_x + \left( A_1 \times \left( y - \frac{h}{2} \right)^2 \right) \right) + \left( \left( \frac{1}{12} \times \left( \frac{b_e}{n} \right) \times t^3 \right) + \left( \left( \frac{b_e}{n} \right) \times t \times \left( h + \frac{t}{2} - y \right)^2 \right) \right)$$

$$\begin{aligned}
 &= \left( 237000 + \left( 2736x \left( 64,743 - \frac{70,8}{2} \right)^2 \right) \right) + \left( \frac{1}{12} x 25 x 20^3 \right) + \left( 25x20x \left( 70,8 + \frac{20}{2} - 64,743 \right)^2 \right) \\
 &= 472572,787 + 16666,667 + 128913,624 \\
 &= 618153,078 \text{ cm}^4
 \end{aligned}$$

$$Y_{ts} = Y_{bs} = \frac{708}{2} = 354 \text{ mm} = 35,4 \text{ cm}$$

$$Y_c = h_{\text{profil}} + \frac{1}{2} x h_{\text{beton}} = 708 + \frac{1}{2} x 200 = 808 \text{ mm} = 80,8 \text{ cm}$$

Balok komposit direncanakan menggunakan dek baja trapesium dengan tinggi rusuk 55 mm dan tebal 4,5 mm.



Gambar 5.14 Titik Berat Penampang Komposit

### 3. Perhitungan Terhadap Tegangan

○ Kontrol terhadap tegangan lentur ( $\sigma$ )

- Pada bagian atas pelat beton

$$\begin{aligned}\sigma_c &= \frac{M_{post} \times (h + t - y)}{n \times I_k} < \bar{\sigma}_c \\ &= \frac{12619768,2 \times (70,8 + 20 - 64,743)}{9 \times 618153,078} < 0,45 \times 250 \\ &= 59,106 \text{ kg/cm}^2 < 112,5 \text{ kg/cm}^2 \dots\dots\dots \text{OK}\end{aligned}$$

- Pada bagian bawah pelat beton

$$\begin{aligned}\sigma_c &= \frac{M_{post} \times (h - y)}{n \times I_k} < \bar{\sigma}_c \\ &= \frac{12619768,2 \times (70,8 - 64,743)}{9 \times 618153,078} < 0,45 \times 250 \\ &= 13,739 \text{ kg/cm}^2 < 112,5 \text{ kg/cm}^2\end{aligned}$$

- Pada sayap atas profil baja

$$\begin{aligned}\sigma_{BS} &= \frac{M_D \times \left(\frac{h}{2}\right)}{I_x} + \frac{M_L \times (h - y)}{I_K} < \bar{\sigma}_s \\ &= \frac{(3966478,8 + 1146861,3) \times \left(\frac{70,8}{2}\right)}{237000} + \frac{(2807812,5 + 4698615,6) \times 6,057}{618153,078} \\ &= 763,764 + 73,552 < 1867 \text{ kg/cm}^2 \\ &= 837,316 \text{ kg/cm}^2 < 1867 \text{ kg/cm}^2 \dots\dots\dots \text{OK}\end{aligned}$$

- Pada sayap bawah profil baja

$$\sigma_{BS} = \frac{M_D \times \left(\frac{h}{2}\right)}{I_x} + \frac{M_L \times y}{I_K} < \bar{\sigma}_s$$

$$\frac{(3966478,8 + 1146861,3) \times \left(\frac{70,8}{2}\right)}{137000} + \frac{(2807812,5 + 4698615,6) \times 64,743}{618153,078}$$

$$= 763,764 + 786,194 < 1867 \text{ kg/cm}^2$$

$$= 1549,958 \text{ kg/cm}^2 < 1867 \text{ kg/cm}^2 \dots\dots\dots \text{OK}$$

○ Diagram tegangan sebelum dan sesudah komposit

- Tegangan sebelum komposit ( pra komposit )

Pada sayap atas profil baja = 592,011 kg/cm<sup>2</sup>

Pada sayap bawah profil baja = 592,011 kg/cm<sup>2</sup>

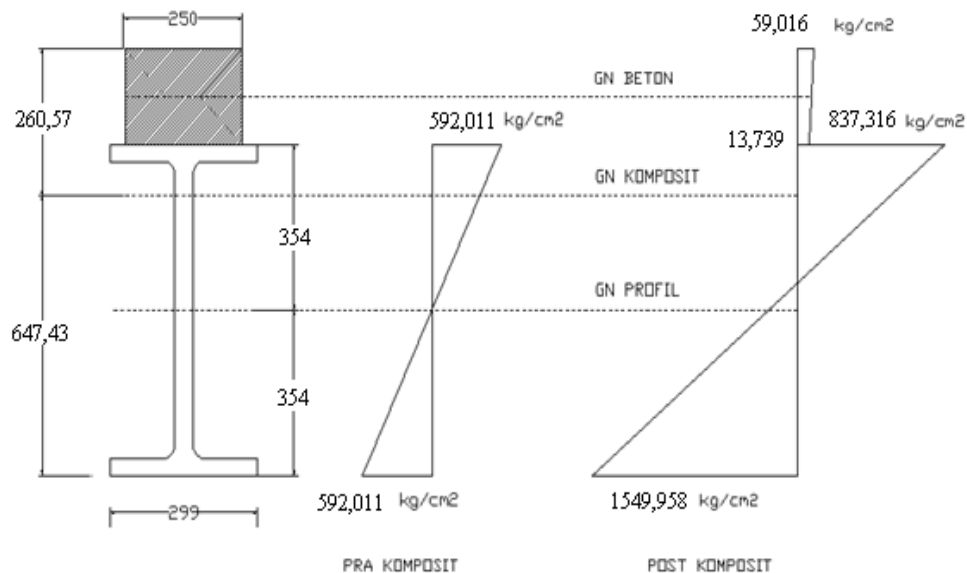
- Tegangan sesudah komposit ( post komposit )

Pada bagian atas pelat beton = 59,016 kg/cm<sup>2</sup>

Pada bagian bawah pelat beton = 13,739 kg/cm<sup>2</sup>

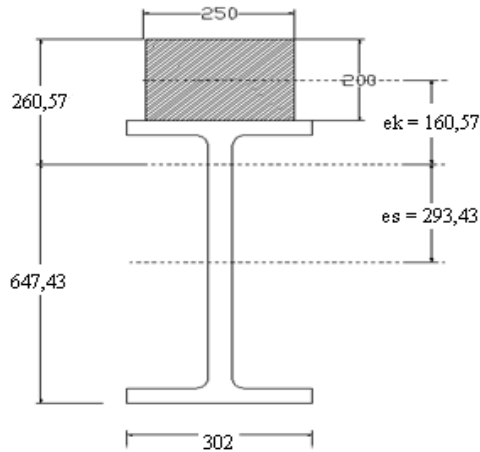
Pada sayap atas profil baja = 837,316 kg/cm<sup>2</sup>

Pada sayap bawah profil baja = 1549,958 kg/cm<sup>2</sup>



Gambar 5.15 Diagram Tegangan Sebelum Dan Sesudah Komposit

- Kontrol terhadap tegangan Geser ( $\tau$ )



Statis momen terhadap garis netral komposit :

- Pada Plat Beton

$$\begin{aligned}
 S_{x1} &= b' \times t \times e_k \\
 &= 250 \times 200 \times 160,57 \\
 &= 8028500 \text{ mm}^3 = 8028,5 \text{ cm}^3
 \end{aligned}$$

- Pada Profil baja

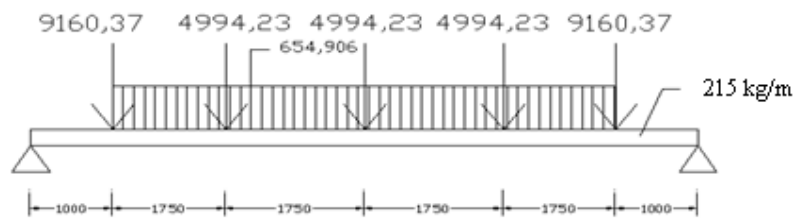
$$\begin{aligned}
 S_{x1} &= A_{\text{profil}} \times e_s \\
 &= 273,6 \times 293,43 \\
 &= 80282,448 \text{ mm}^3 = 80,282 \text{ cm}^3
 \end{aligned}$$

$$\begin{aligned}
 \tau_{\text{TERJADI}} &= \frac{D_{\text{POST}} \times (S_{x1} + S_{x2})}{b \times I_x} < \bar{\tau} \\
 \tau_{\text{TERJADI}} &= \frac{51563,749 \times (8028,5 + 80,282)}{1,5 \times 237000} < 0,58 \times \bar{\sigma} \\
 &= 1176,144 < 1082,86 \text{ kg/cm}^2 \text{ .....OK}
 \end{aligned}$$

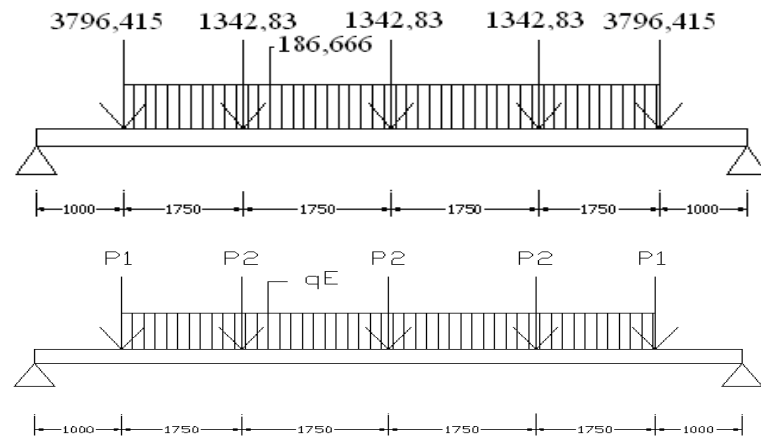
- Kontrol terhadap Lendutan ( $\tau$ )

1. Akibat beban mati ( pada kondisi pre komposit dan post komposit )

- Kondisi pre komposit



- Kondisi post komposit



$$\begin{aligned} P1 &= P_{\text{PRE KOMPOSIT}} + P_{\text{KOMPOSIT}} \\ &= 9160,37 + 3796,415 = 12956,785 \text{ kg} \end{aligned}$$

$$\begin{aligned} P2 &= P_{\text{PRE KOMPOSIT}} + P_{\text{KOMPOSIT}} \\ &= 4994,23 + 1342,83 = 6337,06 \text{ kg} \end{aligned}$$

$$\begin{aligned} qE &= qE_{\text{PRE KOMPOSIT}} + qE_{\text{KOMPOSIT}} \\ &= 654,906 + 186,666 = 841,572 \text{ kg/m} \end{aligned}$$

$$qD = 215 \text{ kg/m}$$

$$q = \frac{(841,572 \times 7) + (215 \times 9)}{9} = 869,556 \text{ kg/m}$$

$$\delta_1 = \left( \frac{5xqxL^4}{384EI_x} \right) + \sum \left( \frac{Pxa}{24EI_x} x(3L^2 - 4a^2) \right) + \left( \frac{PxL^3}{48EI_x} \right)$$

$$= \left( \frac{5 \times 8,67556 \times 900^4}{384(2,1 \times 10^6) \times 237000} \right) + \sum \left( \frac{12956,785 \times 100}{24(2,1 \times 10^6) \times 237000} \times (3 \times 900^2 - 4 \times 100^2) \right) +$$

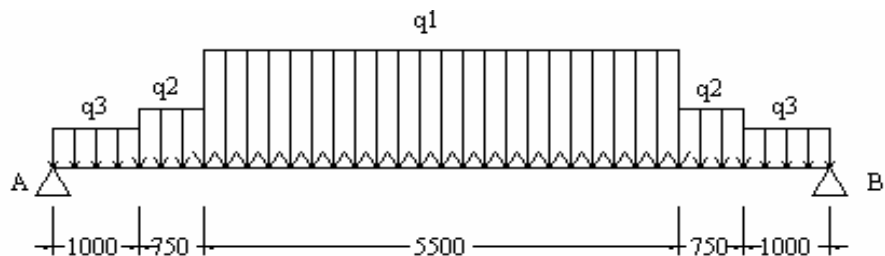
$$\left( \frac{6337,06 \times 275}{24 \times (2,1 \times 10^6) \times 237000} \times ((3 \times 900^2) - (4 \times 100^2)) \right) + \left( \frac{6337,06 \times 900^3}{48 \times (2,1 \times 10^6) \times 237000} \right)$$

$$= 0,148 + 0,259 + 0,348 + 0,193$$

$$= 0,948 \text{ cm}$$

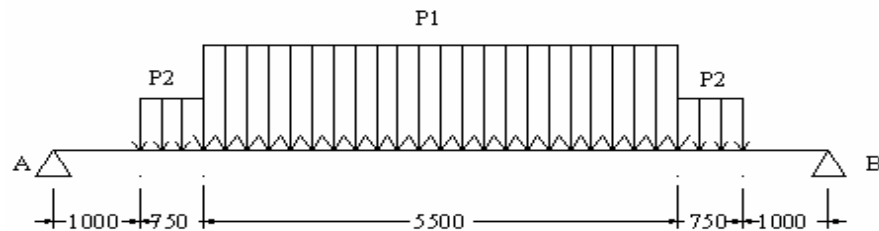
## 2. Akibat beban hidup

### a. Akibat beban terbagi merata ( " q " )



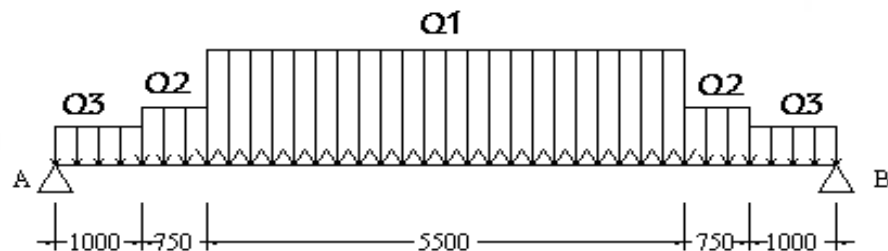
$$q_1 = 3000 \text{ kg/m}, \quad q_2 = 1500 \text{ kg/m}, \quad q_3 = 1500 \text{ kg/m}$$

### b. Akibat beban garis ( " P " )



$$P_1 = 5158 \text{ kg/m}$$

$$P_2 = 2579 \text{ kg/m}$$





$$\begin{aligned} Q1 &= q1 + P1 \\ &= 3000 + 5158 = 8158 \text{ kg/m} \end{aligned}$$

$$\begin{aligned} Q2 &= q2 + P2 \\ &= 1500 + 2579 = 4079 \text{ kg/m} \end{aligned}$$

$$Q3 = q3 = 1500 \text{ kg/m}$$

$$\begin{aligned} q_{\text{EKIVALEN}} &= \frac{(Q1 \times 7) + (2 \times Q2 \times 0,75) + (2 \times Q3 \times 1)}{9} \\ &= \frac{(8158 \times 7) + (2 \times 4079 \times 0,75) + (2 \times 1500 \times 1)}{9} \\ &= 7358,278 \text{ kg/m} \end{aligned}$$

$$\begin{aligned} \delta_2 &= \frac{5 \times q_{\text{ekuivalen}} \times L^4}{384 EI_K} \\ &= \frac{5 \times 73,58278 \times 900^4}{384 \times (2,1 \times 10^6) \times 618153,078} \\ &= 0,484 \text{ cm} \end{aligned}$$

#### Lendutan total ( $\delta_{\text{total}}$ )

$$\begin{aligned} \delta_{\text{total}} &= \delta_1 + \delta_2 \\ &= 0,948 + 0,484 \\ &= 1,432 \text{ cm} \end{aligned}$$

#### Lendutan ijin ( $\delta_{\text{ijin}}$ )

$$\delta_{\text{ijin}} = \frac{L}{500} = \frac{900}{500} = 1,8 \text{ cm}$$

$$\delta_{\text{total}} = 1,432 \text{ cm} < \delta_{\text{ijin}} = 1,8 \text{ cm} \dots \dots \dots \text{OK}$$

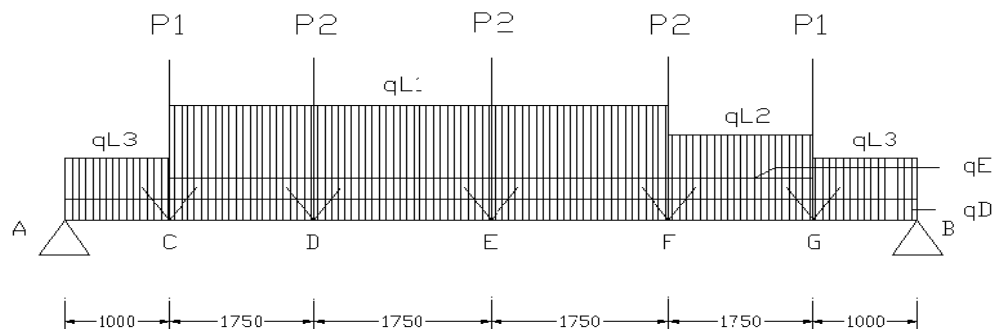
### 5.2.6 Perhitungan Penghubung Geser ( *Shear Connector* )

*Shear Connector* digunakan untuk menahan gaya geser memanjang yang terjadi pada bidang pertemuan anatar pelat beton dengan belok baja.

Syarat teknis perencanaan *shear connector* dengan menggunakan stud adalah :

- Jarak minimal antar stud arah memanjang balok  $5d$  dan tidak kurang 10 cm
- Jarak maksimal antar stud tidak boleh lebih dari delapan kali tebal pelat beton, atau kurang dari 800 mm
- Jarak antar stud tegak lurus balok tidak boleh kurang dari  $d + 3$  cm
- Panjang minimal stud  $4d$
- Jarak minimal ujung stud dengan permukaan beton 4 cm

### Perhitungan Gaya Lintang



Gambar 5.16 Pembebanan Pada Perhitungan Shear Connector

Pembebanan :

#### a. Beban Mati Terpusat

- P1 = beban mati terpusat yang bekerja pada titik C dan G adalah beban mati terpusat pada kondisi pre komposit dan post komposit  
$$= 9160,37 \text{ kg} + 3796,415 \text{ kg}$$
$$= 12956,785 \text{ kg}$$
- P2 = beban mati terpusat yang bekerja pada titik D, E dan f adalah beban mati terpusat pada kondisi pre komposit dan post komposit

$$= 4994,23 \text{ kg} + 1342,83 \text{ kg}$$

$$= 6337,06 \text{ kg}$$

b. Beban Mati Merata

- $q_E = q_{E_{\text{PREKOMPOSIT}}} + q_{E_{\text{POST KOMPOSIT}}}$ 
 $= 654,906 + 186,666$ 
 $= 841,572 \text{ kg/m}$

c. Berat Sendiri Gelagar Melintang

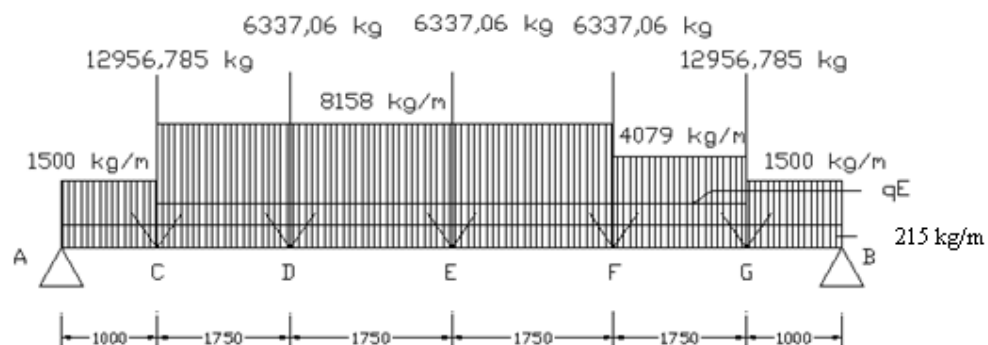
- $q_D = \text{beban mati merata ( berat sendiri gelagar melintang )} = 215 \text{ kg/m}$

d. Berat Hidup ( beban " D " )

- $q_{L1} = \text{beban " D " untuk lebar 5,5 meter}$ 
 $= \text{beban terbagi rata ( q )} + \text{beban gars ( P )}$ 
 $= 3000 \text{ kg/m} + 5158 \text{ kg/m}$ 
 $= 8158 \text{ kg/m}$
- $q_{L2} = \text{beban " D " untuk lebar sisanya ( 2 x 0,75 m )}$ 
 $= \text{beban terbagi rata ( q )} + \text{beban gars ( P )}$ 
 $= 1500 \text{ kg/m} + 2579 \text{ kg/m}$ 
 $= 4079 \text{ kg/m}$

e. Berat Hidup Pada Trotoar

- $q_{L3} = \text{beban hidup pada trotoar} = 1500 \text{ kg/m}$



Reaksi Perletakan :

$$\sum M_A = 0$$

$$(R_B \times 9) - (215 \times 9 \times 4,5) - (841,572 \times 7 \times 4,5) - (1500 \times 1 \times 8,5) - (4079 \times 1,75 \times 7,125) - (8158 \times 5,25 \times 3,625) - (1500 \times 1 \times 0,5) - (12956,785 \times 8) - (6337,06 \times 6,25) - (6337,06 \times 4,5) - (6337,06 \times 2,75) - (12956,785 \times 1) = 0$$

$$(R_B \times 9) - 8626,5 - 26509,518 - 12750 - 50860,03125 - 155256,9375 - 750 - 103654,28 - 39606,625 - 28516,77 - 17426,915 - 12956,785 = 0$$

$$9 R_B = 456914,3618$$

$$R_B = 50768,26242 \text{ kg}$$

$$\sum M_B = 0$$

$$(R_A \times 9) - (215 \times 9 \times 4,5) - (841,572 \times 7 \times 4,5) - (1500 \times 1 \times 0,5) - (4079 \times 1,75 \times 1,875) - (8158 \times 5,25 \times 5,375) - (1500 \times 1 \times 8,5) - (12956,785 \times 1) - (6337,06 \times 2,75) - (6337,06 \times 4,5) - (6337,06 \times 6,25) - (12956,785 \times 8) = 0$$

$$(R_B \times 9) - 8626,5 - 26509,518 - 750 - 13384,21875 - 230208,5625 - 12750 - 12956,785 - 17426,915 - 28516,77 - 39606,625 - 103654,28 = 0$$

$$9 R_B = 494390,1743$$

$$R_B = 54932,24158 \text{ kg}$$

Gaya Lintang :

$$D_A = 54932,24158 \text{ kg}$$

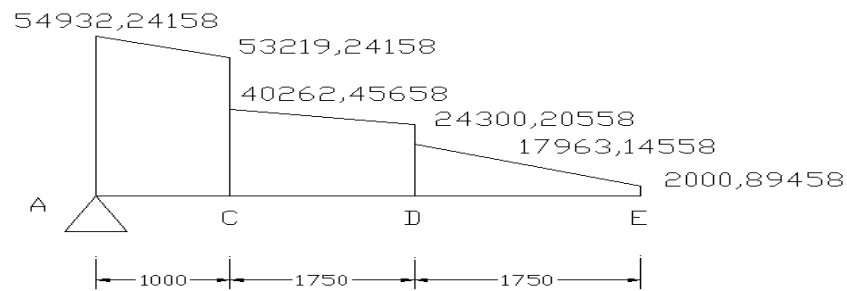
$$D_{A-C} = 54932,24158 - (1500 \times 1) - (213 \times 1) = 53219,24158 \text{ kg}$$

$$D_C = 53219,24158 - 12956,785 = 40262,45658 \text{ kg}$$

$$D_{C-D} = 40262,45658 - (213 \times 1) - (841,572 \times 1,75) - (8158 \times 1,75) = 24300,20558 \text{ kg}$$

$$D_D = 24300,20558 - 6337,06 = 17963,14558 \text{ kg}$$

$$D_{D-E} = 17963,14558 - (213 \times 1) - (841,572 \times 1,75) - (8158 \times 1,75) = 2000,89458 \text{ kg}$$

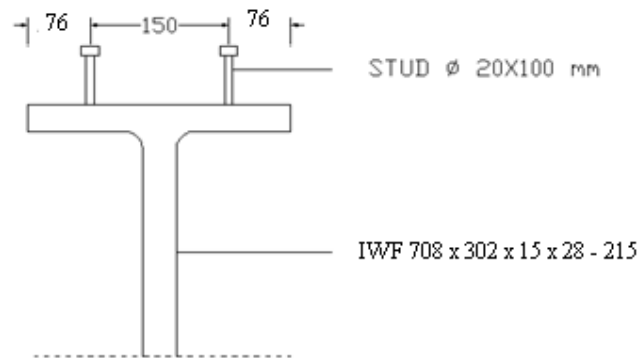


Gambar 5.17 Diagram Gaya Lintang

*Shear connector* direncanakan menggunakan stud Ø 20 mm dengan tinggi stud (H) = 100 mm. Jumlah stud dalam arah tegak lurus sumbu gelagar melintang = 2 buah.

Kekuatan satu stud :

$$Q = 0,0005 A_s \sqrt{f'_c \times E_c}$$



$$\begin{aligned} Q &= 0,0005 \times \left( \frac{1}{4} \pi D^2 \right) \times \sqrt{30 \times (4700 \sqrt{30})} \\ &= 0,0005 \times 314 \times 878,799 \\ &= 137,971 \text{ KN} \\ &= 13797,1 \text{ kg} \end{aligned}$$

$$\bar{Q} = \frac{Q}{2} = 6898,5 \text{ kg}$$

Jarak stud :

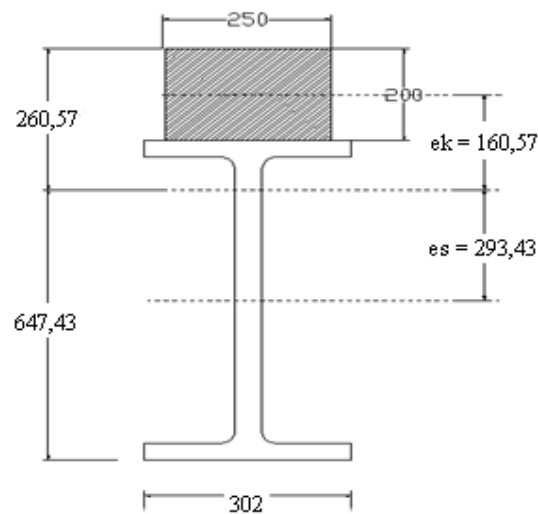
$$D = \frac{\bar{Q} \times I_k}{D \times S}$$

Dimana,  $\bar{Q}$  = Kekuatan stud dalam 1 baris (kg)

$I_k$  = Momen inersia penampang komposit ( $\text{cm}^4$ )

$D$  = Gaya lintang (k)

$S$  = Statis momen bagian yang menggeser terhadap garis netral penampang komposit



$$I_k = 618153,078 \text{ cm}^4$$

$$\begin{aligned} S &= 250 \times 200 \times 160,57 \\ &= 8028500 \text{ mm}^3 \\ &= 8028,5 \text{ cm}^3 \end{aligned}$$

$$d1 = \frac{(2 \times 6898,5) \times 618153,078}{54932,24158 \times 8028,5} = 19,338 \text{ cm} \sim 20 \text{ cm}$$

$$d2 = \frac{(2 \times 6898,5) \times 618153,078}{40262,45658 \times 8028,5} = 24,384 \text{ cm} \sim 25 \text{ cm}$$

$$d3 = \frac{(2 \times 6898,5) \times 618153,078}{17963,14558 \times 8028,5} = 59,137 \text{ cm} \sim 60 \text{ cm}$$

Sambungan antara stud dan gelagar melintang menggunakan sambungan las sudut.

Perhitungan Las Sudut :

$$a. \text{ Tebal Las } = a \leq \frac{1}{2} t \sqrt{2}$$

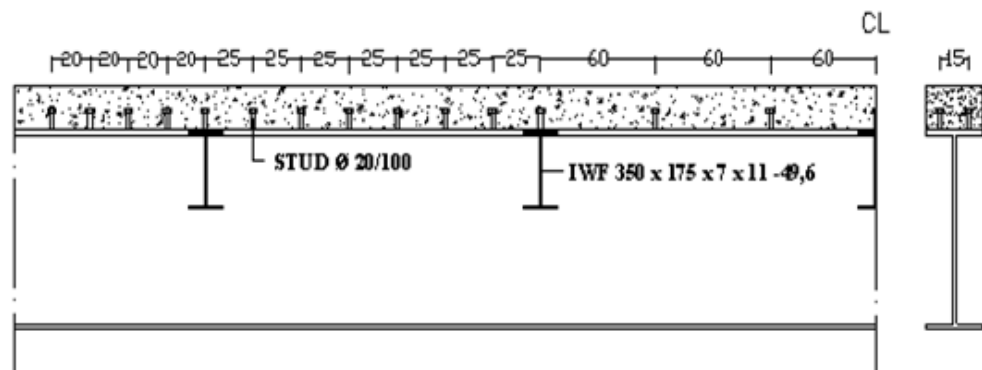
$$a \leq \frac{1}{2} \times 2,3 \times \sqrt{2}$$

$$a \approx 1,626 \text{ cm}$$

$$\begin{aligned} b. \text{ Luas Bidang Las} &= 0,25 \times \pi \times d^2 \\ &= 0,25 \times \pi \times (3,252 - 2)^2 \\ &= 1,230 \text{ cm}^2 \end{aligned}$$

$$\begin{aligned} c. \text{ Kekuatan Las} &= F \times \bar{\sigma} \\ &= 1,230 \times 6350 \\ &= 7810,5 \text{ kg} \end{aligned}$$

$$\text{Kekuatan satu stud} = 6898,5 \text{ kg} < 7810,5 \text{ kg}$$



Gambar 5.18 Pemasangan Shear Connector

### 5.2.7 Perhitungan Sambungan Gelagar Melintang dan Gelagar Memanjang

Besarnya  $D_{\max}$  gelagar memanjang (P) = 8211,75 kg

Untuk penyambungan antara gelagar melintang dan memanjang digunakan pelat penyambung profil L 130.130.14

Sambungan direncanakan menggunakan baut  $\phi$  2,54 cm

- Jarak antar baut :

$$3d \leq a \leq 6d$$

$$60 \leq a \leq 120$$

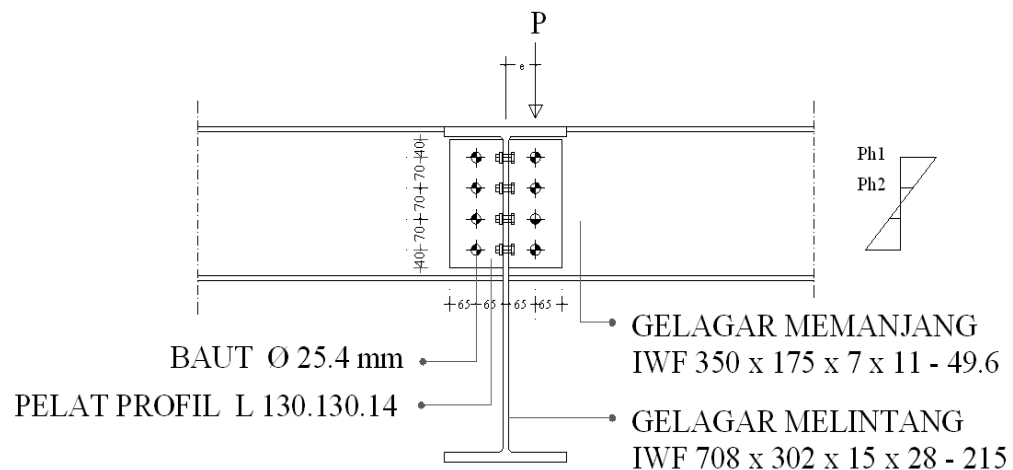
a diambil 70 mm

- Jarak baut ke tepi sambungan :

$$c \geq 2d$$

$$c \geq 40$$

$s_1$  diambil 40 mm



Gambar 5.19 Sambungan Gelagar Memanjang Dengan Protol Siku

Perhitungan gaya yang bekerja pada sambungan :

Pengaruh Desak

$$\frac{\delta}{d} = \frac{0,7}{2,54} = 0,275 < 0,628 \text{ (pengaruh desak)}$$



$$n_{ds} = \frac{P}{2 \times \sigma \times \delta \times d} = \frac{8211,75}{2 \times 1867 \times 0,7 \times 2,54} = 1,236 \sim \text{diambil 4 baut}$$

$$\text{Eksentrisitas (e)} = \frac{15}{2} + 65 = 72,5 \text{ mm}$$

Momen Luar (  $M_{\text{LUAR}}$  ) :

$$\begin{aligned} M_{\text{LUAR}} &= P \times e \\ &= 8211,75 \times 7,25 \\ &= 55429,312 \text{ kgcm} \end{aligned}$$

Momen Dalam (  $M_{\text{DALAM}}$  ) :

$$\begin{aligned} M_{\text{DALAM}} &= [(Ph \times y_2) + (Ph_1 \times y_1)] \times 2 \\ &= \left[ (Ph \times y_2) + \left( Ph \times \frac{y_1^2}{y_2^2} \right) \right] \times 2 = \frac{2 \times Ph}{y_2} (y_2^2 + y_1^2) \end{aligned}$$

Substitusi :

$$\frac{2 \times Ph}{y_2} (y_2^2 + y_1^2) = P \times e$$

$$Ph = \frac{P \times e \times y_2}{2 \times (y_2^2 + y_1^2)}$$

$$Ph = \frac{55429,312 \times 9}{2 \times (7^2 + 3,5^2)} = 3167,389 \text{ kg}$$

$$P_v = \frac{P}{n_{\text{baut}}} = \frac{8211,75}{4} = 2052,937 \text{ kg}$$

$$\begin{aligned} R &= \sqrt{P_v^2 + Ph^2} \\ &= \sqrt{2052,937^2 + 3167,389^2} \\ &= 3774,507 \text{ kg} \end{aligned}$$

Tegangan yang terjadi pada baut luar :

$$\begin{aligned} \sigma_{ds} &= \frac{R}{\delta \times d} = \frac{3774,507}{0,7 \times 2} < 1,5 \times \bar{\sigma} \\ &= 2696,076 \text{ kg/cm}^2 < 2800,5 \text{ kg/cm}^2 \end{aligned}$$

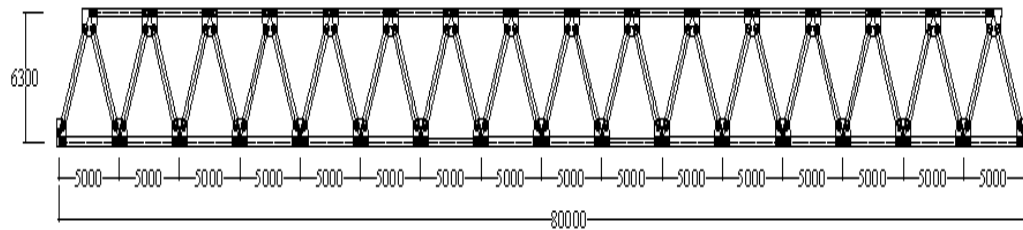


$$n_{ds} = \frac{P}{2 \times \bar{\sigma} \times \delta \times d} = \frac{8211,75}{2 \times 1867 \times 1,5 \times 2,54} = 0,58 \sim \text{diambil 4 baut}$$

Tegangan yang terjadi pada baut :

$$\begin{aligned} \sigma_{BAUT} &= \frac{P}{2 \times n_{BAUT} \times \left( \frac{1}{4} \times \pi \times d^2 \right)} < 0,6 \times \bar{\sigma} \\ &= \frac{8211,75}{2 \times 4 \times \left( \frac{1}{4} \times 3,14 \times 2,54^2 \right)} < 1120,2 \text{ kg/cm}^2 \\ &= 202,679 \text{ kg/cm}^2 < 1120,2 \text{ kg/cm}^2 \end{aligned}$$

## 5.2.9 Perhitungan Pertambahan Angin



Gambar 5.21 Bidang Rangka yang Terkena Angin

Data teknis perencanaan pertambahan angin :

Tekanan angin :  $150 \text{ kg/m}^2$

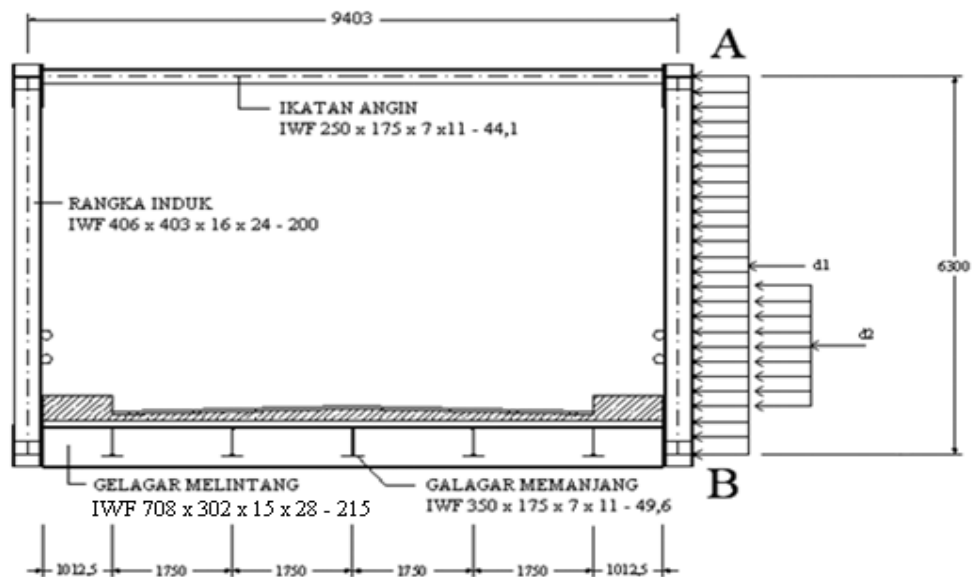
Panjang sisi bawah jembatan : 80 m

Panjang sisi atas jembatan : 75 m

Tinggi jembatan : 6,3 m

Luas bidang rangka utama :  $\left( \frac{80 + 75}{2} \right) \times 6,3 = 488,25 \text{ m}^2$

### 5.2.9.1 Pembebanan Ikatan Angin



Gambar 5.22 Penyebaran Beban Angin

**Bagian jembatan yang langsung terkena angin ( angin Tekan ) :**

- Beban angin pada sisi rangka jembatan (d1) :

$$\begin{aligned} d1 &= 50\% \times ((30\% \times A)) \times w \\ &= 50\% \times ((30\% \times 488,25)) \times 150 \\ &= 10985,625 \text{ kg} \end{aligned}$$

- Beban angin pada muatan hidup setinggi 2 m (d2) :

$$\begin{aligned} d2 &= 100\% \times w \times L \times 2 \\ &= 100\% \times 150 \times 80 \times 2 \\ &= 24000 \text{ kg} \end{aligned}$$

**Penentuan titik tangkap gaya akibat beban angin ( s ) :**

- Beban angin pada sisi rangka jembatan ( s1 )

$$\begin{aligned} s1 &= \frac{1}{2} \times \text{tinggi jembatan} \\ &= \frac{1}{2} \times 6,30 \text{ m} \\ &= 3,15 \text{ m} \end{aligned}$$

- Beban angin pada muatan hidup setinggi 2 m ( s2 )

Tinggi profil gelagar melintang ( h1 ) : 70,8 cm ( 708x302x15x28-215 )

Tebal sayap gelagar melintang ( h2 ) : 2,8 cm

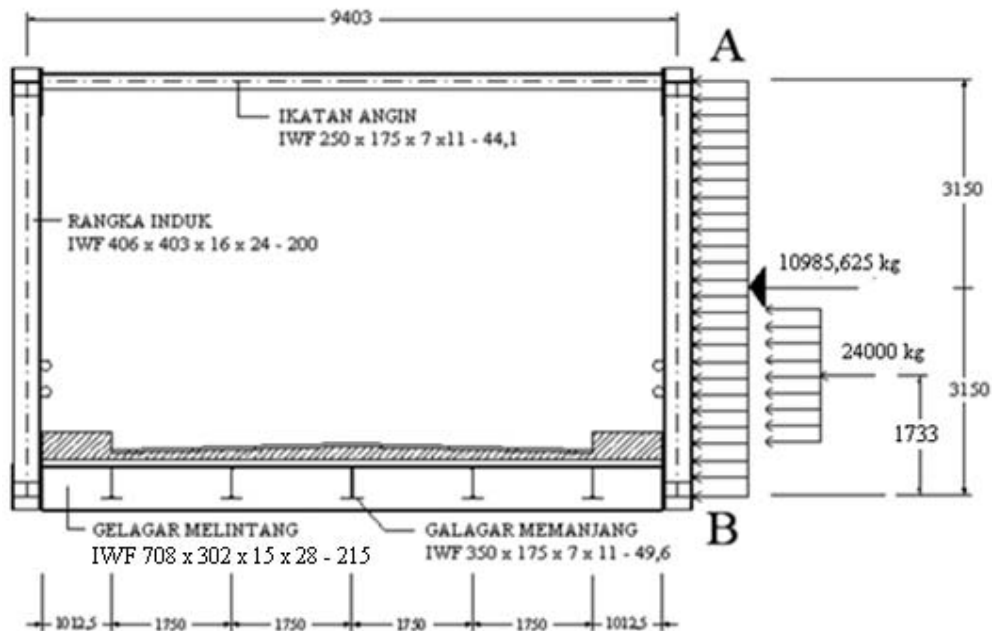
Lebar profil rangka induk ( h3 ) : 40,3 cm ( 406x403x16x24-200 )

Tebal plat lantai kendaraan ( h4 ) : 20 cm

Tebal perkerasan ( h5 ) : 5 cm

Tinggi bidang vertikal beban hidup ( h6 ): 200 cm

$$\begin{aligned} s2 &= \left( h1 - h2 - \frac{h3}{2} \right) + h4 + h5 + \frac{h6}{2} \\ &= ( 70,8 - 2,3 - 20,15 ) + 20 + 5 + 100 \\ &= 173,35 \text{ cm} = 1,733 \text{ m} \end{aligned}$$



Gambar 5.23 Titik Tangkap Gaya Angin Tekan

$$\begin{aligned}\Sigma M_B &= 0 \\ (R_A \times 6,30) - (d1 \times s1) - (d2 \times s2) &= 0 \\ (R_A \times 6,30) - (10985,625 \times 3,15) - (24000 \times 1,733) &= 0 \\ R_A &= \frac{76196,718}{6,3} = 12094,717 \text{ kg}\end{aligned}$$

$$\begin{aligned}\Sigma M_A &= 0 \\ (R_B \times 6,3) - (d1 \times s1) - (d2 \times (6,3 - s2)) &= 0 \\ (R_B \times 6,3) - (10985,625 \times 3,15) - (24000 \times 4,567) &= 0 \\ R_B &= \frac{144044,718}{6,3} = 22864,241 \text{ kg}\end{aligned}$$

Distribusi beban angin :

- Pada pertambahan angin atas

$$P1 = \frac{R_{A1}}{15} = \frac{12094,717}{15} = 806,314 \text{ kg}$$

- Pada pertambahan angin bawah

$$P_1 = \frac{R_B}{16} = \frac{22864,241}{16} = 1429,015 \text{ kg}$$

**Bagian jembatan yang tidak langsung terkena angin ( angin hisap ) :**

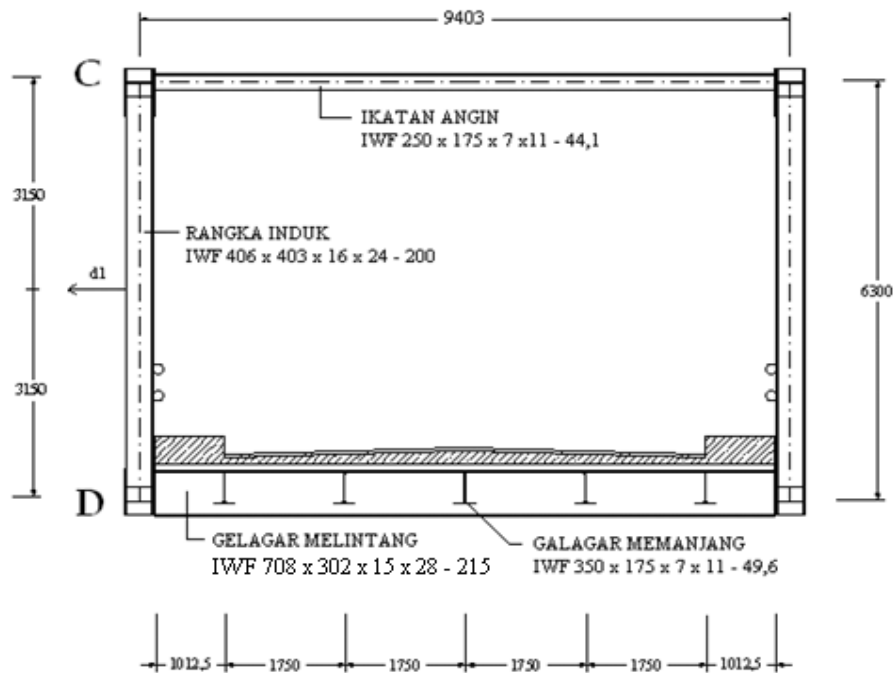
- Beban angin pada sisi rangka jembatan (d1) :

$$\begin{aligned} d1 &= 50\% \times (( 15\% \times A )) \times w \\ &= 50\% \times (( 15\% \times 488,25 )) \times 150 \\ &= 5492,812 \text{ kg} \end{aligned}$$

**Penentuan titik tangkap gaya akibat beban angin ( s ) :**

- Beban angin pada sisi rangka jembatan (s1)

$$\begin{aligned} s1 &= \frac{1}{2} \times \text{tinggi jembatan} \\ &= \frac{1}{2} \times 6,30 \text{ m} \\ &= 3,150 \text{ m} \end{aligned}$$



Gambar 5.24 Titik Tangkap Gaya Angin Hisap

$$R_A = R_B = \frac{5492,812}{6,30} = 871,875 \text{ kg}$$

Distribusi beban angin :

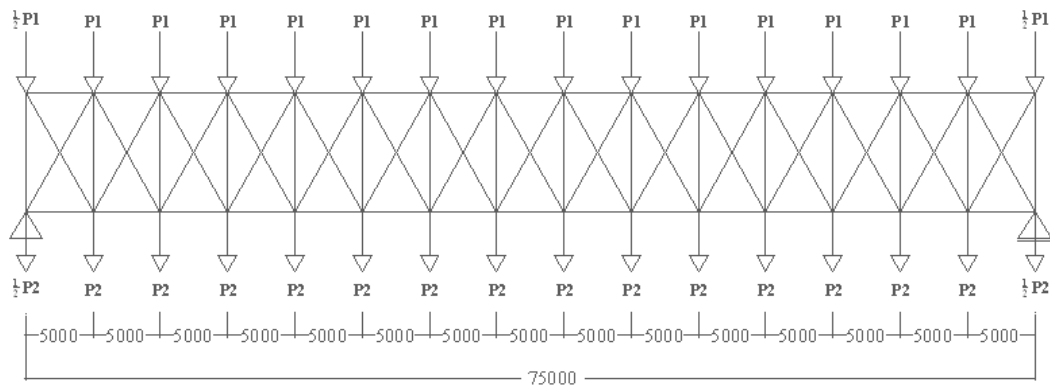
- Pada pertambahan angin atas

$$P2 = \frac{R_{A1}}{15} = \frac{871,875}{15} = 58,125 \text{ kg}$$

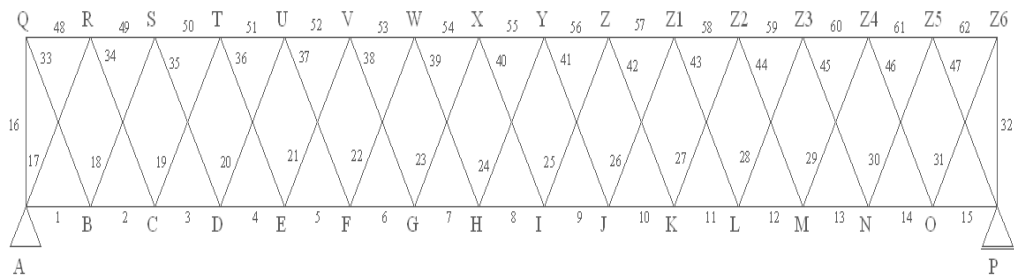
- Pada pertambahan angin bawah

$$P2 = \frac{R_B}{16} = \frac{871,875}{16} = 54,492 \text{ kg}$$

### 5.2.9.2 Pertambahan Angin Atas



Gambar 5.25 Penyebaran Beban Angin Pada Ikatan Angin Atas



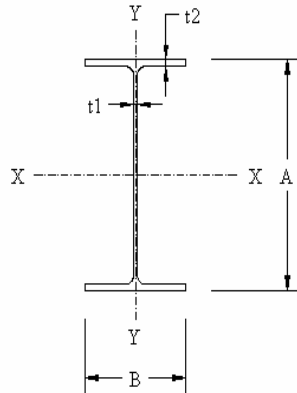
Gambar 5.26 Penomoran Pada Ikatan Angin Atas

Pendimensian batang 16,33,34,35,36,37,38,39,40,41,42,43,44,45,46,47,dan 32 direncanakan menggunakan profil IWF 250x175x7x11-44,1, sedangkan Batang 17,18,19,20,21,22,23,24,25,26,27,28,29,30,dan 31 direncanakan menggunakan batang L.50.50.5



## Pendimensian Batang Ikatan Angin Atas

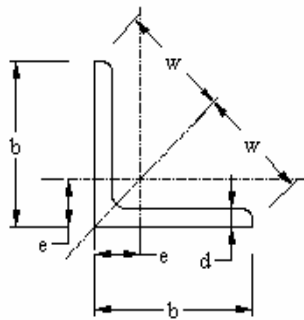
Profil IWF 250x175x7x11-44,1



Profil WF	Berat (kg/m)	Ukuran (mm)				
		A	B	t1	t2	r
250 x 175	44,1	244	175	7	11	16

Luas tampang	Momen Inersia		Jari-jari Inersia		Momen Lawan	
	Ix	Iy	ix	iy	Wx	Wy
56,24	6,120	984	10,4	4,18	502	113

Profil L 80.80.8



Profil L	Ukuran (mm)				F (cm <sup>2</sup> )	Berat (kg/m)	Jarak titik berat (cm)		
	b	d	r	r1			e	w	v
80 x 80 8	80	85	10	5	12,3	9,66	2,26	5,66	3,20

Ix = Iy (cm <sup>4</sup> )	Wx = Wy (cm <sup>3</sup> )	ix = iy (cm)	kx = ky	Iξ (cm <sup>4</sup> )	iξ (cm)	kξ	Iη (cm <sup>4</sup> )	Wη (cm <sup>3</sup> )	iη (cm)	kη
72,3	12,6	2,42	2,09	115	3,06	1,32	29,6	9,25	1,55	5,11

Dari hasil perhitungan program SAP 2000 versi 7.42, diperoleh gaya batang terbesar, dengan resiko tekuk adalah :

Batang	Gaya Batang (P) kg	Panjang (L) m	Keterangan
17 & 47	-3899,840	10,649	tekan
31 & 33	2956,51	10,649	tekan

### Batang tekan

Profil IWF 250x175x7x11-44,1

S = -3899,840 kg ( batang 17 & 47 )

Lk =  $\sqrt{9,403^2 + 5^2} = 10,649$  m

Rumus umum menurut PPBBI hal 9 Bab 4.1 Pasal 1 dan 2, untuk stabilitas batang tekan terhadap bahaya tekuk :

$$\frac{(S \times \omega)}{F} < \sigma \text{ ijin baja}$$

Dimana :

S = gaya tekan pada batang tersebut

F = luas penampang batang

$\omega$  = faktor tekuk yang tergantung dari kelangsingan dan macam bajanya

Menghitung kelangsingan batang tunggal :

$$\lambda = \frac{Lk}{I_{\min}} = \frac{1064,9}{4,18} = 254,760 \text{ cm}$$

$$\lambda_g = \pi \times \sqrt{\frac{E}{0,7 \times \sigma_1}} = 3,14 \times \sqrt{\frac{2,1 \times 10^5}{0,7 \times 240}} = 111,016$$

$$\lambda_s = \frac{\lambda}{\lambda_g} = \frac{254,760}{111,016} = 2,294$$

Untuk  $\lambda_s \geq 1$  :

$$\begin{aligned} \omega &= 2,381 \times \lambda_s^2 \\ &= 2,381 \times (2,294)^2 \\ &= 12,529 \end{aligned}$$

Maka :

$$\frac{(S \times \omega)}{F} = \frac{-3899,84 \times 12,529}{56,24} = 868,796 \text{ kg/cm}^2 < 1867 \text{ kg/cm}^2 \dots \text{OK}$$

### Batang tarik

Profil L 80.80.8

$$S = 2956,51 \text{ kg ( batang 8 \& 9 )}$$

$$F_{nt} = 0,85 \times F_{\text{profil}}$$

$$= 0,85 \times 12,3$$

$$= 10,455 \text{ cm}^2$$

Cek Tegangan :

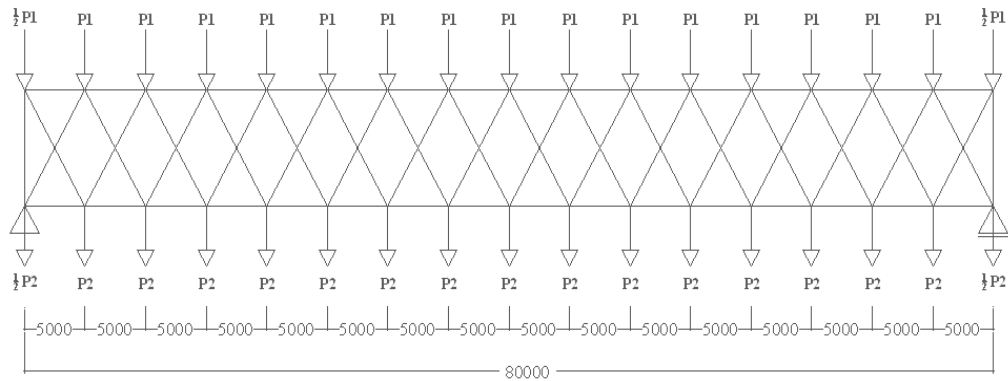
$$\sigma = \frac{S}{F_{nt}} < 1867 \text{ kg/cm}^2$$

$$= \frac{2956,51}{10,455} < 1867 \text{ kg/cm}^2$$

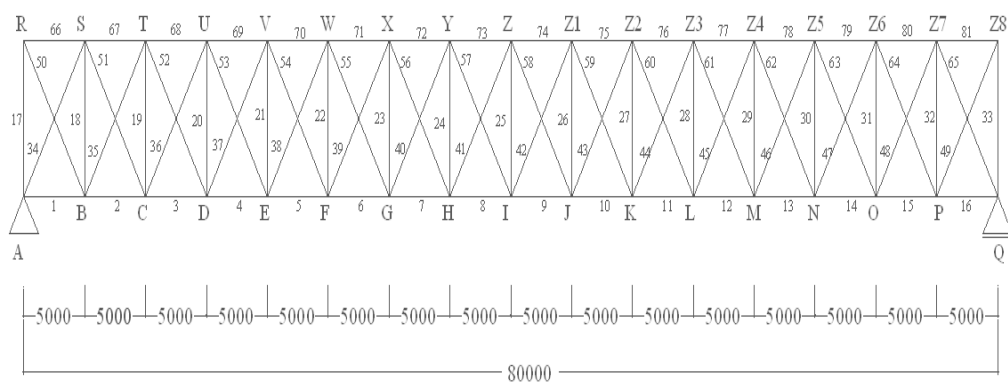
$$= 282,784 \text{ kg/cm}^2 < 1867 \text{ kg/cm}^2 \dots \text{OK}$$

### 5.2.9.3 Pertambahan Angin Bawah

Untuk pertambahan angin bawah digunakan profil L 200x200x16-48,5



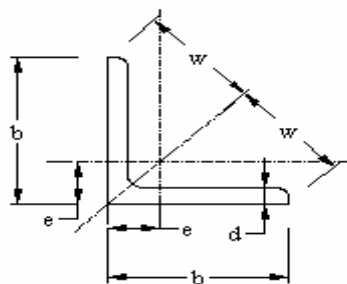
Gambar 5.27 Penyebaran Beban Angin Pada Ikatan Angin Bawah



Gambar 5.28 Penomoran Pada Ikatan Angin Bawah

### Pendimensian Batang

- Profil. L 200x200x16-48,5



Profil L	Ukuran (mm)				F (cm <sup>2</sup> )	Berat (kg/m)	Jarak titik berat (cm)		
	b	d	r	r1			e	w	v
200 x 200 x 16	200	16	18	9	61,8	48,5	5,52	14,1	7,80

I <sub>x</sub> = I <sub>y</sub> (cm <sup>4</sup> )	W <sub>x</sub> = W <sub>y</sub> (cm <sup>3</sup> )	i <sub>x</sub> = i <sub>y</sub> (cm)	k <sub>x</sub> = k <sub>y</sub>	I <sub>ξ</sub> (cm <sup>4</sup> )	i <sub>ξ</sub> (cm)	k <sub>ξ</sub>	I <sub>η</sub> (cm <sup>4</sup> )	W <sub>η</sub> (cm <sup>3</sup> )	i <sub>η</sub> (cm)	k <sub>η</sub>
2340	162	6,15	1,65	3740	7,78	1,03	943	121	3,91	4,05

Dari hasil perhitungan program SAP 2000 versi 7.42, diperoleh gaya batang terbesar, dengan resiko tekuk adalah :

Batang	Gaya Batang (P) kg	Panjang (L) m	Keterangan
34 & 65	-7757,50	10,649	tekan

### Batang tekan

- Profil L 200x200x16-48,5 kg/m  
S = -7757,50 kg ( batang 34 & 65 )

$$L_k = \sqrt{9,403^2 + 5^2} = 10,649 \text{ m}$$

Rumus umum menurut PPBBI hal 9 Bab 4.1 Pasal 1 dan 2, untuk stabilitas batang tekan terhadap bahaya tekuk :

$$\frac{(S \times \omega)}{F} < \sigma \text{ ijin baja}$$

Dimana :

S = gaya tekan pada batang tersebut

F = luas penampang batang

$\omega$  = faktor tekuk yang tergantung dari kelangsingan dan macam bajanya

Menghitung kelangsingan batang tunggal :

$$\lambda = \frac{L_k}{I_{\min}} = \frac{1064,9}{3,91} = 272,352 \text{ cm}$$

$$\lambda_g = \pi \times \sqrt{\frac{E}{0,7 \times \sigma_1}} = 3,14 \times \sqrt{\frac{2,1 \times 10^5}{0,7 \times 240}} = 111,016$$

$$\lambda_s = \frac{\lambda}{\lambda_g} = \frac{272,352}{111,016} = 2,453$$

Untuk  $\lambda_s \geq 1$  :

$$\begin{aligned}\omega &= 2,381 \times \lambda_s^2 \\ &= 2,381 \times (2,453)^2 \\ &= 14,326\end{aligned}$$

Maka :

$$\frac{(S \times \omega)}{F} = \frac{7757,50 \times 14,326}{61,8} = 1798,283 \text{ kg/cm}^2 < 1867 \text{ kg/cm}^2 \dots \text{OK}$$

### 5.2.10 Perencanaan Sambungan Pertambahan Angin

Sambungan pertambahan angin direncanakan menggunakan pelat 10 mm, dengan alat penyambung baut Ø 5/8" ( 15,9 mm )

Pengaturan jarak antar baut ( berdasar PPBBI hal 70 ) :

$2,5 d \leq s \leq 7 d$  , atau  $14 t \leq s$  = jarak antar sumbu baut pada arah horizontal

$2,5 d \leq u \leq 7 d$  , atau  $14 t \leq u$  = jarak antar sumbu baut pada arah vertikal

$1,5 d \leq s_1 \leq 3 d$  , atau  $6 t \leq s_1$  = jarak sumbu baut paling luar dengan bagian yang disambung

Jarak antar sumbu baut pada arah horizontal ( s ) :

$2,5 d \leq s \leq 7 d$

$39,75 \leq s \leq 111,3$  diambil 40 mm

Jarak antar sumbu baut pada arah vertikal ( u ) :

$2,5 d \leq u \leq 7 d$

$39,75 \leq s \leq 111,3$  diambil 90 mm

Jarak sumbu baut paling luar dengan bagian yang disambung (  $s_1$  ) :

$1,5 d \leq s_1 \leq 3 d$

$23,85 \leq s_1 \leq 47,7$  diambil 40 mm

#### a. Pertambahan angin atas

1. Profil IWF 250x175x7x11-44,1 dengan rangka induk IWF 406x403x16x24-200

Data teknis perencanaan baut :

- Tebal plat penyambung (  $\delta$  ) = 10 mm
- Diameter baut ( Ø ) = 5/8" ( 15,9 mm )
- Pmaks = -3899,840 kg ( batang 17 & 47 )

Perhitungan gaya yang bekerja pada sambungan :

$$\frac{\delta}{d} = \frac{10}{15,9} = 0,628 > 0,314 \text{ (pengaruh geser)}$$

$$n_{ds} = \frac{P}{2 \times \sigma \times \delta \times d} = \frac{4080,11}{2 \times 1867 \times 1 \times 1,59} = 0,687 \sim \text{diambil 4 baut}$$

Tegangan yang terjadi pada baut :

$$\begin{aligned} \sigma_{BAUT} &= \frac{P}{2 \times n_{BAUT} \times \left( \frac{1}{4} \times \pi \times d^2 \right)} < 0,6 \times \bar{\sigma} \\ &= \frac{3899,840}{2 \times 4 \times \left( \frac{1}{4} \times 3,14 \times 1,59^2 \right)} < 1120,2 \text{ kg/cm}^2 \\ &= 245,590 \text{ kg/cm}^2 < 1120,2 \text{ kg/cm}^2 \end{aligned}$$

## 2. Profil L 80.80.8 dengan rangka induk IWF 406x403x16x24-200

Data teknis perencanaan baut :

- Tebal plat penyambung (  $\delta$  ) = 10 mm
- Diameter baut (  $\varnothing$  ) = 5/8" ( 15,9 mm )
- Pmaks = 2956,51 kg ( batang 31 & 33 )

Perhitungan gaya yang bekerja pada sambungan :

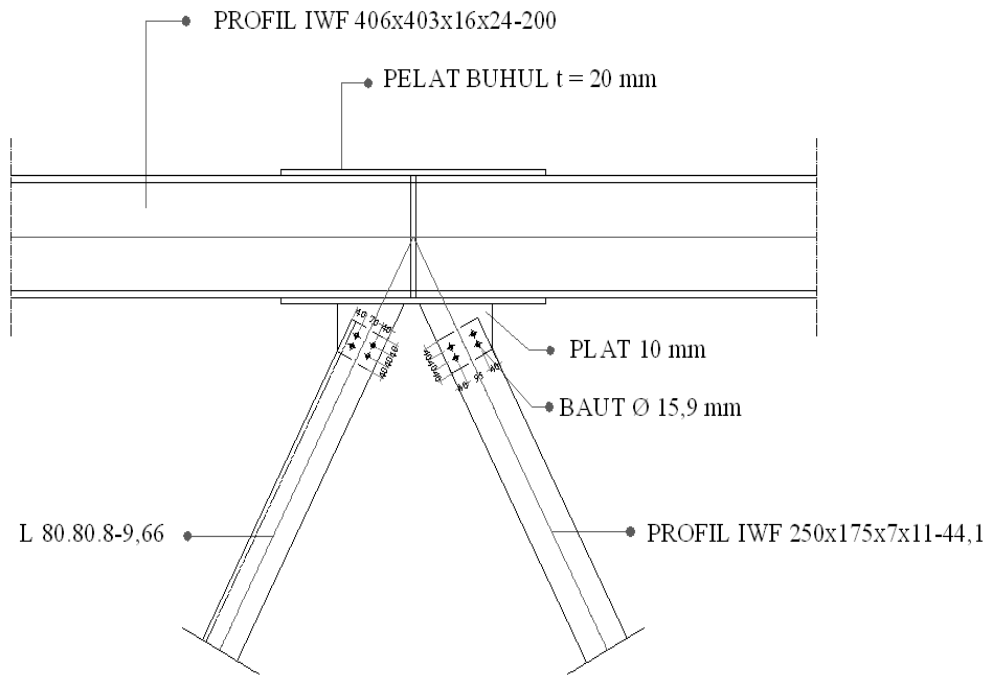
$$\frac{\delta}{d} = \frac{10}{15,9} = 0,628 > 0,314 \text{ (pengaruh geser)}$$

$$n_{ds} = \frac{P}{2 \times \sigma \times \delta \times d} = \frac{2956,51}{2 \times 1867 \times 1 \times 1,59} = 0,497 \sim \text{diambil 2 baut}$$

Tegangan yang terjadi pada baut :

$$\begin{aligned} \sigma_{BAUT} &= \frac{P}{2 \times n_{BAUT} \times \left( \frac{1}{4} \times \pi \times d^2 \right)} < 0,6 \times \bar{\sigma} \\ &= \frac{2956,51}{2 \times 2 \times \left( \frac{1}{4} \times 3,14 \times 1,59^2 \right)} < 1120,2 \text{ kg/cm}^2 \\ &= 372,450 \text{ kg/cm}^2 < 1120,2 \text{ kg/cm}^2 \end{aligned}$$





Gambar 5.29 Sambungan Ikatan Angin Atas

**b. Pertambahan angin bawah**

Menggunakan profil L 200x200x16-48,5 kg/m dengan rangka induk profil IWF 406x403x16x24-200

Data teknis perencanaan baut :

1. Tebal plat penyambung (  $\delta$  ) = 10 mm
2. Diameter baut (  $\varnothing$  ) = 5/8" ( 15,9 mm )
3. Pmaks = -7757,50 kg ( batang 34& 65 )

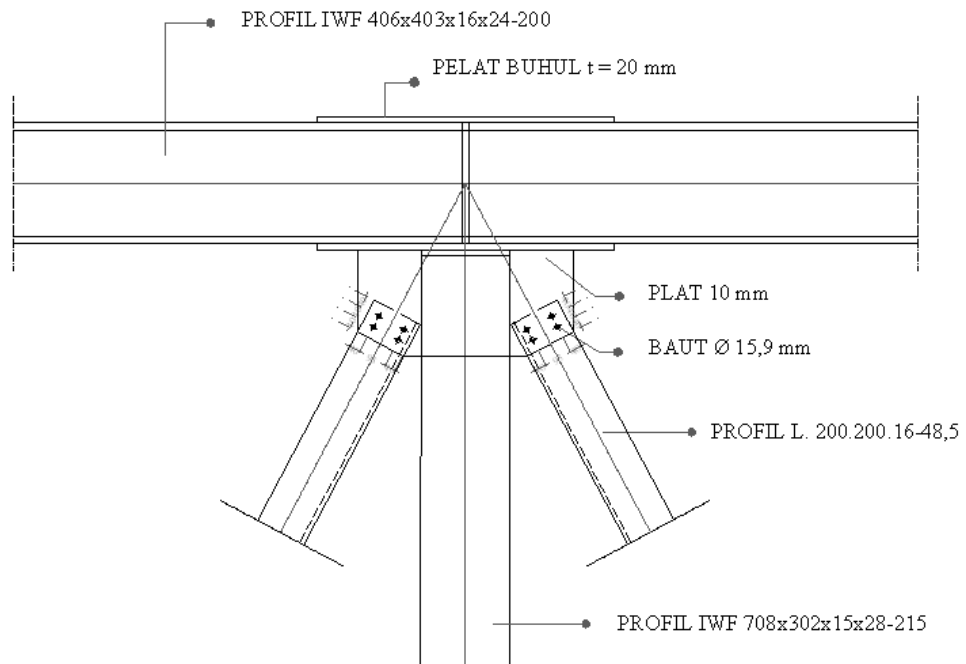
Perhitungan gaya yang bekerja pada sambungan :

$$\frac{\delta}{d} = \frac{10}{15,9} = 0,628 > 0,314 \text{ (pengaruh geser)}$$

$$n_{ds} = \frac{P}{2 \times \bar{\sigma} \times \delta \times d} = \frac{-7757,50}{2 \times 1867 \times 1 \times 1,59} = 1,306 \sim \text{diambil 4 baut}$$

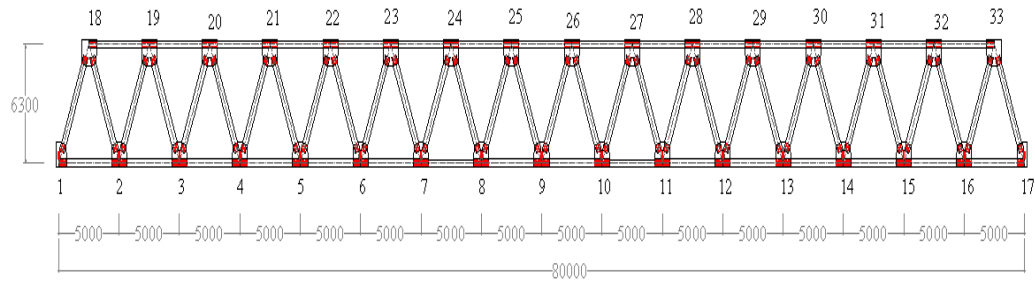
Tegangan yang terjadi pada baut :

$$\begin{aligned} \sigma_{BAUT} &= \frac{P}{2 \times n_{BAUT} \times \left( \frac{1}{4} \times \pi \times d^2 \right)} < 0,6 \times \bar{\sigma} \\ &= \frac{-7757,50}{2 \times 4 \times \left( \frac{1}{4} \times 3,14 \times 1,59^2 \right)} < 1120,2 \text{ kg/cm}^2 \\ &= 488,630 \text{ kg/cm}^2 < 1120,2 \text{ kg/cm}^2 \end{aligned}$$



Gambar 5.30 Sambungan Ikatan Angin Bawah

### 5.2.11 Perencanaan Rangka Induk



Gambar 5.31 Struktur Rangka Induk

#### a. Data Perencanaan

Tebal pelat	= 20 cm
Tebal perkerasan	= 5 cm
Genangan air	= 5 cm
Lebar lantai kendaraan	= 700 cm
Lebar trotoir	= 100 cm
Tebal trotoir	= 25 cm
G memanjang	= IWF 350.175.7.11-49,6 berat = 49,6 kg/m
G melintang	= IWF 708.302.15.28-215 berat = 213 kg/m
Ikatan angin atas	= IWF 250.175.7.11-44,1 berat = 44,1 kg/m = L 80.80.8-9,6 berat = 9,6 kg/m
Ikatan angin bawah	= L 200.200.16-48,5 berat = 48,5 kg/m
Sandaran	= Pipa D 76,3 dengan berat = 7,13 kg/m
Rangka Utama	= IWF 428.407.20.35-283 berat = 283 kg/m

#### b. Pembebanan

Asumsi beban antara rangka induk ditahan masing-masing  $\frac{1}{2}$  nya oleh rangka induk. Dimensi Rangka diasumsikan sama untuk semua rangka.

##### 1. Beban rangka induk

- Buhul 1 dan 17
 
$$= (1/2 \text{ diagonal}) + (1/2 \text{ horisontal})$$

$$= (1/2 \times 6,78 \times 283) + (1/2 \times 5 \times 283)$$

$$= 1366,48 \text{ kg}$$

- Buhul 18 dan 33  $= (1/2 \times 2 \text{ diagonal}) + (1/2 \times \text{horizontal})$   
 $= (1/2 \times 2 \times 6,78 \times 283) + (1/2 \times 5 \times 283)$   
 $= 2152,96 \text{ kg}$
- Buhul 2-16 dan 19-32  $= (1/2 \times 2 \text{ diagonal}) + (1/2 \times 2 \text{ horizontal})$   
 $= (1/2 \times 2 \times 6,78 \times 283) + (1/2 \times 2 \times 5 \times 283)$   
 $= 2732,96 \text{ kg}$

Penambahan beban sebesar 10 %, sebagai asumsi berat pelat buhul beserta bautnya.

- Buhul 1 dan 17  $= 110\% \times 1366,48 \text{ kg} = 1503,128 \text{ kg}$
- Buhul 18 dan 33  $= 110\% \times 2152,96 \text{ kg} = 2368,256 \text{ kg}$
- Buhul 2-16 dan 19-32  $= 110\% \times 2732,96 \text{ kg} = 3006,256 \text{ kg}$

## 2. Beban gelagar memanjang

5 buah profil IWF 350.175.7.11-49,6 kg/m

Distribusi beban pada tiap buhul :

- Buhul 1 dan 17  $= \left( \frac{5 \times 49,6}{2} \right) \times \frac{1}{2} = 62 \text{ kg}$
- Buhul 2-16  $= \left( \frac{5 \times 49,6}{2} \right) \times 2 \times \frac{1}{2} = 124 \text{ kg}$

## 3. Beban gelagar melintang

17 buah profil IWF 708.302.15.28-215

Distribusi beban pada tiap buhul :

- Buhul 1-17  $= \left( \frac{9 \times 215}{2} \right) = 958,5 \text{ kg}$

## 4. Beban pelat beton (termasuk dibawah trotoar)

Tebal pelat beton = 20 cm

Distribusi beban pada tiap buhul :

- Buhul 1 dan 17  $= \left( \frac{9 \times 5 \times 0,2 \times 25}{2} \right) \times \frac{1}{2} = 56,25 \text{ kg}$

- Buhul 2-16 =  $\left( \frac{9 \times 5 \times 0,2 \times 25}{2} \right) = 112,5 \text{ kg}$

#### 5. Beban lapis perkerasan

Tebal lapis perkerasan = 5 cm

Distribusi beban pada tiap buhul :

- Buhul 1 dan 17 =  $\left( \frac{7 \times 5 \times 0,05 \times 22}{2} \right) \times \frac{1}{2} = 9,625 \text{ kg}$
- Buhul 2-16 =  $\left( \frac{7 \times 5 \times 0,05 \times 22}{2} \right) = 19,25 \text{ kg}$

#### 6. Beban trotoar

Tebal trotoar = 25 cm

Lebar trotoar = 1,0 m

Distribusi beban pada tiap buhul :

- Buhul 1 dan 17 =  $(5 \times 1 \times 0,25 \times 25) \times \frac{1}{2} = 15,625 \text{ kg}$
- Buhul 2-16 =  $(5 \times 1 \times 0,25 \times 25) = 31,250 \text{ kg}$

#### 7. Beban air hujan

Tebal genangan = 5 cm

Distribusi beban pada tiap buhul :

- Buhul 1 dan 17 =  $\left( \frac{9 \times 5 \times 0,05 \times 10}{2} \right) \times \frac{1}{2} = 5,625 \text{ kg}$
- Buhul 2-16 =  $\left( \frac{9 \times 5 \times 0,05 \times 10}{2} \right) = 11,25 \text{ kg}$

#### 8. Beban pipa sandaran

Pipa sandaran Ø 76,3 mm, dengan berat 7,13 kg/m

Berat total pipa sandaran =  $2 \times 80 \times 7,13 = 1140,8 \text{ kg}$

Distribusi beban pada tiap buhul :

- Buhul 1 dan 17 =  $\frac{1140,8}{16} \times \frac{1}{2} = 35,65 \text{ kg}$
- Buhul 2-16 =  $\frac{1140,8}{16} = 71,3 \text{ kg}$

#### 9. Beban ikatan angin atas

Profil IWF 250.175.7.11 – 44,1 berat = 44,1 kg/m

Profil L 80.80.8-9,6 berat = 9,6 kg/m

Distribusi beban pada tiap buhul :

- Buhul 18 dan 33 =  $((\frac{1}{2} \times 44,1 \times 10,29) + (\frac{1}{2} \times 9,6 \times 10,29)) \times \frac{1}{2}$   
 $= (226,894 + 49,392) \times \frac{1}{2}$   
 $= 138,143 \text{ kg}$
- Buhul 19-32 =  $((\frac{1}{2} \times 44,1 \times 10,29) + (\frac{1}{2} \times 9,6 \times 10,29)) \times \frac{1}{2} \times 2$   
 $= (226,894 + 49,392) \times \frac{1}{2} \times 2$   
 $= 276,286 \text{ kg}$

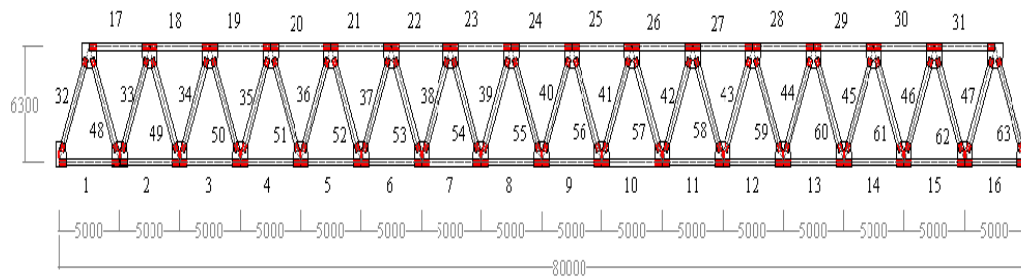
#### 10. Beban ikatan angin bawah

Profil L 200.200.16-48,5 berat = 48,5kg/m

Distribusi beban pada tiap buhul :

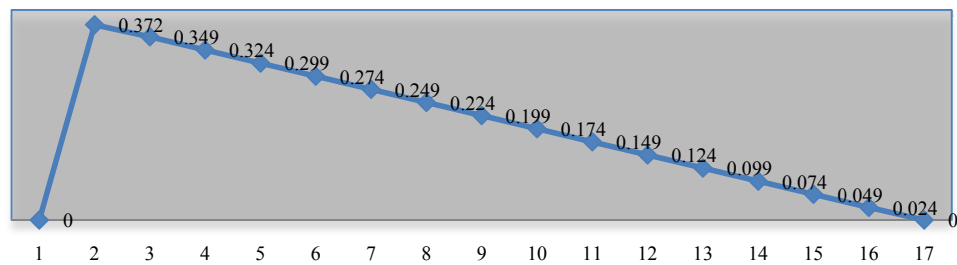
- Buhul 1 dan 17 =  $(2 \times 48,5 \times 10,29) \times \frac{1}{2} \times \frac{1}{2}$   
 $= 249,532 \text{ kg}$
- Buhul 2-16 =  $(2 \times 48,5 \times 10,29) \times \frac{1}{2} \times \frac{1}{2} \times 2$   
 $= 499,065 \text{ kg}$

## c. Garis Pengaruh

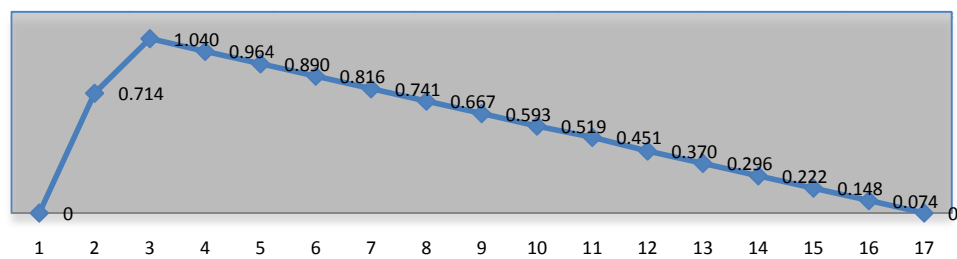


Gambar 5.32 Rangka Utama

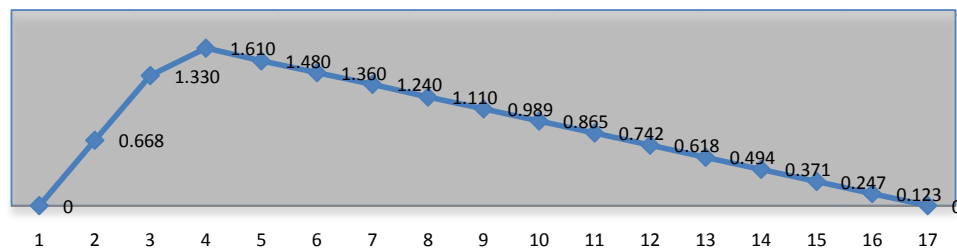
**S1=S16**



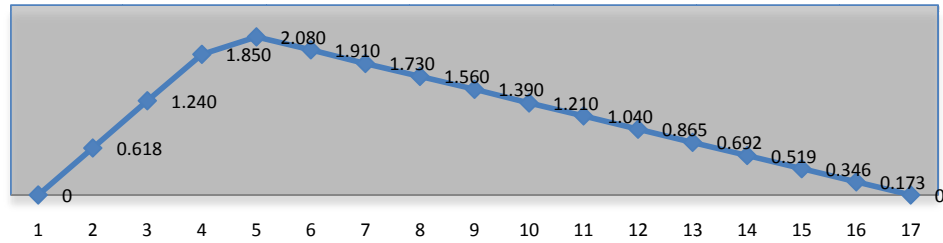
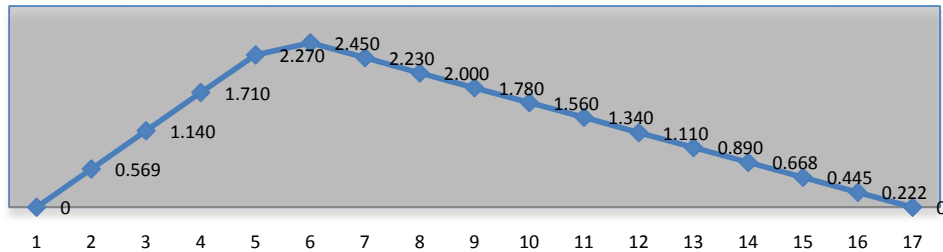
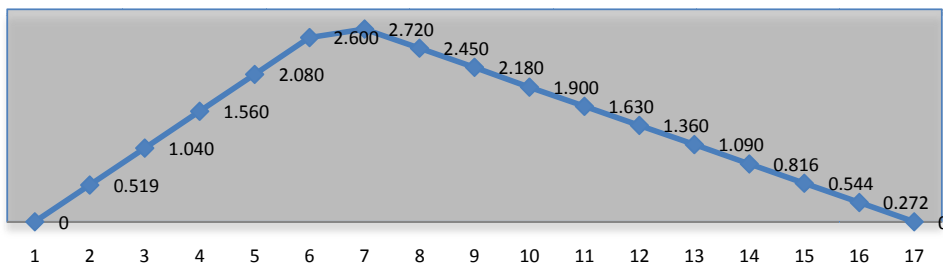
**S2=S15**

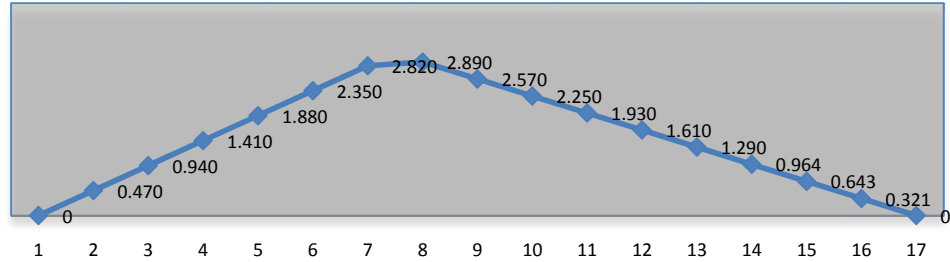
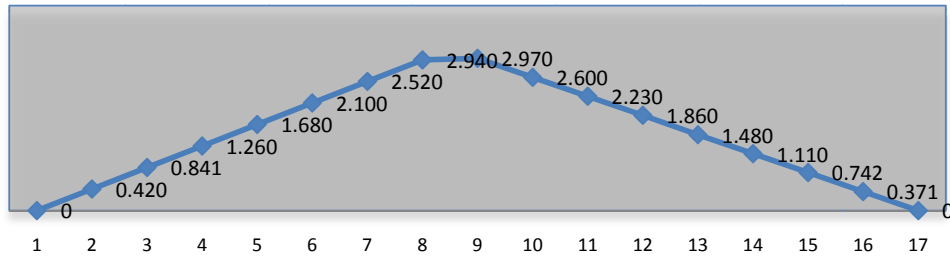
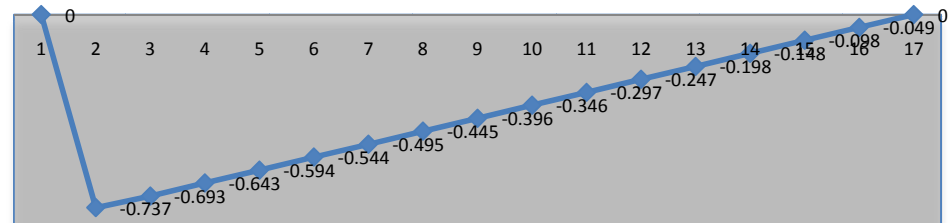
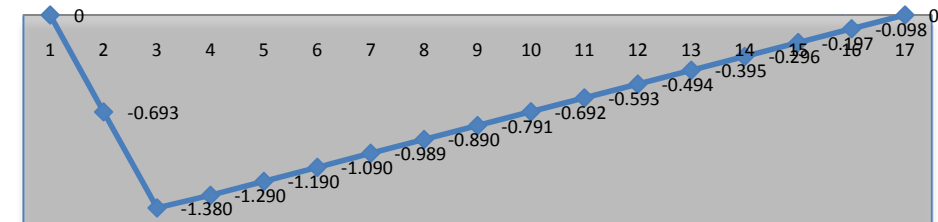


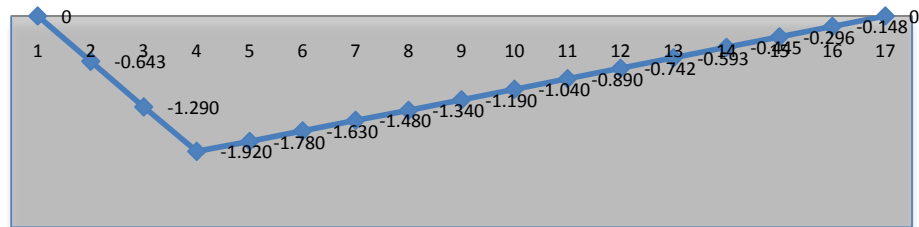
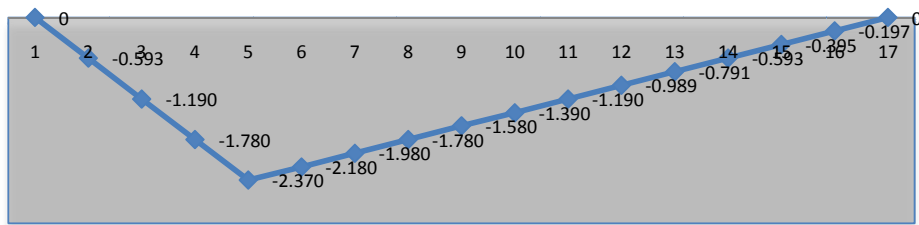
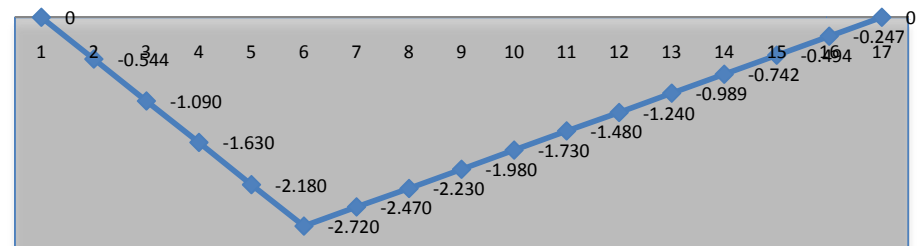
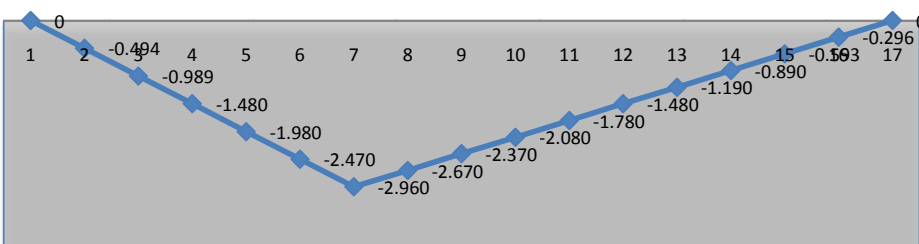
**S3=S14**

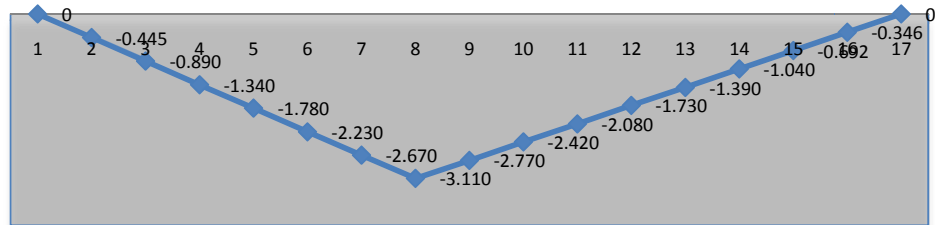
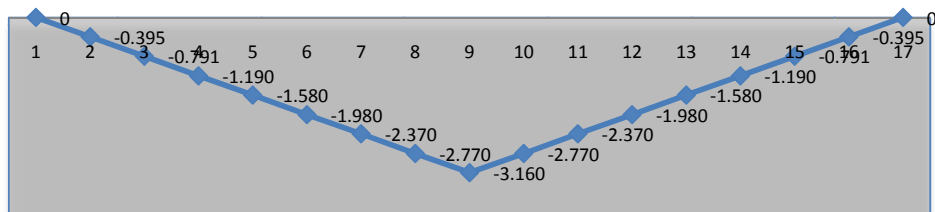
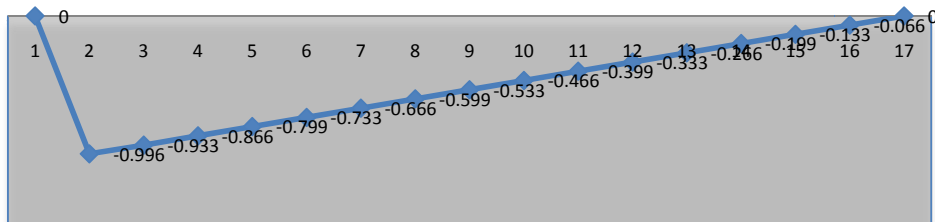
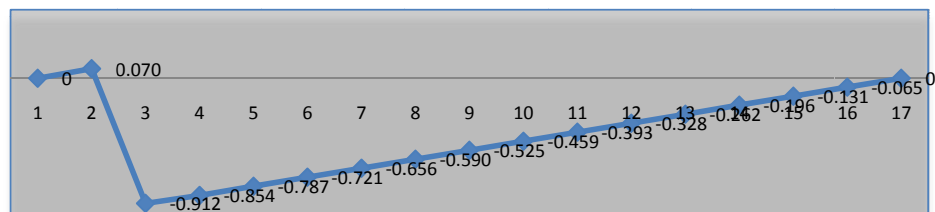


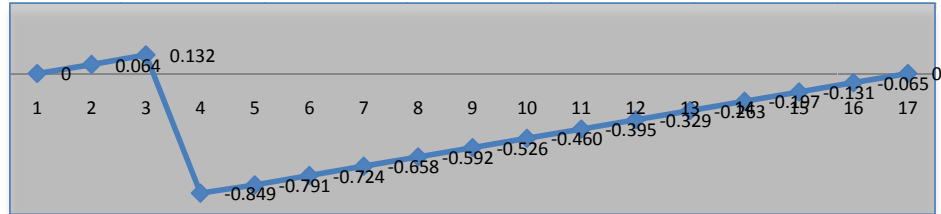
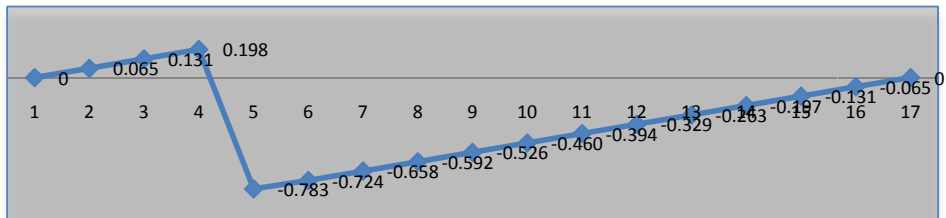
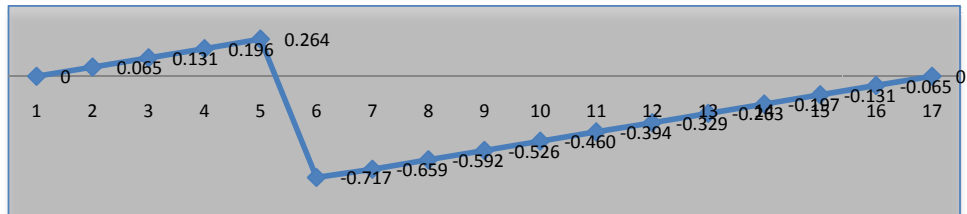
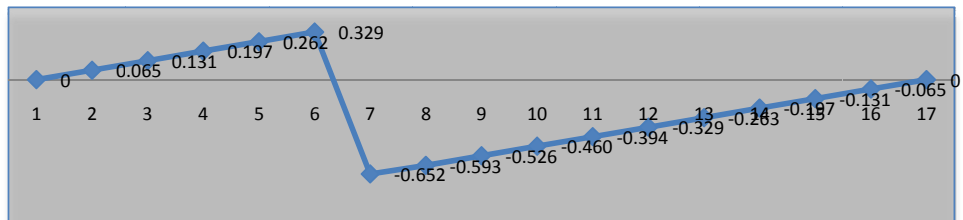


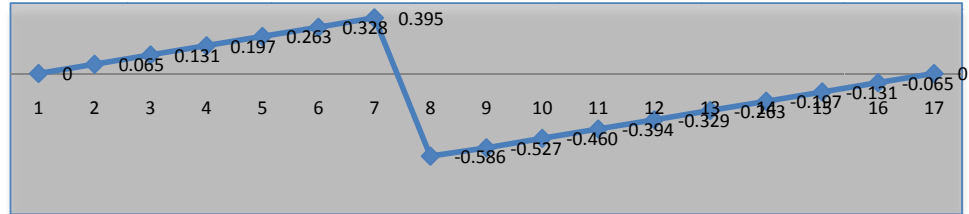
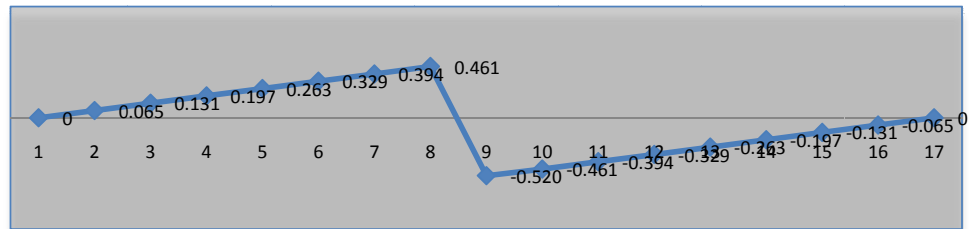
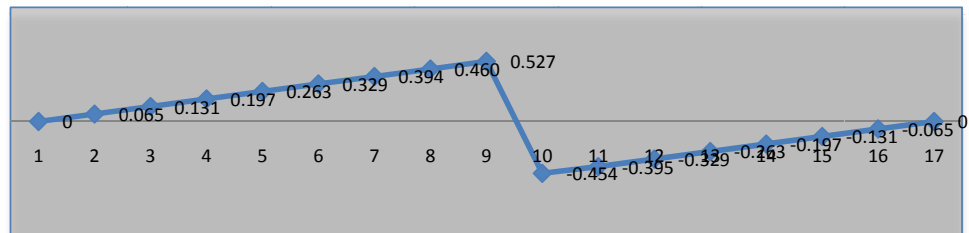
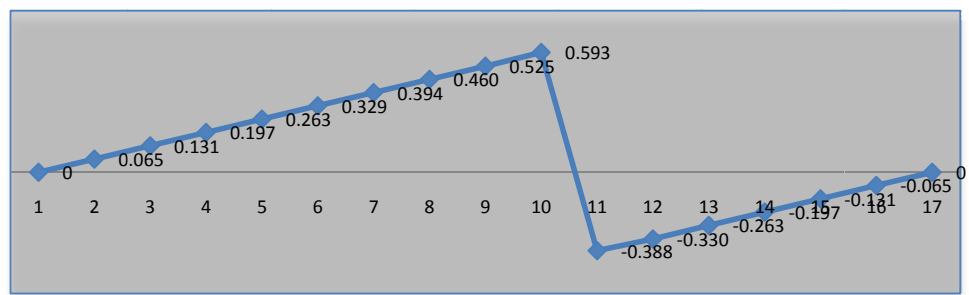
**S4=S13****S5=S12****S6=S11**

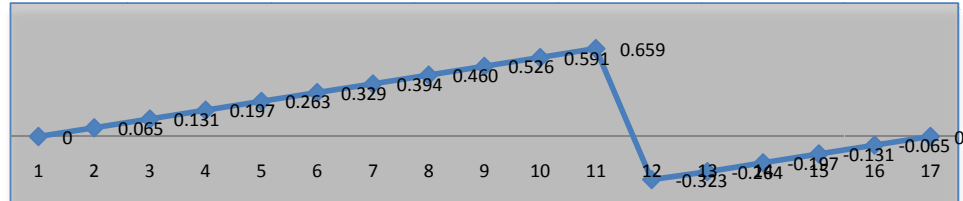
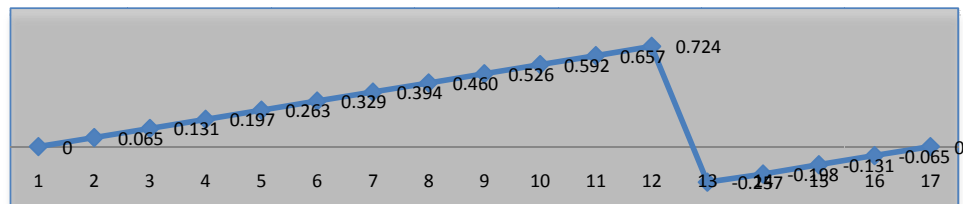
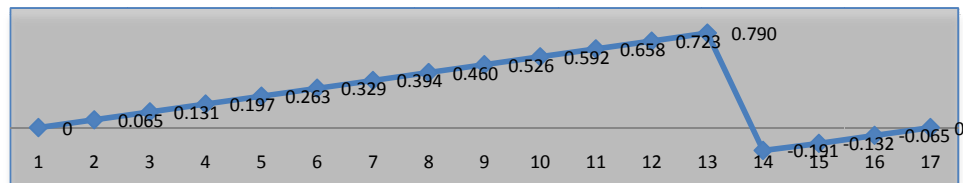
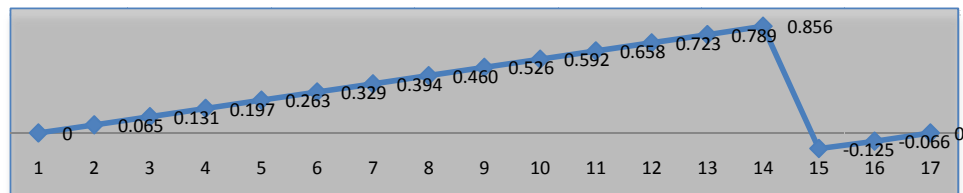
**S7=S10****S8=S9****S17=S31****S18=S30**

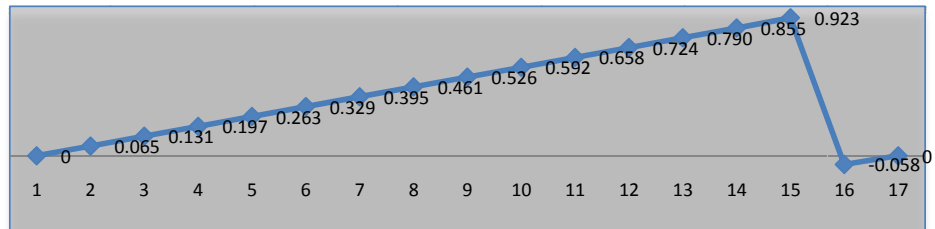
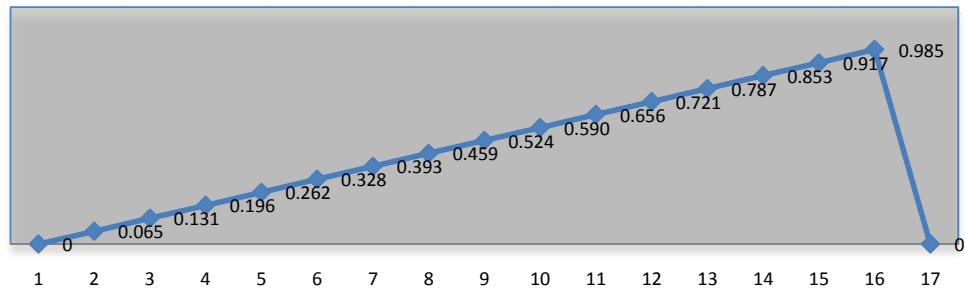
**S19=S29****S20=S28****S21=S27****S22=S26**

**S23=S25****S24****S32=S63****S33=S62**

**S34=S61****S35=S60****S36=S59****S37=S58**

**S38=S57****S39=S56****S40=S55****S41=S54**

**S42=S53****S43=S52****S44=S51****S45=S50**

**S46=S49****S47=S48**



#### d. Perhitungan gaya batang akibat beban dinamis

- Beban terbagi rata (“q”)

Bentang jembatan = 80 m, maka :

$$\begin{aligned}q &= 1.1 (1 + 30/L) \text{ t/m}' \quad \text{untuk } L > 60 \text{ m} \\&= 1.1 (1 + 30/80) \text{ t/m}' \\&= 1,512 \text{ t/m}\end{aligned}$$

- Beban terbagi rata sepanjang gelagar melintang untuk lebar 5,5 m

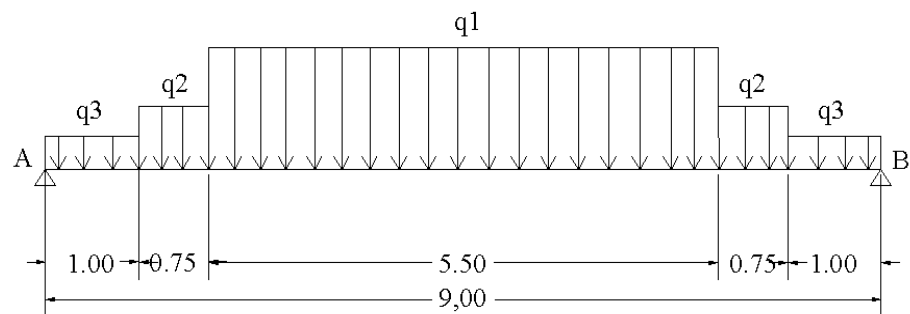
$$q_1 = \frac{q \times 5,5}{2,75} = \frac{1,512 \times 5,5}{2,75} = 3,024 \text{ t/m} = 3024 \text{ kg/m}$$

- Beban terbagi rata untuk lebar sisanya ( 0,75 m )

$$q_2 = 50 \% \times \left( \frac{q}{2,75} \right) \times 0,75 = 0,206 \text{ t/m} = 206 \text{ kg/m}$$

- Beban terbagi rata pada trotoar

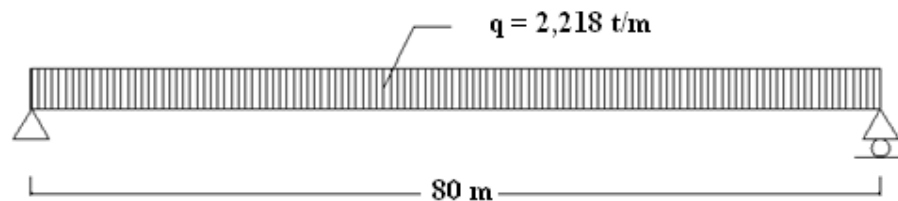
$$q_3 = 1,00 \times 500 \text{ kg/m} = 500 \text{ kg/m}$$



$$\begin{aligned}q_{\text{tot}} &= 3024 + (2 \times 206) + (2 \times 500) \text{ kg/m} \\&= 4436 \text{ kg/m}\end{aligned}$$

Beban q yang diterima satu sisi rangka :

$$q = \frac{4436}{2} = 2218 \text{ kg/m} = 2,218 \text{ t/m}$$



Gambar 5.33 Beban “q” Yang Bekerja Pada Satu Rangka

- Beban “P”

$$P = 12 \text{ ton}$$

$$\text{Koefesien kejut (K)} = 1 + \left( \frac{20}{(50 + L)} \right)$$

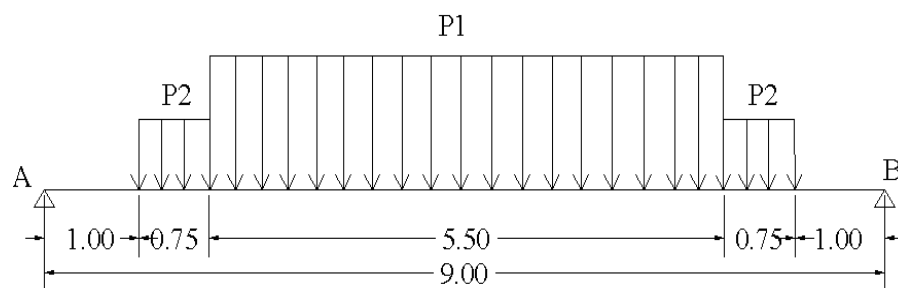
$$K = 1 + \left( \frac{20}{(50 + 80)} \right) = 1,153$$

- Beban P bekerjasepanjang gelagar melintang untuk lebar 5,5 m

$$P_1 = \frac{P}{2,75} \times K \times 5,5 = \frac{12}{2,75} \times 1,153 \times 5,5 = 27,672 \text{ t}$$

- Beban P untuk lebar sisanya ( 50% dari P1 )

$$P_2 = 50 \% \times \frac{P}{2,75} \times K \times 0,75 = 50\% \times \frac{12}{2,75} \times 1,153 \times 0,75 = 1,886 \text{ t}$$



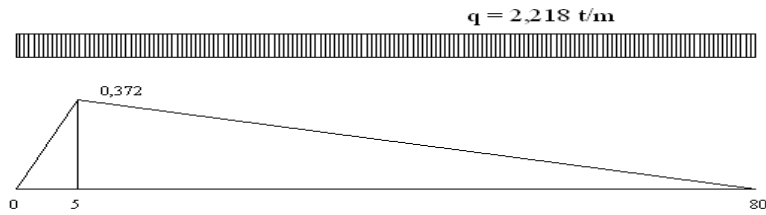
Gambar 5.34 Penyebaran Beban ” P ”

$$P_{\text{tot}} = 27,672 + (2 \times 1,886) = 31,444 \text{ ton}$$

Beban yang diterima atu sisi rangka :

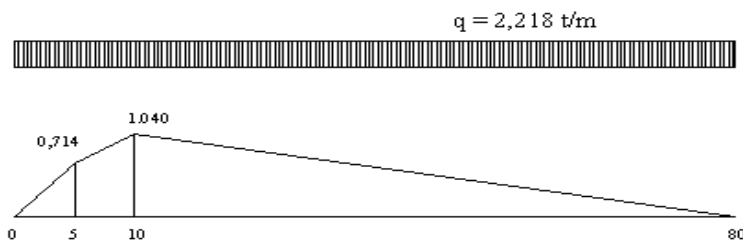
$$P = \frac{31,444}{2} = 15,722 \text{ ton}$$

- **S1 = S16**



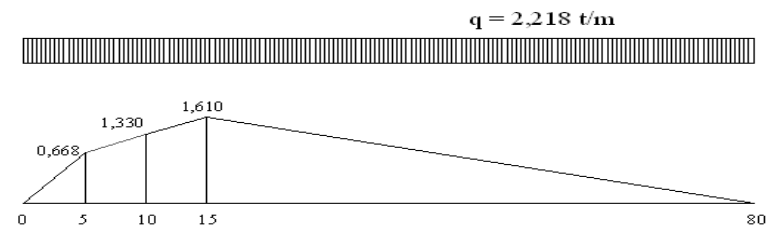
$$S = (0,5 \times 0,372 \times 80) \times 2,218 + (0,372 \times 15,722) = 38,851 \text{ t}$$

- **S2 = S15**



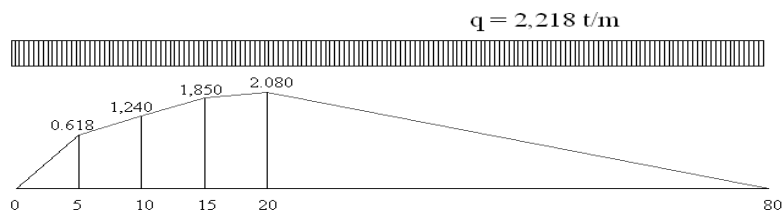
$$S = ((0,5 \times 5 \times 0,714) + (\frac{0,714 + 1,040}{2} \times 5) + (0,5 \times 70 \times 1,040)) \times 2,218 + (1,040 \times 15,722) = 110,770 \text{ t}$$

- **S3 = S14**



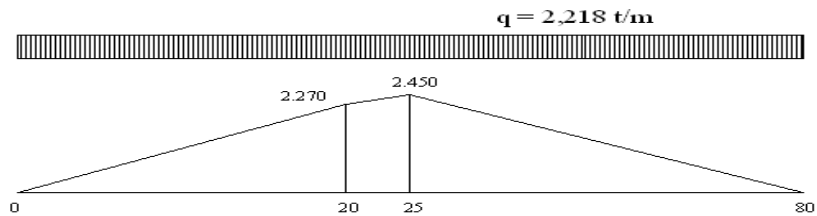
$$S = ((0,5 \times 5 \times 0,668) + (\frac{0,668 + 1,330}{2} \times 5) + (\frac{1,610 + 1,330}{2} \times 5) + (0,5 \times 65 \times 1,610)) \times 2,218 + (1,610 \times 15,722) = 172,454 \text{ t}$$

- **S4 = S13**



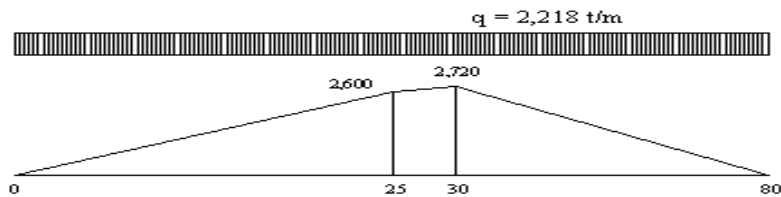
$$S = ((0,5 \times 5 \times 0,618) + (\frac{0,618+1,240}{2} \times 5) + (\frac{1,850+1,240}{2} \times 5) + (\frac{1,850+2,080}{2} \times 5) + (0,5 \times 60 \times 2,080)) \times 2,218 + (2,080 \times 15,722) = 234,017 \text{ t}$$

- **S5 = S12**



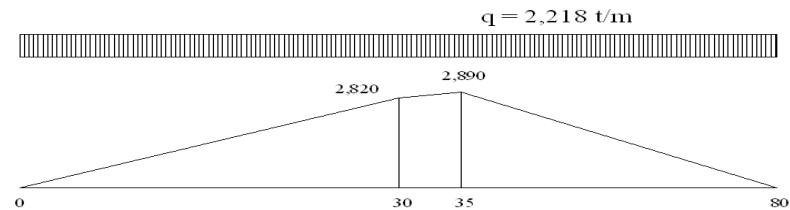
$$S = ((0,5 \times 20 \times 2,270) + (\frac{2,270+2,450}{2} \times 5) + (0,5 \times 25 \times 2,450)) \times 2,218 + (2,450 \times 15,722) = 182,965 \text{ t}$$

- **S6 = S11**



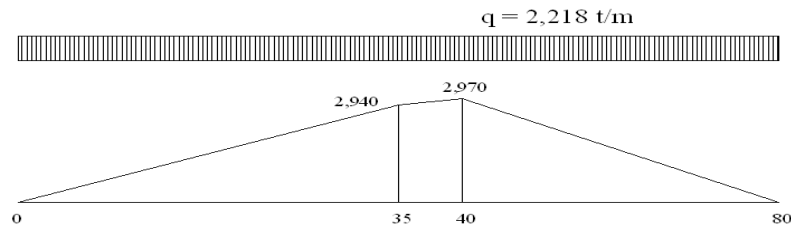
$$S = ((0,5 \times 25 \times 2,600) + (\frac{2,600+2,720}{2} \times 5) + (0,5 \times 30 \times 2,720)) \times 2,218 + (2,720 \times 15,722) = 234,841 \text{ t}$$

- **S7 = S10**



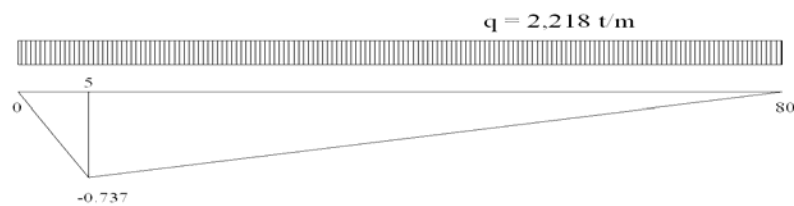
$$S = ((0,5 \times 30 \times 2,820) + (\frac{2,820+2,890}{2} \times 5) + (0,5 \times 45 \times 2,890)) \times 2,218 + (2,890 \times 15,722) = 315,144 \text{ t}$$

- **S8 = S9**



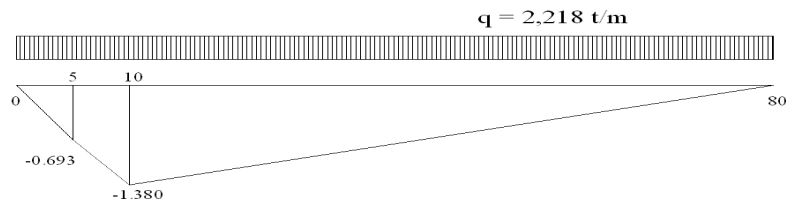
$$S = ((0,5 \times 35 \times 2,940) + (\frac{2,940 + 2,970}{2} \times 5) + (0,5 \times 40 \times 2,970)) \times 2,218 + (2,970 \times 15,722) = 325,330 \text{ t}$$

- **S17 = S31**



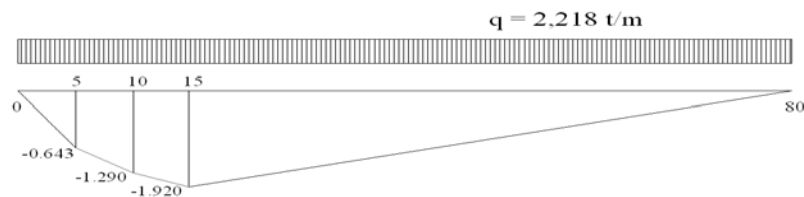
$$S = (0,5 \times -0,737 \times 80) \times 2,218 + (-0,737 \times 15,722) = -76,973 \text{ t}$$

- **S18 = S30**



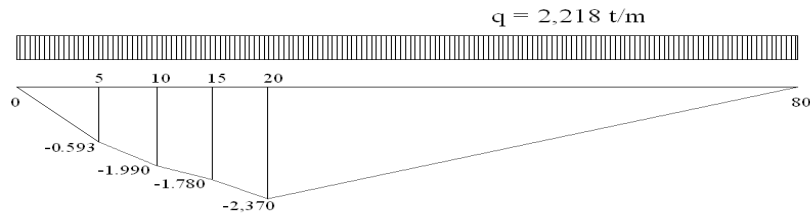
$$S = ((0,5 \times 5 \times -0,693) + (\frac{-0,693 + -1,380}{2} \times 5) + (0,5 \times 70 \times -1,380)) \times 2,218 + (-1,380 \times 15,722) = -144,160 \text{ t}$$

- **S19 = S29**



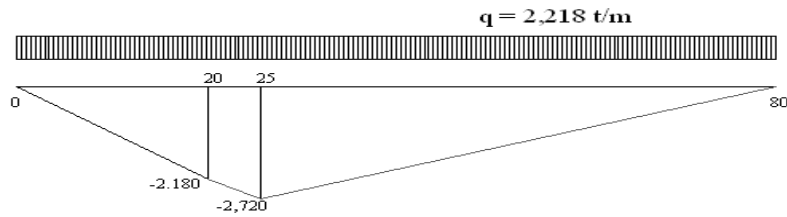
$$S = ((0,5 \times 5 \times -0,643) + (\frac{-0,643 + -1,290}{2} \times 5) + (\frac{-1,290 + -1,920}{2} \times 5) + (0,5 \times 65 \times -1,920)) \times 2,218 + (-1,920 \times 15,722) = -200,670 \text{ t}$$

- **S20 = S28**



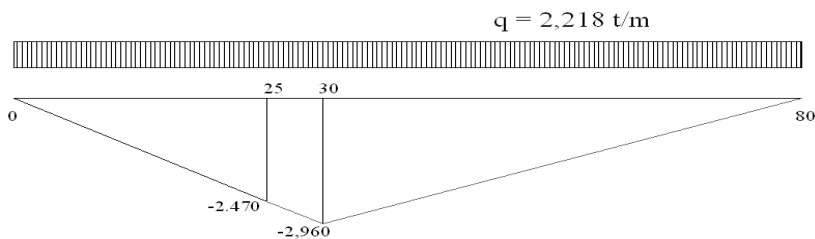
$$S = ((0,5 \times 5 \times -0.593) + (\frac{-0.593 + -1.990}{2} \times 5) + (\frac{-1.990 + -1.780}{2} \times 5) + (\frac{1.780 + 2.370}{2} \times 5) + (0,5 \times 60 \times 2,370)) \times 2,218 + (2,370 \times 15,722) = -256,485 \text{ t}$$

- **S21 = S27**



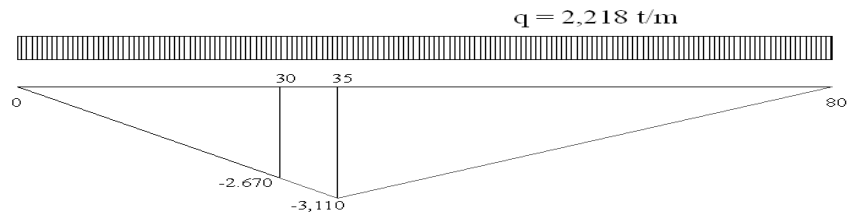
$$S = ((0,5 \times 20 \times -2,180) + (\frac{-2,180 + -2,720}{2} \times 5) + (0,5 \times 55 \times -2,720)) \times 2,218 + (-2,720 \times 15,722) = -284,192 \text{ t}$$

- **S22 = S26**



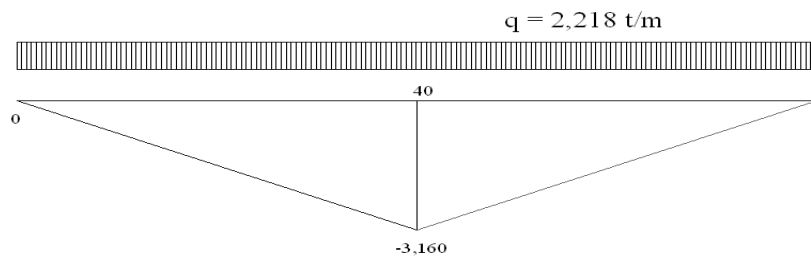
$$S = ((0,5 \times 25 \times -2,470) + (\frac{-2,470 + -2,960}{2} \times 5) + (0,5 \times 50 \times -2,960)) \times 2,218 + (-2,960 \times 15,722) = -309,259 \text{ t}$$

- **S23 = S25**



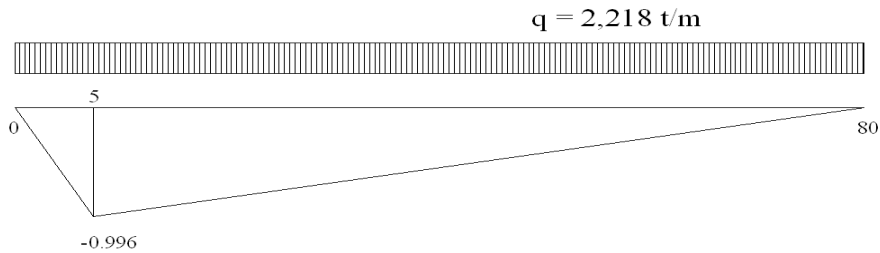
$$S = ((0,5 \times 30 \times -2,670) + (\frac{-2,670 + -3,110}{2} \times 5) + (0,5 \times 45 \times -3,110)) \times 2,218 + (-3,110 \times 15,722) = -324,980 \text{ t}$$

• **S24**



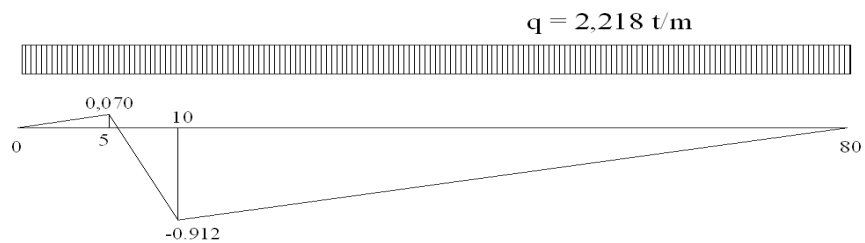
$$S = (0,5 \times 80 \times -3,160) + (-3,160 \times 15,722) = -176,08 \text{ t}$$

- **S32 = S63**



$$S = (0,5 \times 80 \times -0,996) + (-0,996 \times 15,722) = -55,499 \text{ t}$$

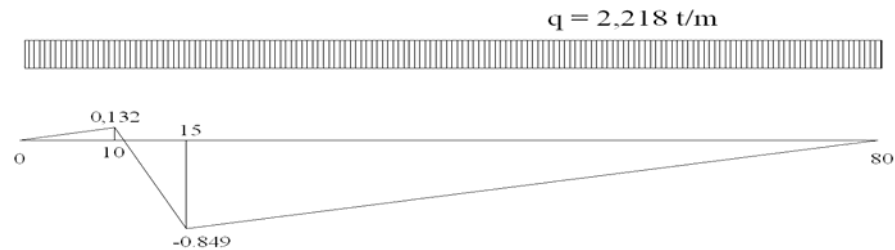
- **S33 = S62**



$$\frac{a}{0,070} = \frac{5-a}{0,912} \quad a = 0,415 \quad b = 4,585$$

$$\begin{aligned}
 S &= (0,5 \times -0,912 \times 74,585) \times 2,218 + (-0,912 \times 15,722) &= - 89,773 \text{ t} \\
 &= (0,5 \times 0,070 \times 5,415) \times 2,218 + (0,092 \times 15,722) &= 1,866 \text{ t}
 \end{aligned}$$

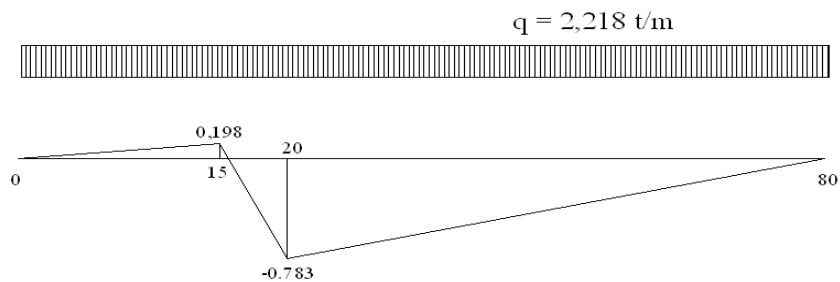
- **S34 = S61**



$$\frac{a}{0,132} = \frac{5-a}{0,849} \quad a = 0,672 \quad b = 4,328$$

$$\begin{aligned}
 S &= (0,5 \times -0,849 \times 69,328) \times 2,218 + (-0,849 \times 15,722) &= - 78,622 \text{ t} \\
 &= (0,5 \times 0,132 \times 10,672) \times 2,218 + (0,132 \times 15,722) &= 3,637 \text{ t}
 \end{aligned}$$

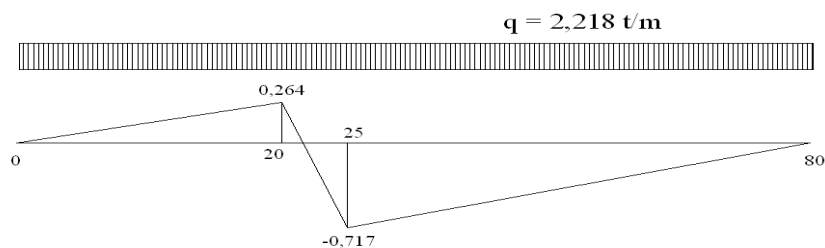
- **S35 = S60**



$$\frac{a}{0,198} = \frac{5-a}{0,783} \quad a = 1,009 \quad b = 3,991$$

$$\begin{aligned}
 S &= (0,5 \times -0,783 \times 63,991) \times 2,218 + (-0,783 \times 15,722) &= - 67,876 \text{ t} \\
 &= (0,5 \times 0,198 \times 16,009) \times 2,218 + (0,198 \times 15,722) &= 6,627 \text{ t}
 \end{aligned}$$

- **S36 = S59**



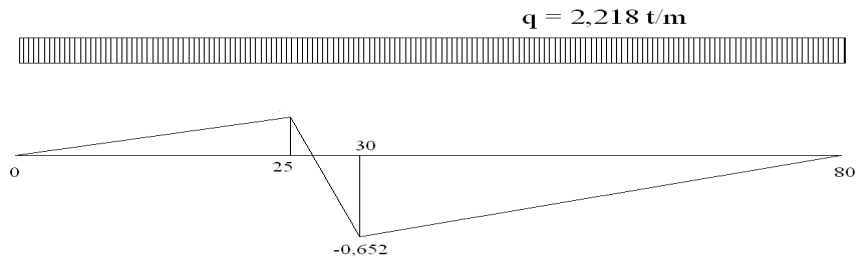


$$\frac{a}{0,264} = \frac{5-a}{0,717}$$

$$a = 1,345 \quad b = 3,665$$

$$\begin{aligned} S &= (0,5 \times -0,717 \times 68,665) \times 2,218 + (-0,717 \times 15,722) = -65,871 \text{ t} \\ &= (0,5 \times 0,264 \times 21,345) \times 2,218 + (0,264 \times 15,722) = 10,399 \text{ t} \end{aligned}$$

• **S37 = S58**

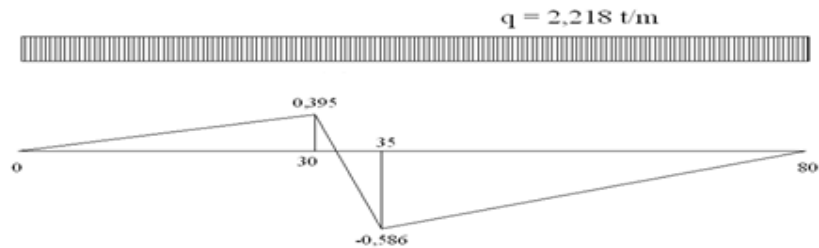


$$\frac{a}{0,329} = \frac{5-a}{0,652}$$

$$a = 1,676 \quad b = 3,324$$

$$\begin{aligned} S &= (0,5 \times -0,652 \times 53,324) \times 2,218 + (-0,652 \times 15,722) = -48,806 \text{ t} \\ &= (0,5 \times 0,329 \times 26,676) \times 2,218 + (0,329 \times 15,722) = 14,905 \text{ t} \end{aligned}$$

• **S38 = S57**

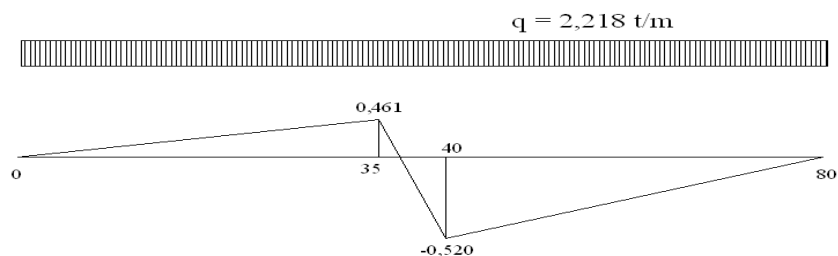


$$\frac{a}{0,395} = \frac{5-a}{0,586}$$

$$a = 2,013 \quad b = 2,987$$

$$\begin{aligned} S &= (0,5 \times -0,586 \times 37,987) \times 2,218 + (-0,586 \times 15,722) = -33,849 \text{ t} \\ &= (0,5 \times 0,395 \times 32,013) \times 2,218 + (0,395 \times 15,722) = 12,532 \text{ t} \end{aligned}$$

• **S39 = S56**

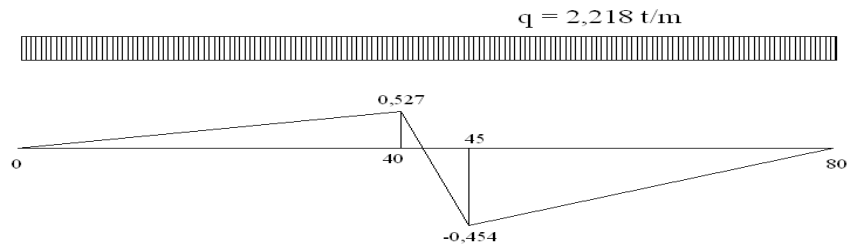


$$\frac{a}{0,461} = \frac{5-a}{0,520}$$

$$a = 1,879 \quad b = 3,121$$

$$\begin{aligned} S &= (0,5 \times -0,520 \times 43,121) \times 2,218 + (-0,520 \times 15,722) = -33,042 \text{ t} \\ &= (0,5 \times 0,461 \times 36,879) \times 2,218 + (0,461 \times 15,722) = 26,101 \text{ t} \end{aligned}$$

- **S40 = S55**

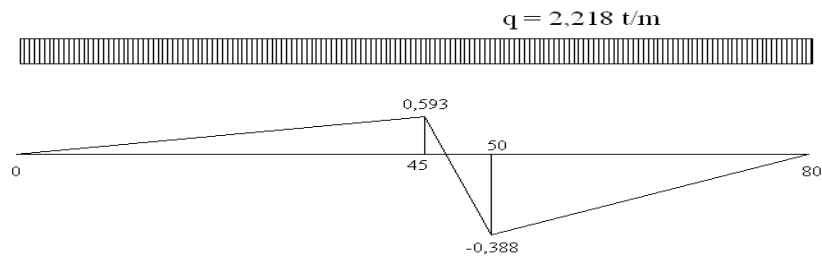


$$\frac{a}{0,527} = \frac{5-a}{0,454}$$

$$a = 2,686 \quad b = 2,314$$

$$\begin{aligned} S &= (0,5 \times -0,454 \times 37,314) \times 2,218 + (-0,454 \times 15,722) = -25,924 \text{ t} \\ &= (0,5 \times 0,527 \times 42,686) \times 2,218 + (0,527 \times 15,722) = 33,232 \text{ t} \end{aligned}$$

- **S41 = S54**

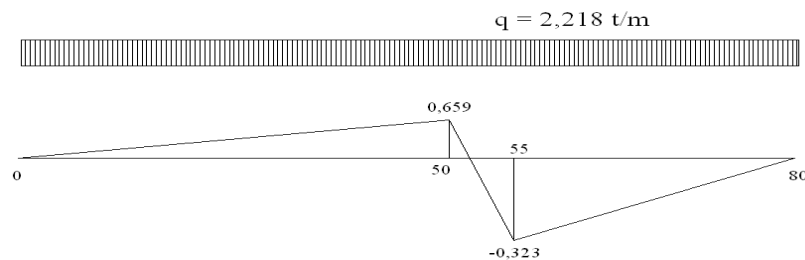


$$\frac{a}{0,593} = \frac{5-a}{0,388}$$

$$a = 3,022 \quad b = 1,978$$

$$\begin{aligned} S &= (0,5 \times -0,388 \times 31,978) \times 2,218 + (-0,388 \times 15,722) = -19,859 \text{ t} \\ &= (0,5 \times 0,593 \times 48,022) \times 2,218 + (0,593 \times 15,722) = 40,904 \text{ t} \end{aligned}$$

- **S42 = S53**

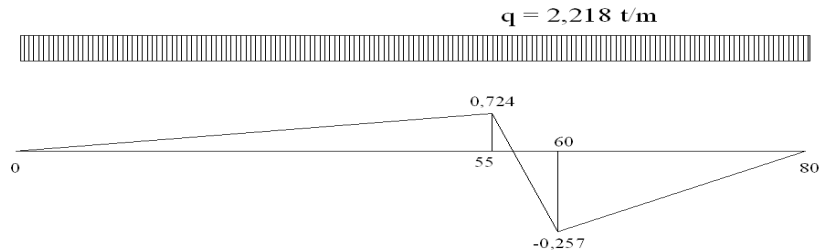


$$\frac{a}{0,659} = \frac{5-a}{0,323}$$

$$a = 3,355 \quad b = 1,645$$

$$\begin{aligned} S &= (0,5 \times -0,323 \times 26,645) \times 2,218 + (-0,323 \times 15,722) = -14,622 \text{ t} \\ &= (0,5 \times 0,659 \times 53,355) \times 2,218 + (0,659 \times 15,722) = 49,353 \text{ t} \end{aligned}$$

• **S43 = S52**

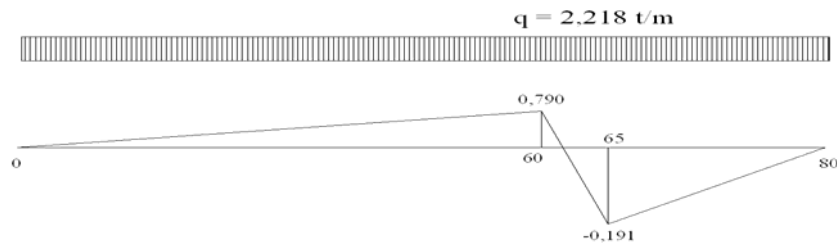


$$\frac{a}{0,724} = \frac{5-a}{0,257}$$

$$a = 3,690 \quad b = 1,310$$

$$\begin{aligned} S &= (0,5 \times -0,257 \times 21,310) \times 2,218 + (-0,257 \times 15,722) = -10,113 \text{ t} \\ &= (0,5 \times 0,724 \times 58,690) \times 2,218 + (0,724 \times 15,722) = 58,505 \text{ t} \end{aligned}$$

• **S44 = S51**

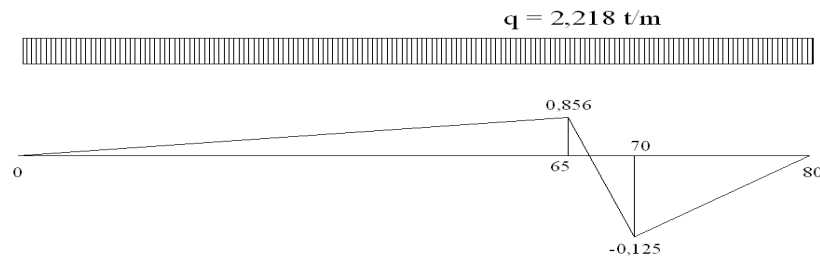


$$\frac{a}{0,790} = \frac{5-a}{0,191}$$

$$a = 4,026 \quad b = 0,974$$

$$\begin{aligned} S &= (0,5 \times -0,191 \times 15,974) \times 2,218 + (-0,191 \times 15,722) = -6,385 \text{ t} \\ &= (0,5 \times 0,790 \times 64,026) \times 2,218 + (0,790 \times 15,722) = 68,513 \text{ t} \end{aligned}$$

• **S45 = S50**



$$\frac{a}{0,856} = \frac{5-a}{0,125}$$

$$a = 4,362 \quad b = 0,638$$

$$\begin{aligned} S &= (0,5 \times -0,125 \times 10,638) \times 2,218 + (-0,125 \times 15,722) &= -3,439 \text{ t} \\ &= (0,5 \times 0,856 \times 69,362) \times 2,218 + (0,856 \times 15,722) &= 79,303 \text{ t} \end{aligned}$$

• **S46 = S49**

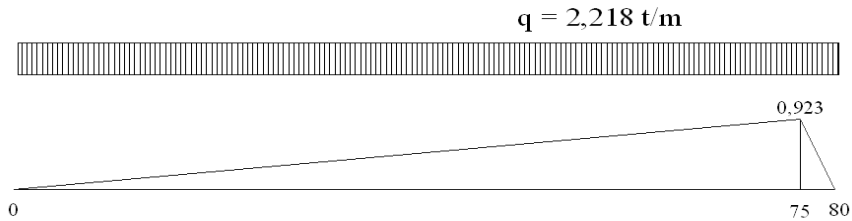


$$\frac{a}{0,923} = \frac{5-a}{0,058}$$

$$a = 4,704 \quad b = 0,296$$

$$\begin{aligned} S &= (0,5 \times -0,058 \times 5,296) \times 2,218 + (-0,058 \times 15,722) &= -1,251 \text{ t} \\ &= (0,5 \times 0,923 \times 74,704) \times 2,218 + (0,923 \times 15,722) &= 90,978 \text{ t} \end{aligned}$$

• **S47 = S48**



$$S = (0,5 \times 0,923 \times 80) \times 2,218 + (0,923 \times 15,722) = 96,399 \text{ t}$$

Tabel 5 .1 Rekapitulasi Gaya Batang (Ton)

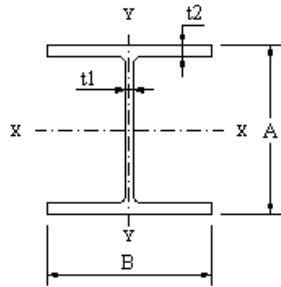
Btg.	Gaya Batang				Gaya Batang		Ket.
	Beban Mati ( t )		Beban Hidup ( t )		Total ( t )		
	Tarik	Tekan	Tarik	Tekan	Tarik	Tekan	
1	24.626		38.851		63.477		tarik
2	69.348		110.77		180.118		tarik
3	107.936		172.454		280.390		tarik
4	140.057		234.017		374.074		tarik
5	165.758		182.965		348.723		tarik
6	185.034		234.841		419.875		tarik
7	197.844		315.144		512.988		tarik
8	204.310		325.33		529.640		tarik
9	204.310		325.33		529.640		tarik
10	197.884		315.144		513.028		tarik
11	185.034		234.841		419.875		tarik
12	165.758		182.965		348.723		tarik
13	140.057		234.017		374.074		tarik
14	107.936		172.454		280.390		tarik
15	69.348		110.77		180.118		tarik
16	24.626		38.851		63.477		tarik
17		47.862		76.973		124.835	tekan
18		89.613		144.16		233.773	tekan
19		124.950		200.67		325.620	tekan
20		153.864		256.485		410.349	tekan
21		176.352		284.192		460.544	tekan
22		192.415		309.259		501.674	tekan
23		202.053		324.98		527.033	tekan
24		205.265		176.08		381.345	tekan
25		202.053		324.98		527.033	tekan
26		192.415		309.259		501.674	tekan
27		176.352		284.192		460.544	tekan
28		153.864		256.485		410.349	tekan
29		124.950		200.67		325.620	tekan
30		89.613		144.16		233.773	tekan
31		47.862		76.973		124.835	tekan
32		65.824		55.499		121.323	tekan
33		57.071	1.886	89.773		144.958	tekan
34		48.750	3.637	78.622		123.735	tekan
35		40.179	6.627	67.876		101.428	tekan
36		31.634	10.399	65.871		87.106	tekan
37		23.087	14.905	48.806		56.988	tekan
38		14.540	12.532	33.849		35.857	tekan
39		5.933	26.101	33.042		12.874	tekan
40	2.553		33.232	25.924	9.861		tarik
41	11.100		40.904	19.859	32.145		tarik
42	19.647		49.353	14.622	54.378		tarik

43	28.194		58.505	10.113	76.586		tarik
44	36.742		68.513	6.385	98.870		tarik
45	45.278		79.303	3.439	121.142		tarik
46	53.888		90.978	1.251	143.615		tarik
47	62.146		96.399		158.545		tarik
48	62.146		96.399		158.545		tarik
49	53.888		90.978	1.251	143.615		tarik
50	45.278		79.303	3.439	121.142		tarik
51	36.742		68.513	6.385	98.870		tarik
52	28.194		58.505	10.113	76.586		tarik
53	19.647		49.353	14.622	54.378		tarik
54	11.100		40.904	19.859	32.145		tarik
55	2.553		33.232	25.924	9.861		tarik
56		5.993	26.101	33.042		12.934	tekan
57		14.540	12.532	33.849		35.857	tekan
58		23.087	14.905	48.806		56.988	tekan
59		31.634	10.399	65.871		87.106	tekan
60		40.179	6.627	67.876		101.428	tekan
61		48.750	3.637	78.622		123.735	tekan
62		57.071	1.886	89.773		144.958	tekan
63		65.824		55.499		121.323	tekan

### 5.2.12 Pendimensionian Batang Rangka Induk

#### 1) Batang Horizontal Bawah (Batang Tarik)

Direncanakan menggunakan profil IWF 428 x 407 x 20 x 35 - 283



Data Profil :

A = 428 mm	$i_x = 18,2 \text{ cm}$
B = 407 mm	$i_y = 10,4 \text{ cm}$
t1 = 20 mm	$W_x = 5570 \text{ cm}^3$
t2 = 35 mm	$W_y = 1930 \text{ cm}^3$
F = 360,7 cm <sup>2</sup>	
$I_x = 119000 \text{ cm}^4$	
$I_y = 39400 \text{ cm}^4$	

$$S = 529,640 \text{ t} = 529640 \text{ kg ( batang 8 \& 9 )}$$

$$\begin{aligned} F_{nt} &= 0,85 \times F \text{ profil} \\ &= 0,85 \times 360,7 \\ &= 306,595 \text{ cm}^2 \end{aligned}$$

Cek Tegangan :

$$\begin{aligned} \sigma &= \frac{S}{F_{nt}} < 1867 \text{ kg/cm}^2 \\ &= \frac{529640}{306,595} < 1867 \text{ kg/cm}^2 \\ &= 1727,490 \text{ kg/cm}^2 < 1867 \text{ kg/cm}^2 \dots \text{OK} \end{aligned}$$

**2) Batang Horizontal Atas (Batang Tekan)**

Direncanakan menggunakan profil IWF 428 x 407 x 20 x 35 - 283

$$S = -527,033 \text{ t} = -527033 \text{ kg (batang 23 \& 25)}$$

$$Lk = 500 \text{ cm}$$

Rumus umum menurut PPBBI hal 9 Bab 4.1 Pasal 1 dan 2, untuk stabilitas batang tekan terhadap bahaya tekuk :

$$\frac{(S \times \omega)}{F} < \sigma \text{ ijin baja}$$

Dimana :

$S$  = gaya tekan pada batang tersebut

$F$  = luas penampang batang

$\omega$  = faktor tekuk yang tergantung dari kelangsingan dan macam bajanya

Menghitung kelangsingan batang tunggal :

$$\lambda = \frac{Lk}{I_{\min}} = \frac{500}{10,4} = 48,076$$

$$\lambda_g = \pi \times \sqrt{\frac{E}{0,7 \times \sigma_1}} = 3,14 \times \sqrt{\frac{2,1 \times 10^5}{0,7 \times 240}} = 111,015$$

$$\lambda_s = \frac{\lambda}{\lambda_g} = \frac{48,076}{111,015} = 0,433$$

Untuk  $0,183 < \lambda_s < 1$  :

$$\begin{aligned} \omega &= \frac{1,41}{1,593 - \lambda_s} \\ &= \frac{1,41}{1,593 - 0,433} \\ &= 1,16 \end{aligned}$$

Maka :

$$\frac{(S \times \omega)}{F} = \frac{527033 \times 1,160}{360,7} = 1694,922 \text{ Mpa} < 1867 \text{ kg/cm}^2 \text{.....OK}$$



**3) Batang Diagonal (Batang Tarik)**

Direncanakan menggunakan profil IWF 428 x 407 x 20 x 35 - 283

$$S = 158,545 \text{ t} = 158545 \text{ kg (batang 47 \& 48)}$$

$$F_{nt} = 0,85 \times F_{\text{profil}} = 0,85 \times 360,7 = 306,595 \text{ cm}^2$$

Cek Tegangan :

$$\begin{aligned}\sigma &= \frac{S}{F_{nt}} < 0,75 \times 1867 \text{ kg/cm}^2 \\ &= \frac{158545}{306,595} < 1400,25 \text{ kg/cm}^2 \\ &= 517,115 < 1400,25 \text{ kg/cm}^2 \dots \text{OK}\end{aligned}$$

**4) Batang Diagonal (Batang Tekan)**

Direncanakan menggunakan profil IWF 428 x 407 x 20 x 35 - 283

$$S = -144,758 \text{ t} = -144758 \text{ kg (batang 33 \& 62)}$$

$$L_k = 677,790 \text{ cm}$$

Rumus umum menurut PPBBI hal 9 Bab 4.1 Pasal 1 dan 2, untuk stabilitas batang

$$\text{tekan terhadap bahaya tekuk : } \frac{(S \times \omega)}{F} < \sigma \text{ ijin baja}$$

Menghitung kelangsingan batang tunggal :

$$\lambda = \frac{L_k}{I_{\min}} = \frac{677,790}{10,4} = 65,172$$

$$\lambda_g = \pi \times \sqrt{\frac{E}{0,7 \times \sigma_1}} = 3,14 \times \sqrt{\frac{2,1 \times 10^5}{0,7 \times 240}} = 111,015$$

$$\lambda_s = \frac{\lambda}{\lambda_g} = \frac{65,172}{111,015} = 0,587$$

Untuk  $0,183 < \lambda_s < 1$  :

$$\omega = \frac{1,41}{1,593 - \lambda_s} = \frac{1,41}{1,593 - 0,587} = 1,401$$

Maka :

$$\frac{(S \times \omega)}{F} = \frac{(144758 \times 1,401)}{360,7} = 542,878 < 186,7 \text{ Mpa} \dots \text{OK}$$

### 5.2.13 Sambungan Rangka Utama

#### a. Sambungan Antar Rangka Utama

Sambungan Antar rangka utama direncanakan menggunakan alat penyambung dengan baut diameter 25,4 mm

Data teknis perencanaan jumlah baut :

- Tebal plat buhul (  $\delta$  ) = 30 mm
- Diameter baut = 25,4 mm
- Mutu baut = A325 (  $\tau_t = 6350 \text{ kg/cm}^2$  )

Pengaturan jarak antar baut ( berdasar PPBBI hal 70 ) :

$2,5 d \leq s \leq 7 d$  , atau  $14 t \leq s$  = jarak antar sumbu baut pada arah horizontal

$2,5 d \leq u \leq 7 d$  , atau  $14 t \leq u$  = jarak antar sumbu baut pada arah vertikal

$1,5 d \leq s_1 \leq 3 d$  , atau  $6 t \leq s_1$  = jarak sumbu baut paling luar dengan bagian yang disambung

Jarak antar sumbu baut pada arah horizontal (  $s$  ) :

$$2,5 d \leq s \leq 7 d$$

$$63,5 \leq s \leq 177,8 \quad \text{diambil } 80 \text{ mm}$$

Jarak antar sumbu baut pada arah vertikal (  $u$  ) :

$$2,5 d \leq u \leq 7 d$$

$$63,5 \leq s \leq 177,8 \quad \text{diambil } 100 \text{ mm}$$

Jarak sumbu baut paling luar dengan bagian yang disambung (  $s_1$  ) :

$$1,5 d \leq s_1 \leq 3 d$$

$$38,1 \leq s_1 \leq 76,2 \quad \text{diambil } 60 \text{ mm}$$

Sambungan Irisan 1 :

$$\frac{\delta}{d} = \frac{30}{25,4} = 1,180 > 0,314 \text{ ( Pengaruh Geser )}$$

Jumlah baut untuk tiap sisi pelat sambungan:

$$n_{gsr} = \frac{S}{0,6 \times \sigma \times 2 \times \frac{1}{4} \pi d^2}$$

$$n_{gsr} = \frac{S}{0,6 \times 6350 \times 2 \times \frac{1}{4} \times 3,14 \times 2,54^2}$$

$$n_{gsr} = \frac{S}{38591,535}$$

Tabel 5.2 Perhitungan Jumlah Baut

Btg	Gaya batang ( kg )	Pgeser ( kg )	Jumlah Baut	Dipakai
1	63.477	38.591	1.645	14
2	180.118	38.591	4.667	14
3	280.390	38.591	7.266	14
4	374.074	38.591	9.693	14
5	348.723	38.591	9.036	14
6	419.875	38.591	10.880	14
7	512.988	38.591	13.293	14
8	529.640	38.591	13.724	14
9	529.640	38.591	13.724	14
10	513.028	38.591	13.294	14
11	419.875	38.591	10.880	14
12	348.723	38.591	9.036	14
13	374.074	38.591	9.693	14
14	280.390	38.591	7.266	14
15	180.118	38.591	4.667	14
16	63.477	38.591	1.645	14
17	124.835	38.591	3.235	14
18	233.773	38.591	6.058	14
19	325.620	38.591	8.438	14
20	410.349	38.591	10.633	14
21	460.544	38.591	11.934	14
22	501.674	38.591	13.000	14
23	527.033	38.591	13.657	14
24	381.345	38.591	9.882	14
25	527.033	38.591	13.657	14
26	501.674	38.591	13.000	14
27	460.544	38.591	11.934	14
28	410.349	38.591	10.633	14
29	325.620	38.591	8.438	14
30	233.773	38.591	6.058	14
31	124.835	38.591	3.235	14
32	121.323	38.591	3.144	10
33	144.958	38.591	3.756	10
34	123.735	38.591	3.206	10
35	101.428	38.591	2.628	10
36	87.106	38.591	2.257	10
37	56.988	38.591	1.477	10
38	35.857	38.591	0.929	10
39	12.874	38.591	0.334	10
40	9.861	38.591	0.256	10
41	32.145	38.591	0.833	10
42	54.378	38.591	1.409	10
43	76.586	38.591	1.985	10
44	98.870	38.591	2.562	10
45	121.142	38.591	3.139	10
46	143.615	38.591	3.721	10

47	158.545	38.591	4.108	10
48	158.545	38.591	4.108	10
49	143.615	38.591	3.721	10
50	121.142	38.591	3.139	10
51	98.870	38.591	2.562	10
52	76.586	38.591	1.985	10
53	54.378	38.591	1.409	10
54	32.145	38.591	0.833	10
55	9.861	38.591	0.256	10
56	12.934	38.591	0.335	10
57	35.857	38.591	0.929	10
58	56.988	38.591	1.477	10
59	87.106	38.591	2.257	10
60	101.428	38.591	2.628	10
61	123.735	38.591	3.206	10
62	144.958	38.591	3.756	10
63	121.323	38.591	3.144	10

**b. Sambungan Antar Rangka Utama dengan Gelagar Melintang**

Sambungan antara rangka utama dengan gelagar melintang direncanakan menggunakan pelat penyambung dengan tebal 20 mm yang dilas pada ujung gelagar melintang.

Perhitungan Sambungan Las :

$$D_{\text{POST}} \text{ gelagar melintang ( P )} = 51563,749 \text{ kg}$$

$$\text{Tebal plat ( t )} = 20 \text{ mm}$$

- Perhitungan luas bidang las

$$a \leq \frac{1}{2} t \sqrt{2}$$

$$a \leq \frac{1}{2} \times 20 \times \sqrt{2}$$

$$a \leq 14,142 \text{ mm, diambil } a = 12 \text{ mm}$$

- Panjang netto las (  $L_n$  )

$$L_B = \text{tinggi profil gelagar melintang} = 708 \text{ mm}$$

$$L_n = L_B - 3a$$

$$= 708 - (3 \times 12)$$

$$= 672 \text{ mm}$$

$$\text{Syarat panjang las : } 10.a \leq L_n \leq 40.a$$

$$10 \times 12 \leq L_n \leq 40 \times 12$$

$$120 \leq L_n \leq 480 \text{ mm}$$

$$\text{Panjang Las diambil } L_n = 400 \text{ mm}$$

- Luas bidang las ( A )

$$A = 2 \times L_n \times a$$

$$= 2 \times 400 \times 12$$

$$= 11520 \text{ mm}^2 = 115,20 \text{ cm}^2$$

- Kekuatan las (  $\bar{P}$  )

$$\bar{P} = 0,58 \times \bar{\sigma} \times A$$

$$= 0,58 \times 1867 \times 115,20$$

$$= 124745,472 \text{ kg}$$

Karena  $\bar{P} \geq P$ , maka sambungan las antara pelat penyambung dengan gelagar melintang pada ujungnya aman.

Sambungan antara pelat buhul dengan pelat penyambung direncanakan menggunakan baut mutu tinggi A 325 dengan diameter 1 " ( 25,4 mm )

Jarak antar sumbu baut pada arah horizontal ( s ) :

$$2,5 d \leq s \leq 7 d$$

$$63,5 \leq s \leq 177,8 \quad \text{diambil } 70 \text{ mm}$$

Jarak antar sumbu baut pada arah vertikal ( u ) :

$$2,5 d \leq u \leq 7 d$$

$$63,5 \leq s \leq 177,8 \quad \text{diambil } 80 \text{ mm}$$

Jarak sumbu baut paling luar dengan bagian yang disambung ( s<sub>1</sub> ) :

$$1,5 d \leq s_1 \leq 3 d$$

$$38,1 \leq s_1 \leq 76,2 \quad \text{diambil } 40 \text{ mm}$$

Menentukan jumlah baut :

- Tebal plat buhul (  $\delta$  ) = 30 mm
- Diameter baut = 25,4 mm
- Tegangan geser ijin (  $\bar{\sigma}$  ) =  $0,58 \times \bar{\sigma} = 1082 \text{ kg/cm}^2$

Sambungan Irisan 1 :

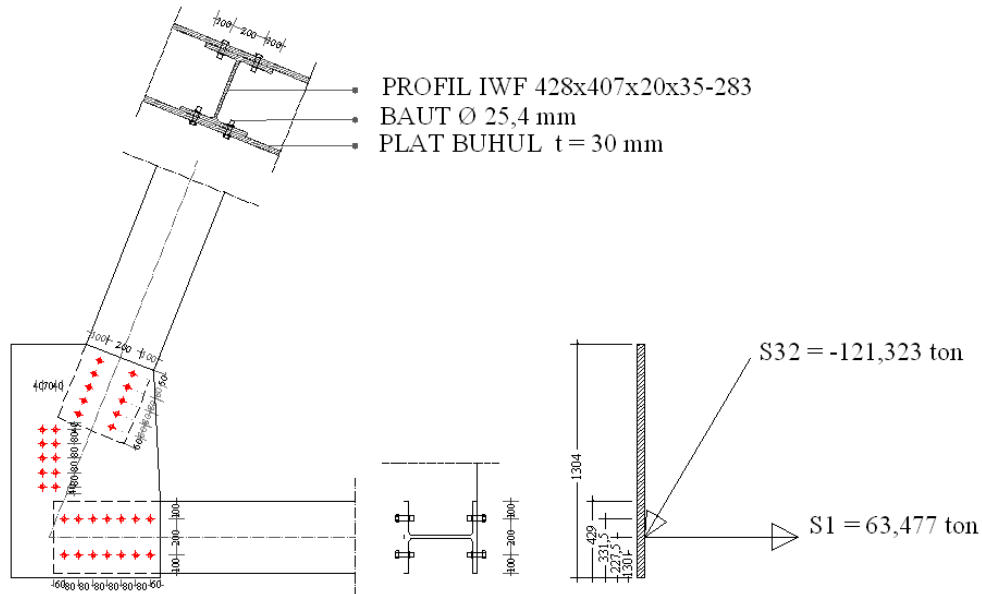
$$\frac{\delta}{d} = \frac{30}{25,4} = 1.181 > 0,314 \text{ ( Pengaruh Geser )}$$

$$n = \frac{P}{\tau \left( \frac{1}{4} \times \pi \times d^2 \right)} = \frac{51563,749}{1082 \times \left( \frac{1}{4} \times 3,14 \times 2,54^2 \right)}$$

$$= 8,960 \longrightarrow \text{dipakai } 10 \text{ baut}$$

## 5.2.14 Perhitungan Stabilitas Pelat Buhul

### a. Buhul A



Gambar 5.35 Detail Buhul A

### Tinjau Pot. A – A

#### Analisa Penampang :

- A bruto =  $3 \times 130,45 = 391,35 \text{ cm}^2$
- A baut =  $2 \times (3 \times 2,54) = 15,24 \text{ cm}^2$
- A netto = A bruto - A netto =  $391,35 - 15,24 = 376,11 \text{ cm}^2$
- Titik berat penampang pada pot. A – A

$$Y = \frac{(376,11 \times 65,2) - ((3 \times 2,54) \times (13 + 22,75))}{376,11} = 64,475 \text{ cm}$$

$$\begin{aligned} \bullet I_{\text{netto}} &= \left( \left( \frac{1}{12} \times 3 \times 130,45^3 \right) + (391,35 \times (65,2 - 64,675)^2) \right) - \\ &\quad (3 \times 2,54 \times ((13 - 64,675)^2 + (33,15 - 64,675)^2)) \\ &= (554973,516 + 109,930) - (7,62 \times (2672,373 + 995,087)) \\ &= 527137,400 \text{ cm}^4 \end{aligned}$$

$$\bullet \text{Watas} = \frac{I_{\text{netto}}}{H - Y} = \frac{527137,400}{130,45 - 64,475} = 8116,049 \text{ cm}^3$$



$$\bullet \quad W_{\text{bawah}} = \frac{I_{\text{netto}}}{Y} = \frac{527137,400}{64,475} = 8178,842 \text{ cm}^3$$

Gaya – Gaya yang bekerja :

$$\bullet \quad N = \frac{1}{2} \times \left( \left( \frac{63,477 \times 12}{14} \right) + (-121,323 \times \cos 68.090) \right) = 4,568 \text{ Ton}$$

$$\bullet \quad D = \frac{1}{2} \times (-121,323 \sin 68.090) = -56,279 \text{ Ton}$$

$$\bullet \quad M =$$

$$\begin{aligned} & \frac{1}{2} \times \left( \left( \frac{63,477 \times 12 \times (64,475 - 13)}{14} \right) + (-121,323 \cos 68.090 \times (64,475 - 33,15)) \right) \\ & = 691,565 \text{ Ton.cm} \end{aligned}$$

Tegangan Yang Terjadi :

- Akibat N

$$\sigma_n = \frac{N}{A_{\text{netto}}} = \frac{4568}{376,11} = 12,145 \text{ kg/cm}^2$$

- Akibat D

$$\tau = \frac{D}{A_{\text{netto}}} = \frac{-56279}{376,11} = -149,634 \text{ kg/cm}^2$$

- Akibat M

$$\sigma_{\text{atas}} = \frac{M}{W_{\text{atas}}} = \frac{691565}{8116,049} = 85,209 \text{ kg/cm}^2$$

$$\sigma_{\text{bawah}} = \frac{M}{W_{\text{bawah}}} = \frac{691565}{8178,842} = 84,555 \text{ kg/cm}^2$$

Tegangan total :

$$\sigma_{\text{atas}} = 85,209 - 12,145 = 73,064 \text{ kg/cm}^2$$

$$\sigma_{\text{bawah}} = 84,555 - 12,145 = 72,410 \text{ kg/cm}^2$$

Tegangan idiil :

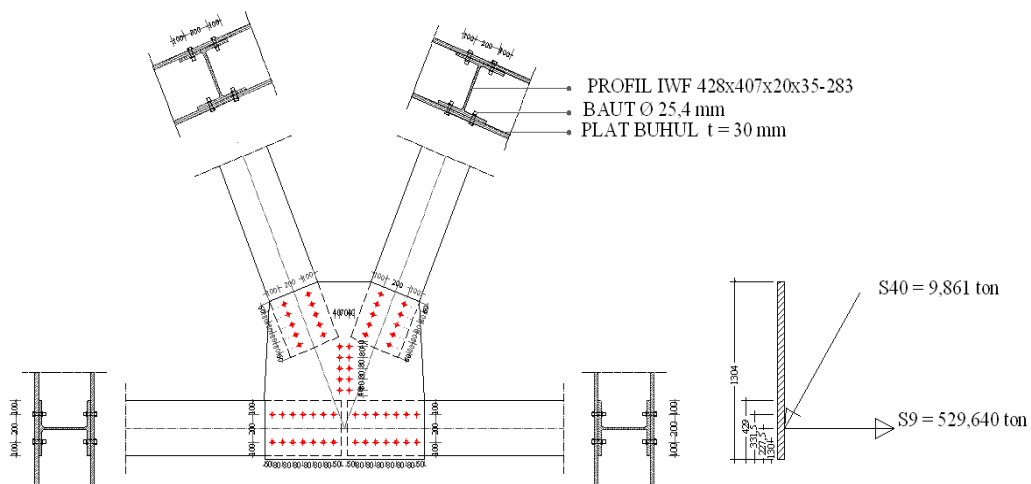
$$\sigma_{\text{idiil}} = \sqrt{(73,064)^2 + (3(-149,634))^2} = 269,275 \text{ kg/cm}^2$$

Syarat Keamanan :

$$\sigma_{diil} < \bar{\sigma}$$

$$269,275 \text{ kg/cm}^2 < 1867 \text{ kg/cm}^2 \text{ .....OK}$$

**b. Buhul B**



Gambar 5.36 Detail Buhul B

**Tinjau Pot. A – A**

Analisa Penampang :

- A bruto =  $3 \times 130,45 = 391,35 \text{ cm}^2$
- A baut =  $2 \times (3 \times 2,54) = 15,24 \text{ cm}^2$
- A netto = A bruto - A netto =  $391,35 - 15,24 = 376,11 \text{ cm}^2$
- Titik berat penampang pada pot. A – A

$$Y = \frac{(376,11 \times 65,2) - ((3 \times 2,54) \times (13 + 22,75))}{376,11} = 64,475 \text{ cm}$$

$$\begin{aligned} \bullet I_{\text{netto}} &= \left( \left( \frac{1}{12} \times 3 \times 130,45^3 \right) + \left( 391,35 \times (65,2 - 64,475)^2 \right) \right) - \\ &\quad \left( 3 \times 2,54 \times \left( (13 - 64,475)^2 + (33,15 - 64,475)^2 \right) \right) \\ &= (554973,516 + 109,930) - (7,62 \times (2672,373 + 995,087)) \\ &= 527137,400 \text{ cm}^4 \end{aligned}$$

- $W_{atas} = \frac{I_{netto}}{H-Y} = \frac{527137,400}{130,45-64,475} = 8116,049 \text{ cm}^3$
- $W_{bawah} = \frac{I_{netto}}{Y} = \frac{527137,400}{64,475} = 8178,842 \text{ cm}^3$

Gaya – Gaya yang bekerja :76

- $N = \frac{1}{2} \times \left( \left( \frac{529,640 \times 12}{14} \right) + (-9,861 \times \cos 68.090) \right) = 225,149 \text{ Ton}$
- $D = \frac{1}{2} \times (-9,861 \sin 68.090) = -4,574 \text{ Ton}$
- $M = \frac{1}{2} \times \left( \left( \frac{529,640 \times 12 \times (64,475-13)}{14} \right) + (-139,769 \cos 68.090 \times (64,475-33,15)) \right)$   
 $= 10867,690 \text{ Ton.cm}$

Tegangan Yang Terjadi :

- Akibat N

$$\sigma_n = \frac{N}{A_{netto}} = \frac{225149}{244.76} = 919,876 \text{ kg/cm}^2$$

- Akibat D

$$\tau = \frac{D}{A_{netto}} = \frac{-4574}{244.76} = -18,687 \text{ kg/cm}^2$$

- Akibat M

$$\sigma_{atas} = \frac{M}{W_{atas}} = \frac{10867690}{8116,049} = 1339,037 \text{ kg/cm}^2$$

$$\sigma_{bawah} = \frac{M}{W_{bawah}} = \frac{10867690}{8178,842} = 1328,756 \text{ kg/cm}^2$$

Tegangan total :

$$\sigma_{atas} = 1339,037 - 919,876 = 419,161 \text{ kg/cm}^2$$

$$\sigma_{bawah} = 1328,756 - 919,876 = 408,88 \text{ kg/cm}^2$$

Tegangan idiil :

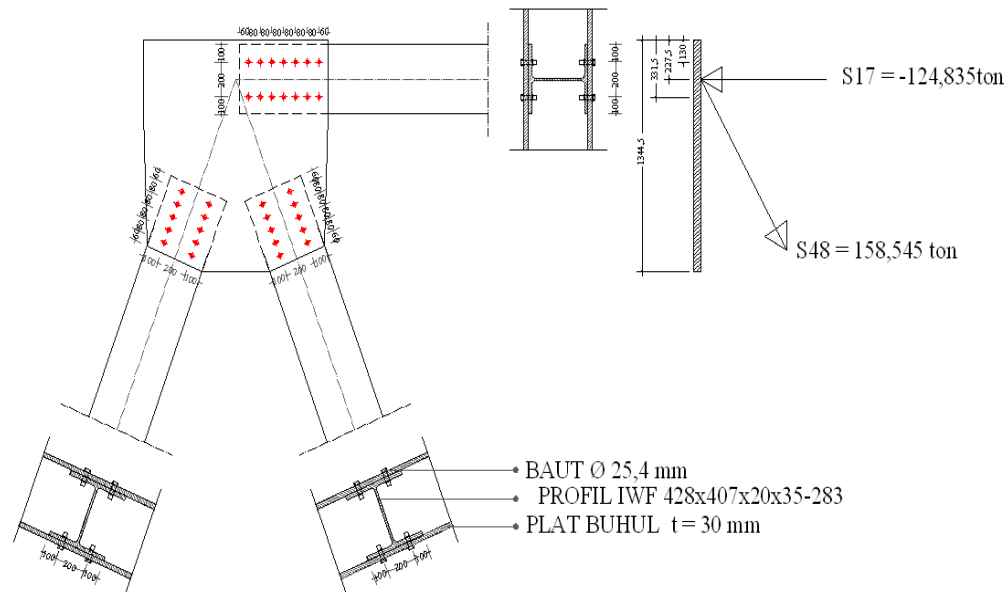
$$\sigma_{idiil} = \sqrt{(419,161)^2 + (3(-18,687)^2)} = 420,408 \text{ kg/cm}^2$$

Syarat Keamanan :

$$\sigma_{idiil} < \bar{\sigma}$$

$$420,408 \text{ kg/cm}^2 < 1867 \text{ kg/cm}^2 \text{ .....OK}$$

### c. Buhul C



Gambar 5.37 Detail Buhul C

### Tinjau Pot. A – A

Analisa Penampang :

- A bruto =  $3 \times 134,45 = 403,35 \text{ cm}^2$
- A baut =  $2 \times (3 \times 2,54) = 15,24 \text{ cm}^2$
- A netto = A bruto - A netto =  $403,35 - 15,24 = 388,11 \text{ cm}^2$
- Titik berat penampang pada pot. A – A

$$Y = \frac{(388,11 \times 67,225) - ((3 \times 2,54) \times (13 + 33,15))}{388,11} = 66,318 \text{ cm}$$

- $$I_{\text{netto}} = \left( \left( \frac{1}{12} \times 3 \times 134,45^3 \right) + \left( 403,35 \times (67,225 - 66,318)^2 \right) \right) - \left( 3 \times 2,54 \times \left( (13 - 66,318)^2 + (33,15 - 66,318)^2 \right) \right)$$

$$= (607606,524 + 331,815) - (7,62 \times (2842,809 + 1100,116))$$

$$= 577893,251 \text{ cm}^4$$
- $$\text{Watas} = \frac{I_{\text{netto}}}{H - Y} = \frac{577893,251}{134,45 - 66,318} = 8481,965 \text{ cm}^3$$
- $$\text{Wbawah} = \frac{I_{\text{netto}}}{Y} = \frac{577893,251}{66,318} = 8713,973 \text{ cm}^3$$

Gaya – Gaya yang bekerja :

- $$N = \frac{1}{2} \times \left( \left( \frac{-124,835 \times 12}{14} \right) + (158,545 \times \cos 68.090) \right) = -83,081 \text{ Ton}$$
- $$D = \frac{1}{2} \times (158,545 \sin 68.090) = 73,546 \text{ Ton}$$
- $$M = \frac{1}{2} \times \left( \left( \frac{-124,835 \times 12 \times (66,318 - 13)}{14} \right) - (158,545 \times \cos 68.090 \times (67,334 - 33,15)) \right)$$

$$= -3863,731 \text{ Ton.cm}$$

Tegangan Yang Terjadi :

- Akibat N

$$\sigma_n = \frac{N}{A_{\text{netto}}} = \frac{-83081}{388,11} = -214,065 \text{ kg/cm}^2$$
- Akibat D

$$\tau = \frac{D}{A_{\text{netto}}} = \frac{73546}{388,11} = 189,497 \text{ kg/cm}^2$$
- Akibat M

$$\sigma_{\text{atas}} = \frac{M}{W_{\text{atas}}} = \frac{-3863731}{8481,965} = -455,253 \text{ kg/cm}^2$$

$$\sigma_{\text{bawah}} = \frac{M}{W_{\text{bawah}}} = \frac{-3863731}{8713,973} = -443,934 \text{ kg/cm}^2$$

Tegangan total :

$$\sigma_{atas} = -455,253 + 214,065 = -241,188 \text{ kg/cm}^2$$

$$\sigma_{bawah} = -443,934 + 229,869 = -214,065 \text{ kg/cm}^2$$

Tegangan idiil :

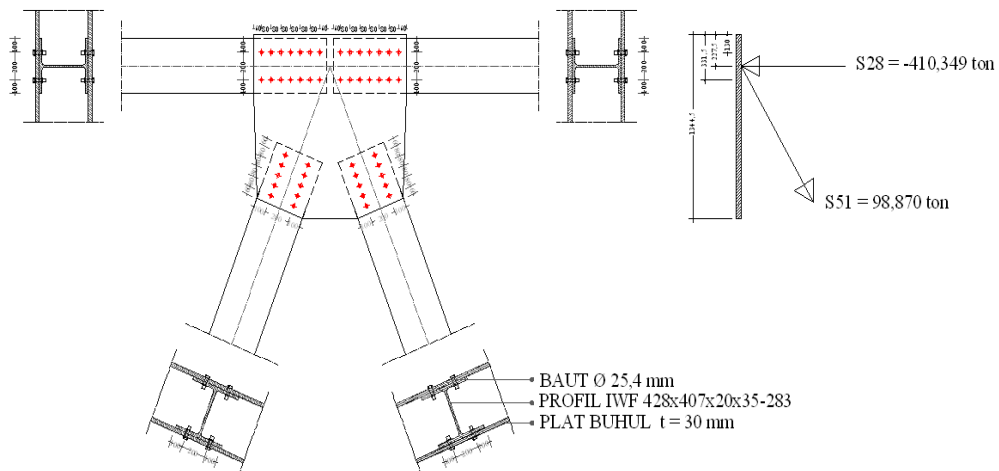
$$\sigma_{idiil} = \sqrt{-(241,188)^2 + (3(-189,497))^2} = 407,306 \text{ kg/cm}^2$$

Syarat Keamanan :

$$\sigma_{idiil} < \bar{\sigma}$$

$$407,306 \text{ kg/cm}^2 < 1867 \text{ kg/cm}^2 \text{ .....OK}$$

#### d. Buhul D



Gambar 5.38 Detail Buhul 19

#### Tinjau Pot. A – A

Analisa Penampang :

- A bruto =  $3 \times 134,45 = 403,35 \text{ cm}^2$
- A baut =  $2 \times (3 \times 2,54) = 15,24 \text{ cm}^2$
- A netto =  $A \text{ bruto} - A \text{ netto} = 403,35 - 15,24 = 388,11 \text{ cm}^2$
- Titik berat penampang pada pot. A – A

$$Y = \frac{(388,11 \times 67,225) - ((3 \times 2.54) \times (13 + 33,15))}{388,11} = 66,318 \text{ cm}$$

- $$I_{\text{netto}} = \left( \left( \frac{1}{12} \times 3 \times 134,45^3 \right) + \left( 403,35 \times (67,225 - 66,318)^2 \right) \right) - \left( 3 \times 2.54 \times ((13 - 66,318)^2 + (33,15 - 66,318)^2) \right)$$

$$= (607606,524 + 331,815) - (7.62 \times (2842,809 + 1100,116))$$

$$= 577893,251 \text{ cm}^4$$
- $$\text{Watas} = \frac{I_{\text{netto}}}{H - Y} = \frac{577893,251}{134,45 - 66,318} = 8481,965 \text{ cm}^3$$
- $$\text{Wbawah} = \frac{I_{\text{netto}}}{Y} = \frac{577893,251}{66,318} = 8713,973 \text{ cm}^3$$

Gaya – Gaya yang bekerja :

- $$N = \frac{1}{2} \times \left( \left( \frac{-410,349 \times 12}{14} \right) + (98,870 \times \cos 68.090) \right) = -157,424 \text{ Ton}$$
- $$D = \frac{1}{2} \times (98,870 \sin 68.090) = 45,628 \text{ Ton}$$
- $$M = \frac{1}{2} \times \left( \left( \frac{-410,349 \times 12 \times (66,318 - 13)}{14} \right) - (98,870 \times \cos 68.090 \times (66,318 - 33,15)) \right)$$

$$= -1549,264 \text{ Ton.cm}$$

Tegangan Yang Terjadi :

- Akibat N

$$\sigma_n = \frac{N}{A_{\text{netto}}} = \frac{-157424}{388,11} = -405,616 \text{ kg/cm}^2$$

- Akibat D

$$\tau = \frac{D}{A_{\text{netto}}} = \frac{45628}{388,11} = 117,564 \text{ kg/cm}^2$$

- Akibat M

$$\sigma_{\text{atas}} = \frac{M}{W_{\text{atas}}} = \frac{-1549264}{8481,965} = -182,653 \text{ kg/cm}^2$$

$$\sigma_{bawah} = \frac{M}{W_{bawah}} = \frac{-1549264}{8713,973} = -177,970 \text{ kg/cm}^2$$

Tegangan total :

$$\sigma_{atas} = -182,653 + 405,616 = 222,963 \text{ kg/cm}^2$$

$$\sigma_{bawah} = -177,970 + 405,616 = 227,646 \text{ kg/cm}^2$$

Tegangan idiil :

$$\sigma_{idiil} = \sqrt{(227,646)^2 + (3 (117,564))^2} = 308,428 \text{ kg/cm}^2$$

Syarat Keamanan :

$$\sigma_{idiil} < \bar{\sigma}$$

$$308,428 \text{ kg/cm}^2 < 1867 \text{ kg/cm}^2 \text{ .....OK}$$



### 5.2.15 Perhitungan Lendutan Rangka Induk

$$\text{Rumus} = \delta_m = \frac{(S_{DL} + S_{LL}) \times L \times S_o}{A \times E}$$

Keterangan :

$\delta_m$  = lendutan dititik ditengah

$S_{DL}$  = Gaya akibat beban mati

$S_{LL}$  = Gaya akibat beban hidup

$L$  = Panjang Batang

$S_o$  = Gaya batang akibat beban P ditengah

$A$  = Luas Profil

$E$  = Modulus Elastisitas Baja ( $2,1 \times 10^6 \text{ kg/cm}^2$ )

Tabel 5.3 Perhitungan Lendutan Rangka Induk

Batan g	$S_{DL} + S_{LL}$ (kg)	L (cm)	$S_o$ (kg)	A (cm <sup>2</sup> )	E (kg/cm <sup>2</sup> )	$\delta$ (cm)
1	63476.530	500	0.199	273.6	2100000	0.011
2	180117.820	500	0.593	273.6	2100000	0.093
3	280389.630	500	0.989	273.6	2100000	0.241
4	374073.840	500	1.39	273.6	2100000	0.452
5	348723.330	500	1.78	273.6	2100000	0.540
6	419874.940	500	2.18	273.6	2100000	0.797
7	512988.390	500	2.57	273.6	2100000	1.147
8	529639.610	500	2.97	273.6	2100000	1.369
9	529639.610	500	2.97	273.6	2100000	1.369
10	513028.390	500	2.57	273.6	2100000	1.147
11	419874.940	500	2.18	273.6	2100000	0.797
12	348723.330	500	1.78	273.6	2100000	0.540
13	374073.840	500	1.39	273.6	2100000	0.452
14	280389.630	500	0.989	273.6	2100000	0.241
15	180117.820	500	0.593	273.6	2100000	0.093
16	63476.530	500	0.199	273.6	2100000	0.011
17	-124835.410	500	-0.396	273.6	2100000	0.043
18	-233772.910	500	-0.791	273.6	2100000	0.161
19	-325619.530	500	-1.19	273.6	2100000	0.337
20	-410348.860	500	-1.58	273.6	2100000	0.564
21	-460543.950	500	-1.98	273.6	2100000	0.794
22	-501674.050	500	-2.37	273.6	2100000	1.035
23	-527032.880	500	-2.77	273.6	2100000	1.270
24	-381345.480	500	-3.16	273.6	2100000	1.049
25	-527032.880	500	-2.77	273.6	2100000	1.270
26	-501674.050	500	-2.37	273.6	2100000	1.035
27	-460543.950	500	-1.98	273.6	2100000	0.794
28	-410348.860	500	-1.58	273.6	2100000	0.564
29	-325619.530	500	-1.19	273.6	2100000	0.337

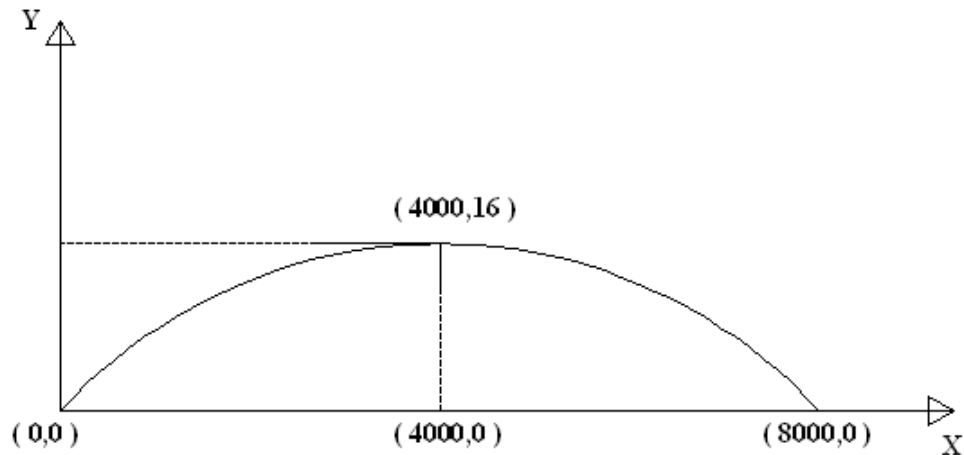
30	-233772.910	500	-0.791	273.6	2100000	0.161
31	-124835.410	500	-0.396	273.6	2100000	0.043
32	-121323.420	677.779	-0.533	273.6	2100000	0.076
33	-144958.280	677.779	-0.525	273.6	2100000	0.090
34	-123735.370	677.779	-0.526	273.6	2100000	0.077
35	-101427.740	677.779	-0.526	273.6	2100000	0.063
36	-87106.100	677.779	-0.526	273.6	2100000	0.054
37	-56988.140	677.779	-0.526	273.6	2100000	0.035
38	-35857.280	677.779	-0.527	273.6	2100000	0.022
39	-12874.440	677.779	-5.205	273.6	2100000	0.079
40	9861.420	677.779	0.527	273.6	2100000	0.006
41	32145.260	677.779	0.525	273.6	2100000	0.020
42	54378.150	677.779	0.526	273.6	2100000	0.034
43	76585.720	677.779	0.526	273.6	2100000	0.048
44	98870.450	677.779	0.526	273.6	2100000	0.061
45	121141.690	677.779	0.526	273.6	2100000	0.075
46	143614.940	677.779	0.526	273.6	2100000	0.089
47	158544.910	677.779	0.524	273.6	2100000	0.072
48	158544.910	677.779	0.524	273.6	2100000	0.072
49	143614.940	677.779	0.526	273.6	2100000	0.089
50	121141.690	677.779	0.526	273.6	2100000	0.075
51	98870.450	677.779	0.526	273.6	2100000	0.061
52	76585.720	677.779	0.526	273.6	2100000	0.048
53	54378.150	677.779	0.526	273.6	2100000	0.034
54	32145.260	677.779	0.525	273.6	2100000	0.020
55	9861.420	677.779	0.527	273.6	2100000	0.006
56	-12934.440	677.779	-0.52	273.6	2100000	0.008
57	-35857.280	677.779	-0.527	273.6	2100000	0.022
58	-56988.140	677.779	-0.526	273.6	2100000	0.035
59	-87106.100	677.779	-0.526	273.6	2100000	0.054
60	-101427.740	677.779	-0.526	273.6	2100000	0.063
61	-123735.370	677.779	-0.526	273.6	2100000	0.077
62	-144958.280	677.779	-0.525	273.6	2100000	0.090
63	-121323.420	677.779	-0.533	273.6	2100000	0.076
$\Sigma$						14,496

Cek lendutan :

$$\Delta \leq 1/500 \times 8000$$

$$14,496 \text{ cm} \leq 16 \text{ cm} \dots\dots\dots \text{OK}$$

### 5.2.16 Perhitungan Chamber



Gambar 5.39 Perhitungan Chamber

- Titik (0,0)

$$X = 0, Y = 0 \quad ax^2 + bx + c = 0, c = 0$$

- Lendutan ijin ( $\delta$  ijin) =  $\frac{L}{500}$   

$$= \frac{8000}{500} = 16 \text{ cm}$$

- Titik maks (4000,16)

$$y = ax^2 + bx + c \quad 16 = a \cdot 4000^2 + 4000 \cdot b + c$$

$$16 = 4000^2 a + 4000 b \quad (1)$$

- Titik (8000,0)

$$y = ax^2 + bx + c \quad 0 = a \cdot 8000^2 + 8000 \cdot b + c$$

$$0 = 8000^2 a + 8000 b \quad (2)$$

$$b = -\frac{8000^2 a}{8000} = -8000a$$

Persamaan (1) dan (2) :

$$16 = 4000^2 a + 4000 b$$

$$16 = 4000^2 a + 4000 (-8000 a)$$

$$16 = 16000000 a - 32000000 a$$

$$16 = 16000000 a - 32000000 a$$

$$a = -\frac{16}{16.10^6}$$
$$= -10^{-6}$$

$$b = -8000(-10^{-6})$$
$$= 8.10^{-3}$$

$$y = -10^{-6}x^2 + 8.10^{-3}x$$

lendutan maksimum terjadi pada tengah bentang  $x = 4000$

$$x = 4000 \implies y = -10^{-6}x^2 + 8.10^{-3}x$$
$$y = -10^{-6}.(4000^2) + 8.10^{-3}(4000)$$
$$y = -10^{-6}.(4000^2) + 8.10^{-3}(4000)$$
$$= -16 + 32$$
$$= 16 \text{ cm}$$

### 5.3 PERHITUNGAN BANGUNAN BAWAH

Fungsi utama bangunan bawah jembatan adalah untuk menyalurkan semua beban yang bekerja pada bangunan atas ke tanah. Perencanaan bangunan bawah bertujuan untuk mendapatkan konstruksi bawah yang kuat, dan efisien. Perhitungan bangunan bawah meliputi :

- Perhitungan Pelat Injak
- Perhitungan Abutment
- Perhitungan Tiang Pancang

#### A. Data Tanah

Data dari hasil penyelidikan tanah, dapat disimpulkan bahwa :

- Pada Kedalaman  $\pm 0,00$  meter sampai dengan  $-1,00$  meter, lapisan tanah berupa jenis lanau kepasiran berwarna coklat tua.
- Kedalaman  $-1,00$  meter sampai  $-2,00$  lapisan tanah berupa jenis lanau kepasiran campur koral dan kerikil dengan nilai SPT  $> 60,00$ .
- Kedalaman  $-2,00$  meter sampai dengan  $-10,00$  meter lapisan tanah berupa jenis koral dengan nilai N SPT  $> 60,00$ .
- Muka air tanah terdapat pada kedalaman  $-1,00$  meter dari permukaan tanah setempat.
- Dipakai pesifikasi sebagai berikut :  
 $\gamma_1 = 1,566 \text{ gr/cm}^3$   
 $\theta_1 = 20^\circ$   
 $C_1 = 0,02 \text{ kg/cm}^2$

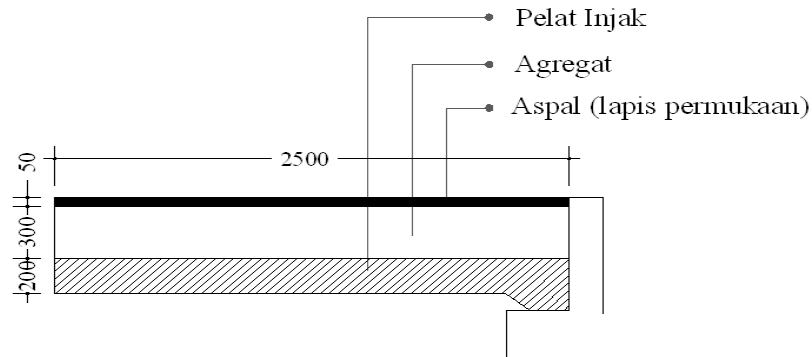
#### B. Spesifikasi Bahan

Adapun spesifikasi bahan yang dipakai antara lain :

- Abutment direncanakan menggunakan beton mutu  $f'c = 35 \text{ Mpa}$ .
- Pelat injak direncanakan menggunakan beton mutu  $f'c = 35 \text{ Mpa}$ .
- Pondasi tiang pancang direncanakan menggunakan beton mutu  $f'c = 40 \text{ Mpa}$ .
- Wingwall direncanakan menggunakan beton mutu  $f'c = 35 \text{ Mpa}$ .

- Tulangan yang digunakan :  
 $\varnothing 8$  dan  $\varnothing 10$  merupakan tulangan polos dengan mutu  $f_y = 240$  Mpa.  
D12, D14, D16, D25 adalah tulangan ulir dengan mutu  $f_y = 240$  Mpa

### 5.3.1 Perhitungan Pelat Injak



Gambar 5.40 Pelat Injak

#### A. Pembebanan Pelat Injak

##### Beban Mati

- Berat Aspal =  $0,05 \times 2200 \times 1 = 110$  kg/m
  - Berat Agregat =  $0,55 \times 1450 \times 1 = 797,5$  kg/m
  - Berat Air Hujan =  $0,05 \times 1000 \times 1 = 50$  kg/m
  - Berat Sendiri Pelat =  $0,20 \times 2500 \times 1 = 500$  kg/m
- berat total ( $q_{DL}$ ) =  $1457,5$  kg/m =  $14,575$  kN/m

$$\begin{aligned}
 M_{DL} &= 1/8 \times Q_{DL} \times L^2 \\
 &= 1/8 \times 14,575 \times 2,5^2 \\
 &= 11,387 \text{ kNm}
 \end{aligned}$$

##### Beban Hidup

Bentang jembatan = 80 m, maka :

$$\begin{aligned}
 q &= 1.1 (1 + 30/L) \text{ t/m'} \quad \text{untuk } L > 60 \text{ m} \\
 &= 1.1 (1 + 30/80) \text{ t/m'} = 1,65 \text{ t/m}
 \end{aligned}$$

$$\begin{aligned}\text{Beban terbagi rata } (q_{LL}) &= \frac{1,65}{2,75} \times 2,5 \\ &= 1,5 \text{ t/m} = 1500 \text{ kg/m} = 15 \text{ kN/m}\end{aligned}$$

$$\begin{aligned}M_{LL} &= 1/8 \times Q_{DL} \times L^2 \\ &= 1/8 \times 15 \times 2,5^2 \\ &= 11,718 \text{ kNm}\end{aligned}$$

$$M_{TOTAL} = 11,387 + 11,718 = 23,105 \text{ kNm}$$

### B. Penulangan Pelat Injak

$$f'_c = 35 \text{ Mpa}$$

$$f_y = 240 \text{ Mpa}$$

$$b = 100 \text{ cm}$$

$$\phi = 14 \text{ mm}$$

$$\begin{aligned}d &= h - p - \frac{1}{2} \phi \text{ tulangan} \\ &= 200 - 40 - 7 = 153 \text{ mm}\end{aligned}$$

$$\frac{Mu}{bd^2} = \rho \times 0,8 \times f_y \left( 1 - 0,588 \times \rho \times \frac{f_y}{f'_c} \right)$$

$$\frac{23,105}{1 \times 1,52^2} = \rho \times 0,8 \times 2400 \left( 1 - 0,588 \times \rho \times \frac{2400}{350} \right)$$

$$7741,44 \rho^2 - 1920 \rho + 10 = 0$$

$$\rho_1 = 0,242$$

$$\rho_2 = -1,03$$

$$\rho_{\min} = \frac{1,4}{f_y} = \frac{1,4}{240} = 0,005$$

$$\rho_{\max} = 0,75 \times \beta_1 \left( \frac{0,85 f'_c}{f_y} \times \frac{600}{600 + f_y} \right) \text{ dan } \beta_1 = 0,85$$

$$\rho_{\max} = 0,75 \times 0,85 \left( \frac{0,85 \times 350}{2400} \times \frac{600}{600 + 2400} \right) \text{ dan } \beta_1 = 0,85$$

$$\rho_{\max} = 0,015$$

Karena  $\rho_{\min} > \rho > \rho_{\max} \rightarrow$  dipakai  $\rho_{\min} = 0,005$

$$A_s = \rho \times b \times d = 0,005 \times 1000 \times 153 = 765 \text{ mm}^2$$

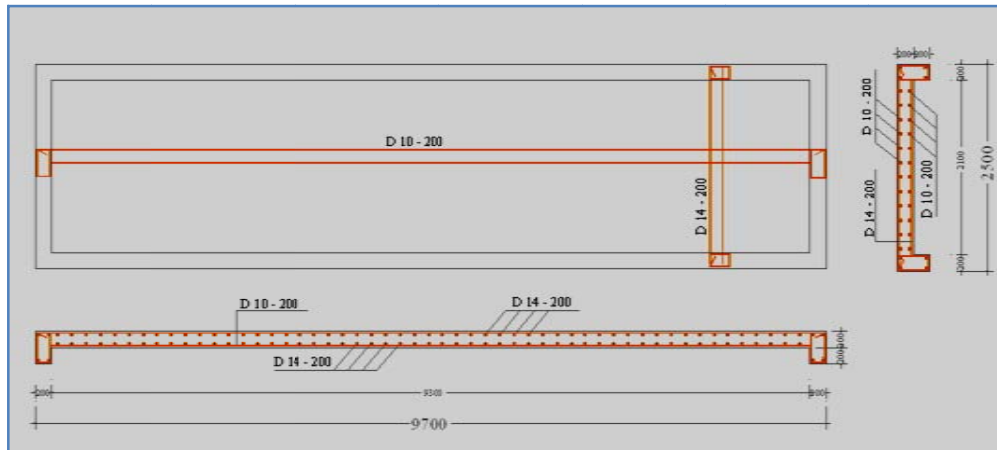
Dipakai tulangan  $\phi 14 - 200$  ( $A_s = 770 \text{ mm}^2$ )

Menurut SKSNI T15-1991-03 pasal 3.16.12, dalam arah tegak lurus terhadap tulangan utama harus disediakan tulangan pembagi (tegangan susut dan suhu) untuk  $f_y = 240 \text{ MPa}$

$$A_s = 0,0025 \times b \times d$$

$$A_s = 0,0025 \times 1000 \times 153 = 382,5 \text{ mm}^2$$

Digunakan tulangan bagi D10-200 ( $A_s = 393 \text{ mm}^2$ )



Gambar 5.41 Denah Penulangan Pelat Injak



### 5.3.2 Perhitungan Abutment

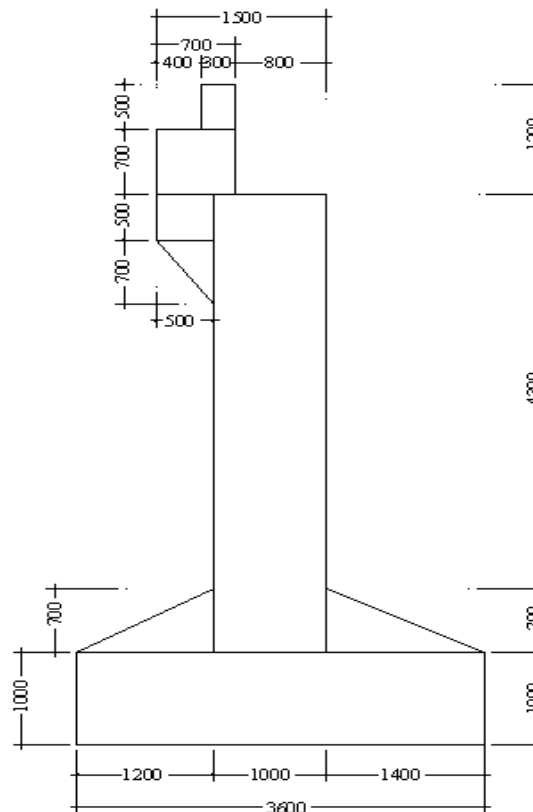
Gaya-gaya yang bekerja pada abutment antara lain:

Beban Mati meliputi:

- Berat sendiri
- Beban mati bangunan atas
- Gaya akibat beban vertikal tanah

Beban Hidup meliputi:

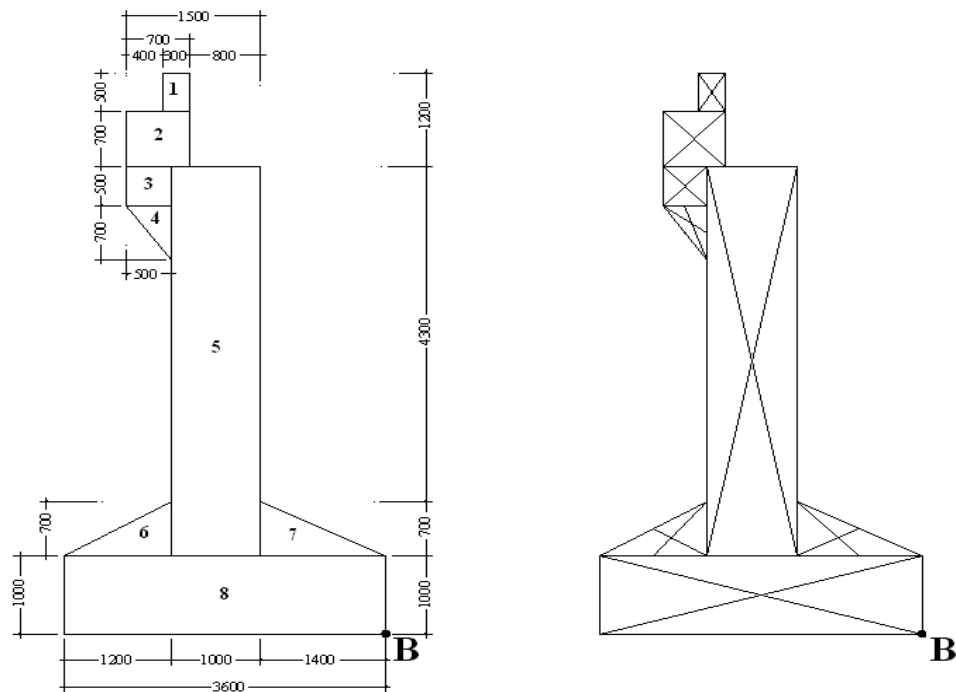
- Beban hidup bangunan atas
- Gaya horisontal akibat rem dan traksi
- Gaya akibat tekanan tanah aktif
- Gaya gesek tumpuan bergerak
- Gaya gempa
- Beban angin



Gambar 5.42 Dimensi Rencana Abutment

#### 5.3.2.1 Perhitungan Pembebanan Abutment

a. Berat Sendiri



Gambar 5.43 Perhitungan Berat Sendiri Abutment

Tabel 5.4 Perhitungan Berat Sendiri Abutment

No.	Luas m <sup>2</sup>	Panjang m	Volume m <sup>3</sup>	Berat Jenis T/m <sup>3</sup>	Berat T	Titik Berat (m)		Momen terhadap B (Tm)	
						X	Y	Mx	My
1	0.150	10.500	1.575	2.500	3.938	2.350	6.950	9.253	27.366
2	0.490	10.500	5.145	2.500	12.863	2.550	6.350	32.799	81.677
3	0.125	10.500	1.313	2.500	3.281	2.650	5.750	8.695	18.867
4	0.175	10.500	1.838	2.500	4.594	2.567	5.267	11.792	24.195
5	5.000	10.500	52.500	2.500	131.250	1.900	3.500	249.375	459.375
6	0.420	10.500	4.410	2.500	11.025	2.700	1.237	29.768	13.638
7	0.490	10.500	5.145	2.500	12.863	0.934	1.237	12.014	15.911
8	3.600	10.500	37.800	2.500	94.500	1.800	0.500	170.100	47.250
					274.313			523.796	688.279

Titik berat abutment :

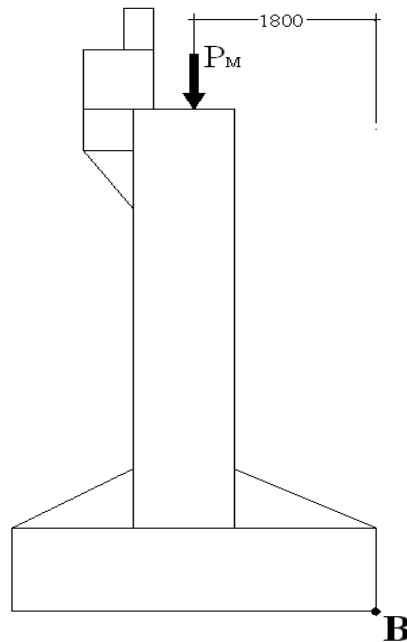
$$X = \frac{\sum Mx}{\sum Berat} = \frac{523,796}{274,313} = 1,909 \text{ m}$$

$$Y = \frac{\sum My}{\sum Berat} = \frac{688,279}{274,313} = 2,509 \text{ m}$$

Momen yang terjadi terhadap titik B :

$$MB = \sum Mx = 523,796 \text{ Tm}$$

#### b. Beban Mati Bangunan Atas



Gambar 5.44 Perhitungan Beban Akibat Konstruksi Atas

Berdasarkan hasil “SAP 2000 Versi 7 ” didapatkan reaksi diatas tumpuan sebesar 64,630 T, dimana satu buah abutment menerima 2 reaksi tumpuan dari 2 rangka baja. Sehingga abutment menerima beban mati sebesar :

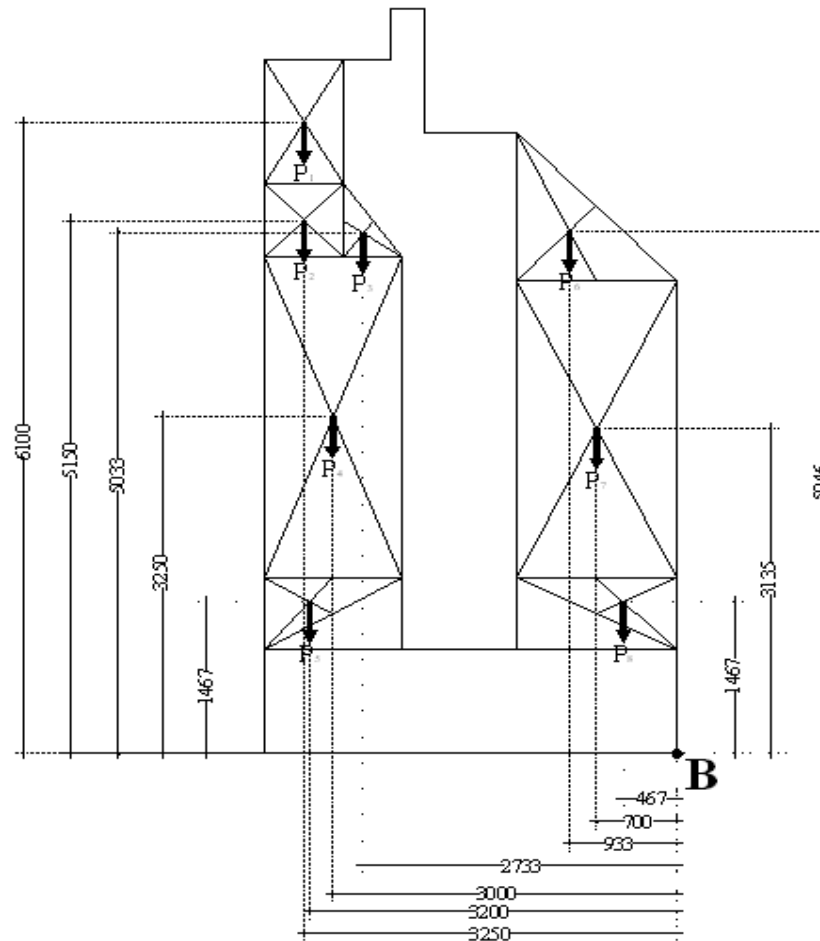
$$P_m = \text{Joint Reaction} = 64,630 \text{ T} \times 2 = 128,720 \text{ T.}$$

Lengan terhadap B ( $Y_a$ )= 1,8 m

Momen terhadap B:

$$\begin{aligned} M_B &= Y_a \times P_m \\ &= 1,8 \times 128,720 \\ &= 231,696 \text{ Tm} \end{aligned}$$

c. Gaya Akibat Beban Vertikal Tanah Timbunan



Gambar 5.45 Perhitungan Beban Akibat Beban Vertikal

Tabel 5.5 Perhitungan Berat Sendiri Abutment

No.	Luas m <sup>2</sup>	Panjang m	Volume m <sup>3</sup>	Berat Jenis T/m <sup>3</sup>	Berat T	Titik Berat		Momen terhadap B	
						X	Y	Mx	My
1	0.840	10.500	8.820	1.678	14.800	3.250	6.100	48.100	90.280
2	0.490	10.500	5.145	1.678	8.633	3.250	5.150	28.058	44.462
3	0.175	10.500	1.838	1.678	3.083	2.733	5.030	8.427	15.509
4	3.720	10.500	39.060	1.678	65.543	3.000	3.250	196.628	213.014
5	0.42	10.500	4.410	1.678	7.400	3.200	1.467	23.680	10.856
6	1.001	10.500	10.511	1.678	17.637	0.933	5.046	16.455	88.994
7	4.018	10.500	42.189	1.678	70.793	0.700	3.135	49.555	221.937
8	0.49	10.500	5.145	1.678	8.633	0.467	1.467	4.032	12.665
					196.522			374.935	697.716

Titik Berat terhadap B :

$$X = \frac{\sum Mx}{\sum Berat} = \frac{374,935}{196,522} = 1,907 \text{ m}$$

$$Y = \frac{\sum My}{\sum Berat} = \frac{697,716}{196,522} = 3,550 \text{ m}$$

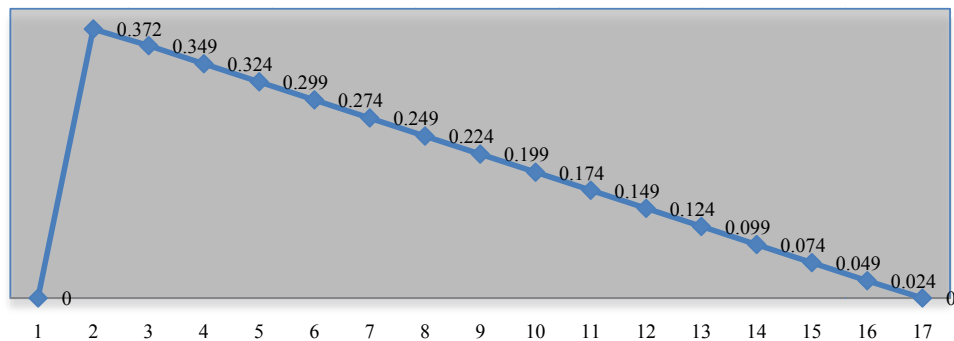
Momen terjadi terhadap B:

$$Ms = \sum Mx = 374,935 \text{ Tm}$$

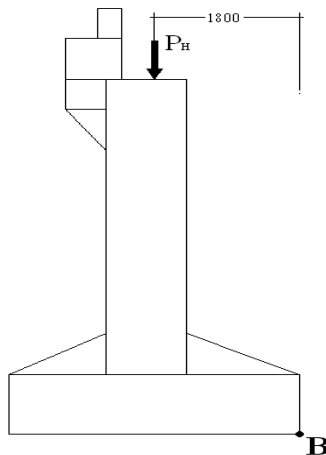
### Beban Hidup

#### a. Beban hidup bangunan atas

- Garis pengaruh S1



$$S = (0,5 \times 0,372 \times 80) \times 2,218 + (0,372 \times 15,722) = 38,851 \text{ t}$$



Gambar 5.46 Perhitungan Beban Akibat Beban hidup Bangunan Atas

$$Ph = R1V = R10V = 38,851 \text{ T}$$

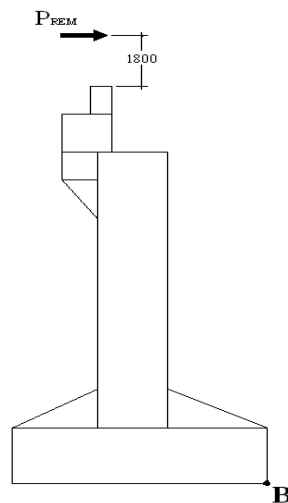
$$\text{Lengan terhadap B} = x = 1,80 \text{ m}$$

$$\text{Momen terhadap B} =$$

$$MB = Ph \times x = 38,851 \times 1,8 = 69,931 \text{ Tm}$$

#### b. Gaya rem

PPJJR : "Besarnya gaya rem =  $5\% \times \text{Beban "D"}$ , titik tangkap berada 1,8 m diatas permukaan lantai jembatan."



Gambar 5.47 Perhitungan Beban Akibat Gaya Rem dan Traksi

$$qL = 2,218 \text{ T/m}, PL = 15,722 \text{ T}$$

$$P_{REM} = 5\% \times (2,218 \times 80 + 15,722)$$

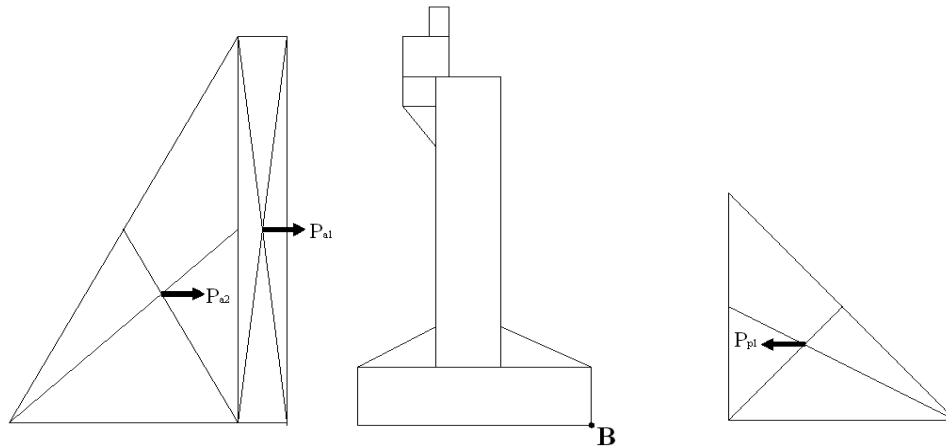
$$= 9,658 \text{ Ton}$$

$$\text{Lengan terhadap B} = y = 1,80 + 7,2 = 9,0 \text{ m}$$

$$\text{Momen terhadap B} =$$

$$MB = Ph \times y = 9,658 \times 9,0 = 86,922 \text{ Tm}$$

#### d. Gaya Akibat Tekanan Tanah Aktif



Gambar 5.48 Perhitungan Beban Akibat Tekanan Tanah aktif

Diketahui :

- Tanah Lapisan 1 (tanah urugan)

$$\gamma_1 = 2,0 \text{ gr/cm}^3$$

$$\phi_1 = 28^\circ$$

$$C_1 = 1 \text{ kg/cm}^2$$

$$H_1 = 6,5 \text{ m}$$

- Tanah lapisan 2 (tanah dasar)

$$\gamma_2 = 1,566 \text{ gr/cm}^3$$

$$\phi_2 = 20^\circ$$

$$C_2 = 0,02 \text{ kg/cm}^2$$

$$H_2 = 5,2 \text{ m}$$

- Koefisien tekanan tanah aktif:

$$K_{a1} = \tan^2 (45^\circ - \phi_1 / 2)$$

$$= \tan^2 (45^\circ - 28 / 2)$$

$$= 0,360$$

$$K_{a2} = \tan^2 (45^\circ - \phi_2 / 2)$$

$$= \tan^2 (45^\circ - 20 / 2)$$

$$= 0,490$$

- Koefisien tekanan tanah pasif:

$$\begin{aligned} K_p &= \tan^2 (45^\circ + \varphi / 2) \\ &= \tan^2 (45^\circ + 20^\circ / 2) \\ &= 2,039 \end{aligned}$$

- Perhitungan tinggi kritis dari timbunan:

$$H_{cr} = \frac{C_u \times N_c}{\gamma_{timbunan}}$$

$N_c$  : factor daya dukung untuk  $\Theta_2 = 11.52$

$$SF : \frac{1 \times 5,5}{2} = 2.25 < 3 \quad \dots\dots\dots ( \text{aman} )$$

Menurut pasal 1.4 P3JJR SKBI 1.3.28.1987, muatan lau lintas dapat diperhitungkan sebagai beban merata senilai dengan tekanan tanah setinggi:  $h = 60$  cm, jadi beban lalu lintas ( $q_x$ ) :

$$\begin{aligned} q_x &= \gamma_1 \times h \\ &= 2, \times 0,6 \\ &= 1,2 \text{ t/m}^2 \end{aligned}$$

$$\begin{aligned} q_1 &= q_{\text{pelat injak}} + q_x \\ &= 1,457 + 1,2 \\ &= 2,657 \text{ T/m}^2 \end{aligned}$$

Gaya tekanan tanah aktif:

$$\begin{aligned} P_1 &= K a_1 \times q_1 \times H_1 \times B \\ &= 0,36 \times 2,657 \times 6,5 \times 10,5 \\ &= 65,282 \text{ Ton} \end{aligned}$$

$$\begin{aligned} P_2 &= \frac{1}{2} \times \gamma_1 \times K a_1 \times H_1 \times B \\ &= \frac{1}{2} \times 2,657 \times 0,360 \times 6,5 \times 10,5 \\ &= 32,641 \text{ T} \end{aligned}$$



Gaya tekanan tanah pasif:

$$Pp1 = 1/2 \times Kp \times \gamma_1 \times D^2$$

$$= 1/2 \times 2,039 \times 1,566 \times 5,2^2$$

$$= 43,170 \text{ T}$$

$$F = P1 + P2 - Pp1$$

$$= 65,282 + 32,641 - 43,170$$

$$= 54,753 \text{ T}$$

$$Yf = \frac{\sum_{i=1}^4 (Ti \times Yi)}{f}$$

$$= \frac{(65,282 \times 3,25) + (32,641 \times 2,167) - (43,170 \times 1,733)}{54,753}$$

$$= 3,800 \text{ m}$$

Momen terhadap titik G:

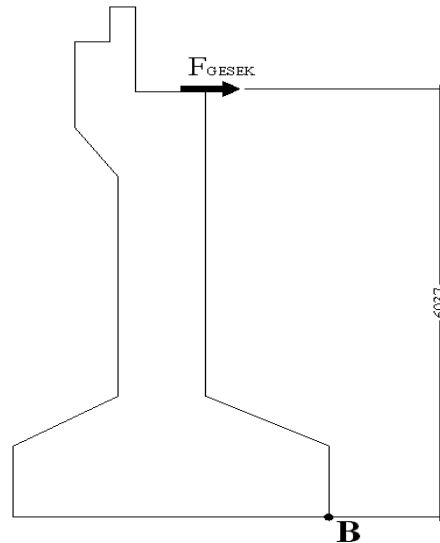
$$Mg = F \times Yf$$

$$= 54,753 \times 3,800$$

$$= 208,061 \text{ Tm}$$

**e. Gaya gesek akibat tumpuan–tumpuan bergerak:**

Menurut pasal 2.6 halaman 15 PPJIR SKBI 1.3.28.1987, gaya gesek yang timbul hanya ditinjau akibat beban mati saja, sedangkan besarnya ditentukan berdasarkan koefisien gesek, pada tumpuan yang bersangkutan.



Gambar 5.49 Gaya gesek tumpuan bergerak

$$F_{\text{ges}} = P_m \times C \quad \text{dimana:}$$

$f_{\text{ges}}$  = gaya gesek tumpuan bergerak

$P_m$  = beban mati konstruksi atas (T) =

$C$  = koefisien tumpuan karet dengan baja = 0,15

$$F_{\text{ges}} = 128,720 \times 0,15 = 19,308 \text{ T}$$

Lengan gaya terhadap titik G :

$$Y_{\text{ges}} = 6,037 \text{ m}$$

Momen terhadap titik G :

$$M_{\text{ges}} = F_{\text{ges}} \times Y_{\text{ges}}$$

$$= 19,308 \times 6,037$$

$$= 116,562 \text{ Tm}$$

**f. Gaya Gempa**

$$h = E \times M$$

dimana :

$h$  : gaya horisontal akibat gempa

$E$  : Koefesien gempa untuk daerah Jawa Tengah pada wilayah II = 0,14 ( Peraturan Muatan Untuk Jalan Raya no.12/1970)

$M$  : Muatan mati dari konstruksi yang ditinjau

- Gaya gempa terhadap berat sendiri abutment :

$$P_{BB} = 274,313 \text{ T}$$

$$Gh_{mB} = 274,313 \text{ T} \times 0,14 = 38,403 \text{ T}$$

$$Y_B = 2,509 \text{ m}$$

$$M = 38,403 \text{ T} \times 2,509 \text{ m} = 96,355 \text{ Tm}$$

- Gaya gempa terhadap bangunan atas :

$$P_{MB} = 128,720 \text{ T}$$

$$Gh_{mB} = 128,720 \text{ T} \times 0,14 = 18,020 \text{ T}$$

$$Y_{mB} = 6,037 \text{ m}$$

$$M = 18,020 \text{ T} \times 6,037 \text{ m} = 108,786 \text{ T}$$

- Gaya gempa terhadap tanah diatas abutment :

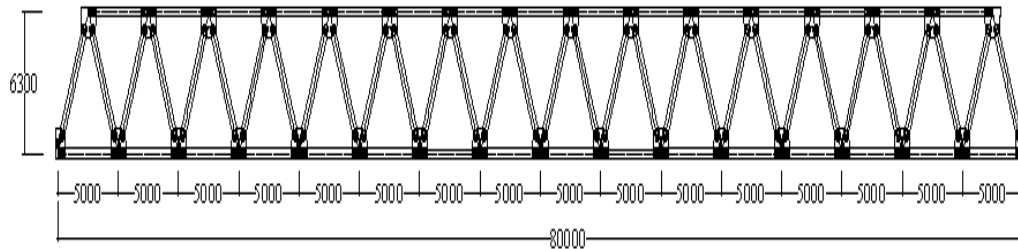
$$P_{TB} = 196,522 \text{ T}$$

$$Gh_{TB} = 196,522 \text{ T} \times 0,14 = 27,513 \text{ T}$$

$$Y_{TB} = 3,550 \text{ m}$$

$$M = 97,671 \text{ Tm}$$

### g. Gaya Angin



Gambar 5.50 Bidang Rangka Induk

Data teknis perencanaan pertambahan angin :

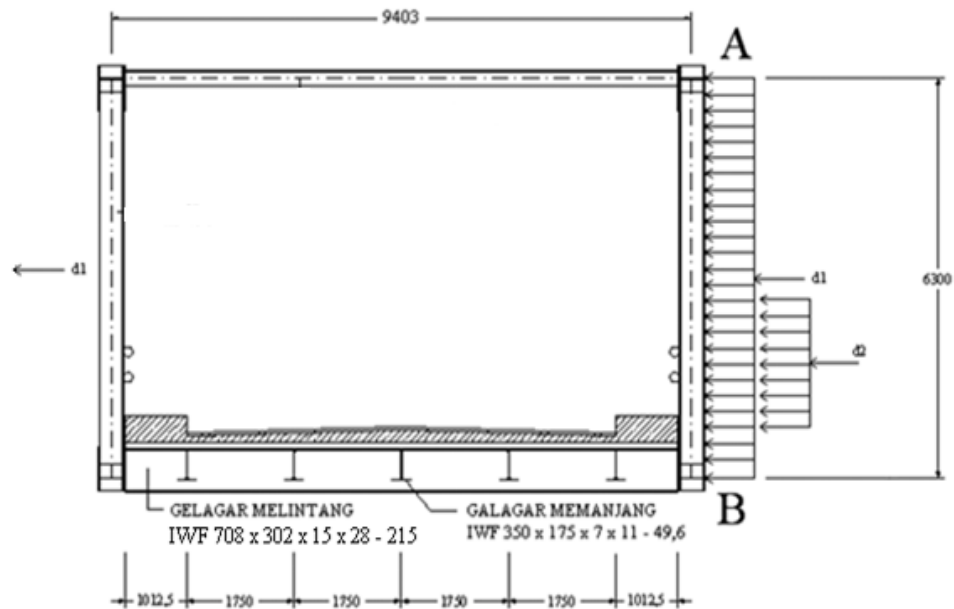
Tekanan angin :  $150 \text{ kg/m}^2$

Panjang sisi bawah jembatan :  $80 \text{ m}$

Panjang sisi atas jembatan :  $75 \text{ m}$

Tinggi jembatan :  $6,3 \text{ m}$

Luas bidang rangka utama :  $\left( \frac{80 + 75}{2} \right) \times 6,3 = 488,25 \text{ m}^2$



Gambar 5.51 Penyebaran Beban Angin

- Beban angin pada sisi rangka jembatan ( $d_1$ ) :

$$\begin{aligned} d1 &= 50\% \times ((30\% \times A)) \times w \\ &= 50\% \times ((30\% \times 488,25)) \times 150 \\ &= 10985,625 \text{ kg} \end{aligned}$$

- Beban angin pada muatan hidup setinggi 2 m ( $d_2$ ) :

$$\begin{aligned} d2 &= 100\% \times w \times L \times 2 \\ &= 100\% \times 150 \times 80 \times 2 \\ &= 24000 \text{ kg} \end{aligned}$$

- Beban angin pada sisi rangka jembatan ( $d_1$ ) :

$$\begin{aligned} d1 &= 50\% \times ((15\% \times A)) \times w \\ &= 50\% \times ((15\% \times 488,25)) \times 150 \\ &= 5492,812 \text{ kg} \end{aligned}$$

- Beban angin pada sisi rangka jembatan ( $s_1$ )

$$\begin{aligned} s1 &= \frac{1}{2} \times \text{tinggi jembatan} \\ &= \frac{1}{2} \times 6,30 \text{ m} \\ &= 3,15 \text{ m} \end{aligned}$$

- Beban angin pada muatan hidup seringgi 2 m ( $s_2$ )

Tinggi profil gelagar melintang ( $h_1$ ) : 70,8 cm ( 708x302x15x28-215 )

Tebal sayap gelagar melintang ( $h_2$ ) : 2,8 cm

Lebar profil rangka induk ( $h_3$ ) : 40,3 cm ( 428x407x20x35-283 )

Tebal plat lantai kendaraan ( $h_4$ ) : 20 cm

Tebal perkerasan ( $h_5$ ) : 5 cm

Tinggi bidang vertikal beban hidup ( $h_6$ ) : 200 cm

$$\begin{aligned} s_2 &= \left( h_1 - h_2 - \frac{h_3}{2} \right) + h_4 + h_5 + \frac{h_6}{2} \\ &= (70,8 - 2,3 - 20,35) + 20 + 5 + 100 \\ &= 173,35 \text{ cm} = 1,733 \text{ m} \end{aligned}$$

Lengan terhadap B :

$$Y_1 = Y_2 = 3,15 + 6,037 = 9,187 \text{ m}$$

$$Y_3 = 1,733 + 6,037 = 7,767 \text{ m}$$

Momen terhadap titik B :

$$\begin{aligned}M_B &= d_1 \times y_1 + d_2 \times y_2 + d_3 \times y_3 \\&= 10,985 \times 9,187 + 24 \times 9,187 + 5,492 \times 7,767 \\&= 364,063 \text{ Tm}\end{aligned}$$

**h. Gaya Tekanan Tanah Akibat Gempa Bumi**

$$PT_t = 54,753 \text{ T}$$

$$Ta = 54,753 \times 0,14 = 7,665 \text{ T}$$

$$YT_t = 3,800 \text{ m}$$

Momen terhadap titik B :

$$M_{TA} = 7,665 \times 3,800 = 29,127 \text{ Tm}$$

### 5.3.2.2 Kombinasi Pembebanan

Tabel 5.6 Kombinasi Pembebanan

Kombinasi Pembebanan dan Gaya	Tegangan yang digunakan dalam prosen terhadap tegangan izin keadaan elastis
I. $M + (H+K) + Ta + Tu$	100 %
II. $M + Ah + A + Ta + Gg + SR + Tm$	125 %
III. $Komb. I + Rm + Gg + A + SR + Tm + S$	140 %
IV. $M + Gh + Tag + Gg + AHg + Tu$	150 %
V. $M + P1$	130 %
VI. $M + (H+K) + Ta + S + Tb$	150%

Keterangan :

A : Beban angin

Ah : gaya akibat aliran dan hanyutan

Ahg : Gaya aliran dan hanyutan pada waktu gempa

Gg : gaya gesek pada tumpuan bergerak

Gh : gaya horizontal ekivalen akibat gempa bumi

H+K : beban hidup dengan kejut

M : beban mati

P1 : gaya – gaya pada waktu pelaksanaan

Rm : gaya rem

S : gaya setrifugal

SR : gaya akibat susut dan rangkak

Tm : gaya akibat perubahan suhu ( selain susut dan rangkak )

Ta : gaya tekanan tanah

Tag : gaya tekanan tanah akibat gempa bumi

Tb : gaya tumbuk

Tu : gaya angkat ( bouyancy )

Beban nominal : jumlah total beban

Beban ijin : beban nominal dibagi presentase terhadap tegangan ijin

Tabel 5.7 Kombinasi 1

Beban		Gaya		Jarak Lengan		Momen	
Jenis	Bagian	V	H	X	Y	MV	MH
M	abutment	274.313		1.909		523.796	
	Bangunan atas	128.720		1.800		231.696	
	Timbunan tanah	196.522		1.907		374.935	
H + K	Bangunan atas	38.851		1.800		69.931	
Ta			54.753		3.800		208.061
Tu							
Nominal		638.406	54.753			1200.358	208.061
ijin		638.406	54.753			1200.358	208.061

Tabel 5.8 Kombinasi 2

Beban		Gaya		Jarak Lengan		Momen	
Jenis	Bagian	V	H	Xo	Yo	MVo	MH
M	abutment	274.313		1.909		523.796	
	Bangunan atas	128.720		1.800		231.696	
	Timbunan tanah	196.522		1.907		374.935	
Ta			54.753		3.800		208.061
Ah							
Gg			19.308		6.037		116.522
A	Angin tekan		10.985		9.187		100.919
	Angin hisap		5.492		7.767		42.656
	muatan 2 m		24.000		9.187		220.488
SR							
Tm							
Nominal		599.555	114.538			1130.427	688.646
ijin		479.644	91.630			904.341	550.916



Tabel 5.9 Kombinasi 3

Beban		Gaya		Jarak Lengan		Momen	
Jenis	Bagian	V	H	X	Y	MV	MH
Komb. 1		638.406	54.753			1200.358	208.061
Rm			9.658		9.000		86.922
Gg			19.308		6.037		116.522
A	Angin tekan		10.985		9.187		100.919
	Angin hisap		5.492		7.767		42.656
	muatan 2 m		24.000		9.187		220.488
SR							
Tm							
S							
Nominal		638.406	124.960			1200.358	775.568
ijin		510.724	88.675			968.286	553.755

Tabel 5.10 Kombinasi 4

Beban		Gaya		Jarak Lengan		Momen	
Jenis	Bagian	V	H	X	Y	MV	MH
M	abutment	274.313		1.909		523.796	
	Bangunan atas	128.720		1.800		231.696	
	Timbunan tanah	196.522		1.907		374.935	
Gh	abutment		38.403		2.509		96.355
	Bangunan atas		18.020		6.037		108.786
	Timbunan tanah		27.513		3.550		97.671
T <sub>AG</sub>			7.665		3,800		29.127
Gg			19.308		6.037		116.522
Ahg							
T <sub>U</sub>							
Nominal		599.555	110.909			1130.427	448.461
ijin		359.733	66.545			678.256	269.076

Tabel 5.11 Kombinasi 5

Beban		Gaya		Jarak Lengan		Momen	
Jenis	Bagian	V	H	X	Y	MV	MH
M	abutment	274.313		1.909		523.796	
	Bangunan atas	128.720		1.800		231.696	
	Timbunan tanah	196.522		1.907		374.935	
P1							
Nominal		599.555				1130.427	
ijin		461.057				869.298	

Tabel 5.12 Kombinasi 6

Beban		Gaya		Jarak Lengan		Momen	
Jenis	Bagian	V	H	X	Y	MV	MH
M	abutment	274.313		1.909		523.796	
	Bangunan atas	128.720		1.800		231.696	
	Timbunan tanah	196.522		1.907		374.935	
H+K		38.851		1.800		69.931	
T <sub>A</sub>			54.753		3.800		208.061
STb							
Nominal		638.406	54.753			1200.358	208.061
ijin		425.816	36.520			800.638	138.776

### 5.3.2.3 Kontrol Stabilitas Abutment

Kestabilan konstruksi diperiksa terhadap kombinasi gaya dan muatan yang paling menentukan.

◦ Terhadap guling ( Fg ) =  $\frac{\sum MVg}{\sum MH} \geq SF$  , Dimana :

$\Sigma MV$  = jumlah momen vertical yang terjadi

$\Sigma MH$  = jumlah momen horisontal yang terjadi

SF = safety factor = 1,5

Tabel 5.13 kontrol terhadap guling

Komb.	MV (Tm)	MH (Tm)	F	SF	Ket
I	1200.358	208.061	5.769	1.5	aman
II	1130.427	688.466	1.642	1.5	aman
III	1200.358	775.568	1.548	1.5	aman
IV	1130.427	448.461	2.521	1.5	aman
V	1130.427	-	-	1.5	aman
VI	1200.358	208.061	5.769	1.5	aman

◦ Terhadap Geser (FS) =  $\frac{\sum V \times \tan \delta + Ca \times B}{\sum H}$  , Dimana :

Tan  $\delta$  = faktor geser tanah antara tanah dan dasar tembok (Buku Teknik Sipil)

= 0,45 (Beton dengan tanah lempung padat dan pasir gravelan padat)

Ca = adhesi antara tanah dan dasar tembok = 0

B = lebar dasar pondasi = 3,600 meter

Tabel 5.14 kontrol terhadap geser

Komb.	V (Tm)	Tan $\delta$	Ca	B (m)	H (m)	FS	SF	Ket
I	638.406	0.45	0	3.600	54.753	5.247	1.5	aman
II	599.555	0.45	0	3.600	114.538	2.356	1.5	aman
III	638.406	0.45	0	3.600	124.96	2.299	1.5	aman
IV	599.555	0.45	0	3.600	110.909	2.433	1.5	aman
V	599.555	0.45	0	3.600	-	-	1.5	aman
VI	638.406	0.45	0	3.600	54.753	5.247	1.5	aman

$$\circ \text{ Terhadap eksentrisitas ( } e \text{ )} = \frac{B}{2} - \frac{\sum M_v - \sum M_h}{\sum V} < \frac{B}{6} = \frac{3,60}{6} = 0,600 \text{ m}$$

Tabel 5.15 kontrol terhadap eksentrisitas

Komb.	0,5 B (m)	MV (Tm)	MH (Tm)	V (Tm)	e (m)	1/6 B (m)	Ket
I	1.800	1200.358	208.061	638.406	-0.406	0.600	aman
II	1.800	1130.427	688.466	599.555	-1.234	0.600	aman
III	1.800	1200.358	775.568	638.406	-1.295	0.600	aman
IV	1.800	1130.427	448.461	599.555	-0.833	0.600	aman
V	1.800	1130.427	-	599.555	-0.085	0.600	aman
VI	1.800	1200.358	208.061	638.406	-0.406	0.600	aman

◦ **Terhadap Daya Dukung Tanah**

Kapasitas dukung tanah dasar (bearing capacity) dipengaruhi oleh parameter  $\phi, c, \text{ dan } \gamma$ . Besarnya kapasitas dukung tanah dasar dapat dihitung dengan metode Terzaghi, yaitu :

$$Q_{ult} = A_p \cdot (c \cdot N_c (1 + 0,3B / L) + \gamma \cdot D_f \cdot N_q + 0,5 \cdot \gamma \cdot B \cdot N_\gamma \cdot (1 - 0,2B / L))$$

dimana :

$Q_{ult}$  = daya dukung ultimate tanah dasar (t/m<sup>2</sup>)

$c$  = kohesi tanah dasar (t/m<sup>2</sup>)

$\gamma$  = berat isi tanah dasar (t/m<sup>3</sup>)

$B=D$  = lebar pondasi (meter)

$D_f$  = kedalaman pondasi (meter)

$N_\gamma, N_q, N_c$  = faktor daya dukung Terzaghi

$A_p$  = luas dasar pondasi

$B$  = lebar pondasi

$L$  = panjang pondasi

Tabel 5.16 Nilai-nilai daya dukung Terzaghi

$\varphi$	Keruntuhan Geser Umum			Keruntuhan Geser Lokal		
	$N_c$	$N_q$	$N_\gamma$	$N'_c$	$N'_q$	$N'_\gamma$
0	5,7	1,0	0,0	5,7	1,0	0,0
5	7,3	1,6	0,5	6,7	1,4	0,2
10	9,6	2,7	1,2	8,0	1,9	0,5
15	12,9	4,4	2,5	9,7	2,7	0,9
20	17,7	7,4	5,0	11,8	3,9	1,7
25	25,1	12,7	9,7	14,8	5,6	3,2
30	37,2	22,5	19,7	19,0	8,3	5,7
34	52,6	36,5	35,0	23,7	11,7	9,0
35	57,8	41,4	42,4	25,2	12,6	10,1
40	95,7	81,3	100,4	34,9	20,5	18,8
45	172,3	173,3	297,5	51,2	35,1	37,7
48	258,3	287,9	780,1	66,8	50,5	60,4
50	347,6	415,3	1153,2	81,3	65,6	87,1

Berdasarkan data tanah :

$$\gamma_2 = 1.566 \text{ gr/cm}^3, c_2 = 0.02 \text{ kg/cm}^2, \phi_2 = 20^\circ$$

$$Q_{ult} = (c \cdot N_c (1 + 0,3B/L) + \gamma \cdot D_f \cdot N_q + 0,5 \cdot \gamma \cdot B \cdot N_\gamma \cdot (1 - 0,2B/L))$$

$$= (0,02 \times 172,3 (1 + 0,3 \times 3,60 / 10,50) + 1,566 \times 10^{-3} \times 400 \times 7,4 + 0,5 \times 1,566 \times 10^{-3} \times 3,60 \times 5 (1 - 0,2 \times 3,60 / 10,50))$$

$$= 7,649 \text{ kg/cm}^2 = 76,490 \text{ T/m}^2$$

$$\sigma_{all} = (1/3) \cdot Q_{ult}$$

$$\sigma_{all} = (1/3) \cdot 76,490 = 25,496 \text{ Ton}$$

$$\sigma = \frac{\sum V}{A} \pm \frac{\sum MV + \sum MH}{W} \leq \sigma_{all}$$

Dimana :

SF = safety factor 1.5 ~ 3

B = lebar abutment = 3.60 meter

L = panjang abutment = 10.50 meter

A =  $3,60 \times 10,50 = 54.59 \text{ m}^2$

$$W = 1/6 * L * B^2 = 1/6 * 10.50 * 3,60^2 = 22,680 \text{ m}^3$$

V = gaya vertical ( ton )

MV = jumlah momen vertical yang terjadi

MH = jumlah momen vertical vertical yang terjadi

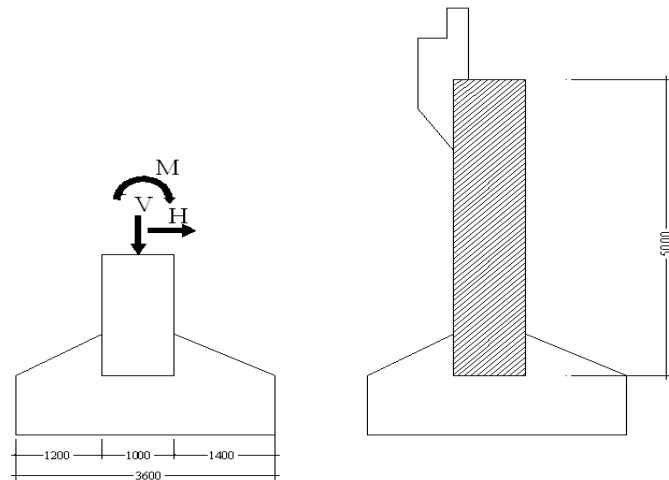
Tabel 5.17 kontrol terhadap daya dukung tanah

Komb	$\Sigma V$ (T)	$\Sigma MV + \Sigma MH$ (Tm)	A (m <sup>2</sup> )	W (m <sup>3</sup> )	$\sigma_{ALL}$ (Tm)	$\sigma_{MIN}$ (T)	$\sigma_{MAX}$ (T)	Ket
I	638.406	1408.419	54.59	22.680	25.496	-50.405	73.794	tdk aman
II	599.555	1818.893	54.59	22.680	25.496	-69.215	91.181	tdk aman
III	638.406	1975.926	54.59	22.680	25.496	-75.427	98.817	tdk aman
IV	599.555	1578.888	54.59	22.680	25.496	-58.633	80.599	tdk aman
V	599.555	1130.427	54.59	22.680	25.496	-38.860	60.825	tdk aman
VI	638.406	1408.419	54.59	22.680	25.496	-50.405	73.794	tdk aman

Karena tinjauan stabilitas abutment tidak aman, maka dipasang pondasi tiang pancang untuk menanggulangi kegagalan konstruksi.

### 5.3.2.4 Penulangan Abutment

#### a. Penulangan Badan Abutment



Gambar 5.52 Penulangan Badan Abutment

Beban yang digunakan dalam penulangan badan abutment diambil dari kombinasi pembebanan yang menghasilkan beban dan momen terbesar yaitu kombinasi pembebanan III.

Tabel 5.18 Kombinasi Pembebanan Maksimum

Beban		Gaya		Jarak Lengan		Momen	
Jenis	Bagian	V	H	X	Y	MV	MH
Komb. 1		638.406	54.753			1200.358	208.061
Rm			9.658		9.000		86.922
Gg			19.308		6.037		116.522
A	Angin tekan		10.985		9.187		100.919
	Angin hisap		5.492		7.767		42.656
	muatan 2 m		24.000		9.187		220.488
SR							
Tm							
S							
Nominal		638.406	124.960			1200.358	775.568
ijin		510.724	88.675			968.286	553.755

- Data Teknis Perencanaan :

$$f'_c = 35 \text{ MPa}$$

$$f_y = 240 \text{ Mpa}$$

$$A_g = \text{luas penampang}$$

$$= 1000 \times 1000$$

$$= 10^6 \text{ mm}^2$$

$$H_t = \text{tinggi badan} = 5000 \text{ mm}$$

$$b = 1000 \text{ mm (tiap meter lebar abutment)}$$

$$h = 1000 \text{ mm}$$

Diameter tulangan utama dipakai D20, dan tulangan pembagi dipakai D16, sehingga :

$$d' = h - (50 + 16 + \frac{1}{2} 20) = 1000 - (50 + 16 + 10) = 924 \text{ mm}$$

$$\Phi = 0,65$$

$$\frac{P_u}{\phi \times A_g \times 0,81 \times f'_c} = \frac{638406}{0,65 \times 10^6 \times 0,81 \times 35} = 0,035$$

$$e_t = \frac{M_u}{P_u} = \frac{1200,358}{638,406} = 1,880 \text{ m} = 1880 \text{ mm}$$

$$\frac{e_t}{h} = \frac{1880}{1000} = 1,880$$

$$\frac{P_u}{\phi \times A_g \times 0,81 \times f'_c} \times \frac{e_t}{h} = 0,035 \times 1,880 = 0,0658$$

Dari perhitungan diatas dipakai grafik 6.1.d (*Grafik dan Tabel Perhitungan Beton Bertulang halaman 86*)

$$r = 0,01$$

$$f'_c = 35 \text{ maka } \beta = 1,33$$

$$\rho = r \times \beta = 0,01 \times 1,33 = 0,0133$$

- Tulangan Pokok

$$A_{s_{tot}} = \rho \times A_g$$

$$= 0,0133 \times 10^6 = 13300 \text{ mm}^2$$



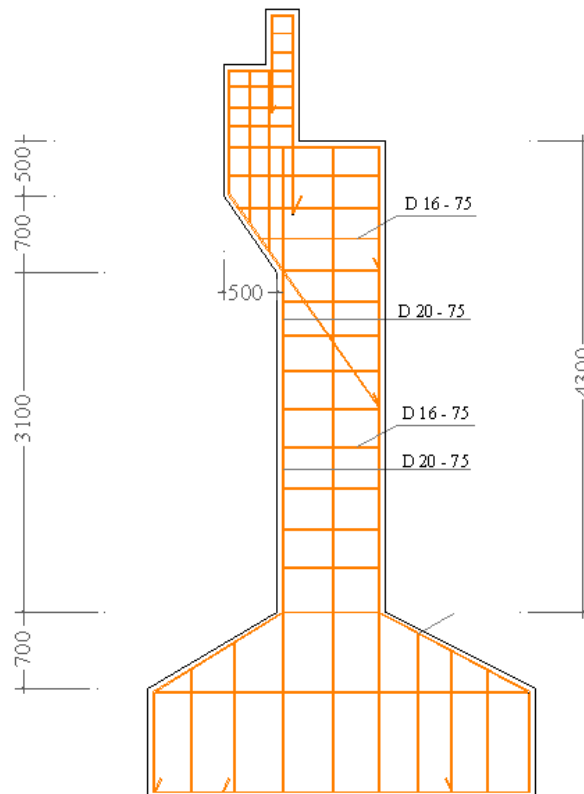
$$A_{s_{kiri}} = A_{s_{kanan}} = 0,25 A_{s_{total}} = 3325 \text{ mm}^2$$

Dipakai tulangan rangkap D20 – 75 ( $A_{st} = 4189 \text{ mm}^2$ )

- Tulangan bagi

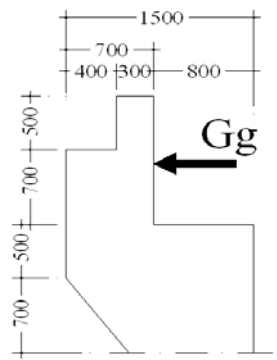
Diambil sebesar 20 % dari tulangan utama =  $2660 \text{ mm}^2$

Dipakai tulangan rangkap D16 – 75 ( $A_s = 2681 \text{ mm}^2$ )



Gambar 5.53 Penulangan Badan Abutment

## b. Penulangan Kepala Abutment



Gambar 5.54 Dimensi Kepala Abutment

- Gaya horisontal gempa ( $G_g$ ) :

Gaya gempa terhadap berat sendiri abutment :

$$P_{BB} = 274,313 \text{ T}$$

$$G_{mB} = 274,313 \text{ T} \times 0,14 = 38,403 \text{ T}$$

$$Y_B = 2,509 \text{ m}$$

$$M = 38,403 \text{ T} \times 2,509 \text{ m} = 96,355 \text{ Tm}$$

Gaya gempa terhadap bangunan atas :

$$P_{MB} = 128,720 \text{ T}$$

$$G_{mB} = 128,720 \text{ T} \times 0,14 = 18,020 \text{ T}$$

$$Y_{mB} = 6,037 \text{ m}$$

$$M = 18,020 \text{ T} \times 6,037 \text{ m} = 108,786 \text{ T}$$

$$M_t = 96,355 + 108,786 = 205,141 \text{ T}$$

- Penulangan Kepala Abutment

$$f'_c = 35 \text{ MPa}$$

$$f_y = 240 \text{ Mpa}$$

$$b = 300 \text{ mm}$$

$$h = 1000 \text{ mm}$$

Diameter tulangan utama dipakai D20, dan tulangan pembagi dipakai D10, sehingga :

$$d' = h - (50 + 10 + \frac{1}{2} 20) = 1000 - (50 + 10 + 10) = 930 \text{ mm}$$

$$\Phi = 0,65$$

$$\frac{Mu}{bd^2} = \rho \times 0,8 \times f_y \left( 1 - 0,588 \times \rho \times \frac{f_y}{f'_c} \right)$$

$$\frac{205141}{1 \times 0,930^2} = \rho \times 0,8 \times 2400 \left( 1 - 0,588 \times \rho \times \frac{2400}{350} \right)$$

$$7741,44 \rho^2 - 1920 \rho + 15615,337 = 0, \quad \rho = 1,538$$

$$\rho_{\min} = \frac{1,4}{f_y} = \frac{1,4}{240} = 0,0058$$

$$\rho_{\max} = 0,75 \times \beta_1 \left( \frac{0,85 f'_c}{f_y} \times \frac{600}{600 + f_y} \right) \text{ dan } \beta_1 = 0,85$$

$$\rho_{\max} = 0,75 \times 0,85 \left( \frac{0,85 \times 350}{2400} \times \frac{600}{600 + 2400} \right) \text{ dan } \beta_1 = 0,85; \rho_{\max} = 0,015$$

dipakai  $\rho_{\min} = 0,0058$

- Tulangan Pokok

$$A_{s_{\text{total}}} = \rho \times b \times d = 0,0058 \times 300 \times 924 = 1607,776 \text{ mm}^2$$

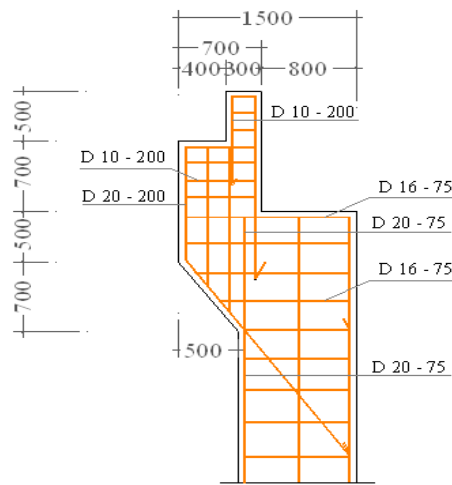
$$A_{s_{\text{kiri}}} = A_{s_{\text{kanan}}} = 0,5 A_{s_{\text{total}}} = 803,88 \text{ mm}^2$$

Dipakai tulangan rangkap D20 – 200 ( $A_s = 1571 \text{ mm}^2$ )

- Tulangan bagi

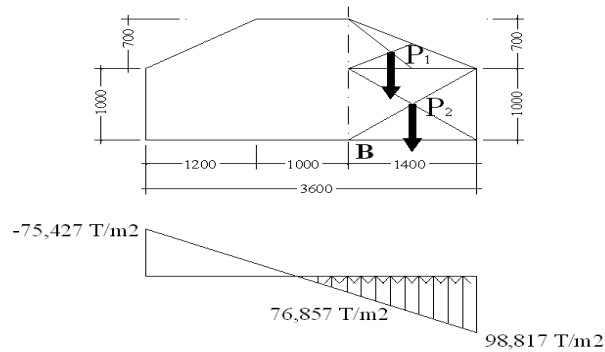
Diambil sebesar 20 % dari tulangan utama =  $321,552 \text{ mm}^2$

Dipakai tulangan rangkap D10 – 200 ( $A_s = 393 \text{ mm}^2$ )



Gambar 5.55 Penulangan Kepala Abutment

c. Penulangan Poer



Gambar 5.56 Pembebanan Poer

$$P1 = 0,5 \times 1,4 \times 0,7 \times 2,5 \times 1 = 1,225 \text{ T}$$

$$P2 = 1,8 \times 1,4 \times 2,5 \times 1 = 6,3 \text{ T}$$

Momen yang terjadi pada potongan A:

$$\begin{aligned} M_B &= P_{maks} \times 1,044 - P1 \times 0,6 - P2 \times 0,9 \\ &= 98,817 \times 1,40 - 1,225 \times 0,467 - 6,30 \times 0,70 \\ &= 133,361 \text{ Tm} \end{aligned}$$

Direncanakan :

$$f'_c = 35 \text{ MPa}$$

$$f_y = 240 \text{ Mpa}$$

$$b = 1400 \text{ mm}$$

$$h = 1000 \text{ mm}$$

Diameter tulangan utama dipakai D20, dan tulangan pembagi dipakai D16, sehingga :

$$d' = h - (50 + 16 + \frac{1}{2} 20) = 1000 - (50 + 16 + 10) = 924 \text{ mm}$$

$$\Phi = 0,65$$

$$\frac{Mu}{bd^2} = \rho \times 0,8 \times f_y \left( 1 - 0,588 \times \rho \times \frac{f_y}{f'_c} \right)$$

$$\frac{133361}{1 \times 0,924^2} = \rho \times 0,8 \times 2400 \left(1 - 0,588 \times \rho \times \frac{2400}{350}\right)$$

$$7741,44 \rho^2 - 1920 \rho + 156201,392 = 0, \quad \rho = 1,638$$

$$\rho_{\min} = \frac{1,4}{f_y} = \frac{1,4}{240} = 0,0058$$

$$\rho_{\max} = 0,75 \times \beta_1 \left( \frac{0,85 f'_c}{f_y} \times \frac{600}{600 + f_y} \right) \text{ dan } \beta_1 = 0,85$$

$$\rho_{\max} = 0,75 \times 0,85 \left( \frac{0,85 \times 350}{2400} \times \frac{600}{600 + 2400} \right) \text{ dan } \beta_1 = 0,85$$

$$\rho_{\max} = 0,015$$

dipakai  $\rho_{\min} = 0,0058$

- Tulangan Pokok

$$A_{s_{\text{total}}} = \rho \times b \times d = 0,0058 \times 1400 \times 924 = 7502,88 \text{ mm}^2$$

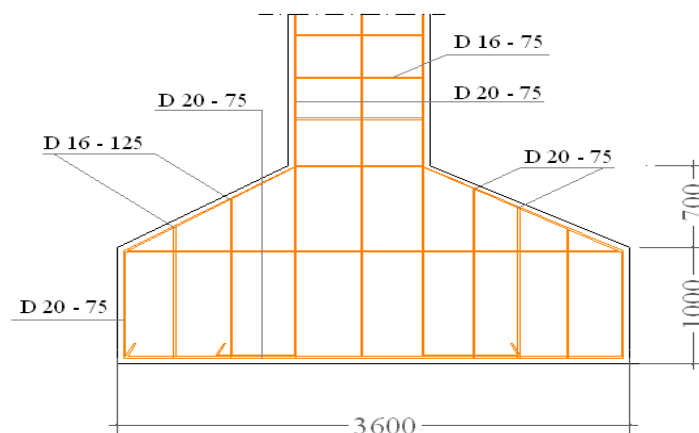
$$A_{s_{\text{kiri}}} = A_{s_{\text{kanan}}} = 0,5 A_{s_{\text{total}}} = 3751,44 \text{ mm}^2$$

Dipakai tulangan rangkap D20 – 75 ( $A_s = 4189 \text{ mm}^2$ )

- Tulangan bagi

Diambil sebesar 20 % dari tulangan utama =  $1500,576 \text{ mm}^2$

Dipakai tulangan rangkap D16 – 125 ( $A_s = 1608 \text{ mm}^2$ )



Gambar 5.57 Penulangan Poer

### 5.3.3 Perhitungan Pondasi Tiang Pancang

Perencanaan beban maksimal( $P_{\max}$ ) yang mampu ditahan tiang pancang ditinjau terhadap empat kombinasi pembebanan terhadap titik pusat tiang pancang.

Pondasi menggunakan tiang pancang dari beton dengan spesifikasi :

Ø tiang	= 35 cm
Luas penampang ( A )	= $\frac{1}{4} \pi D^2 = 961,625 \text{ cm}^2$
Keliling penampang tiang	= $\pi D = 109,90 \text{ cm}$
Panjang tiang pancang	= 14 meter
Berat permeter tiang	= $961,625 * 2500 * 10^{-4} = 240,41 \text{ kg/m}$
Berat tiang pancang	= $240,41 * 14 = 4808,2 \text{ kg} = 4,808 \text{ ton}$

$$P_{\max} = \frac{PV}{n} + \frac{M * X_{\max}}{n y * \sum X^2}$$

Dimana :

$P_{\max}$  = beban maksimum yang diterima tiang pancang

PV = beban vertical ( normal )

M = jumlah momen yang bekerja pada titik berat tiang pancang

$X_{\max}$  = jarak terjauh tiang kepusat berat kelompok tiang = 1,6 m

n = jumlah pondasi tiang pancang = 14 bh

$n y$  = jumlah pondasi tiang pancang dalam satu baris arah tegak lurus bidang  
momen = 7 bh

$$\sum X^2 = 1,6^2 = 2,56 \text{ m}$$

Gaya maksimum yang dipikul tiang pancang

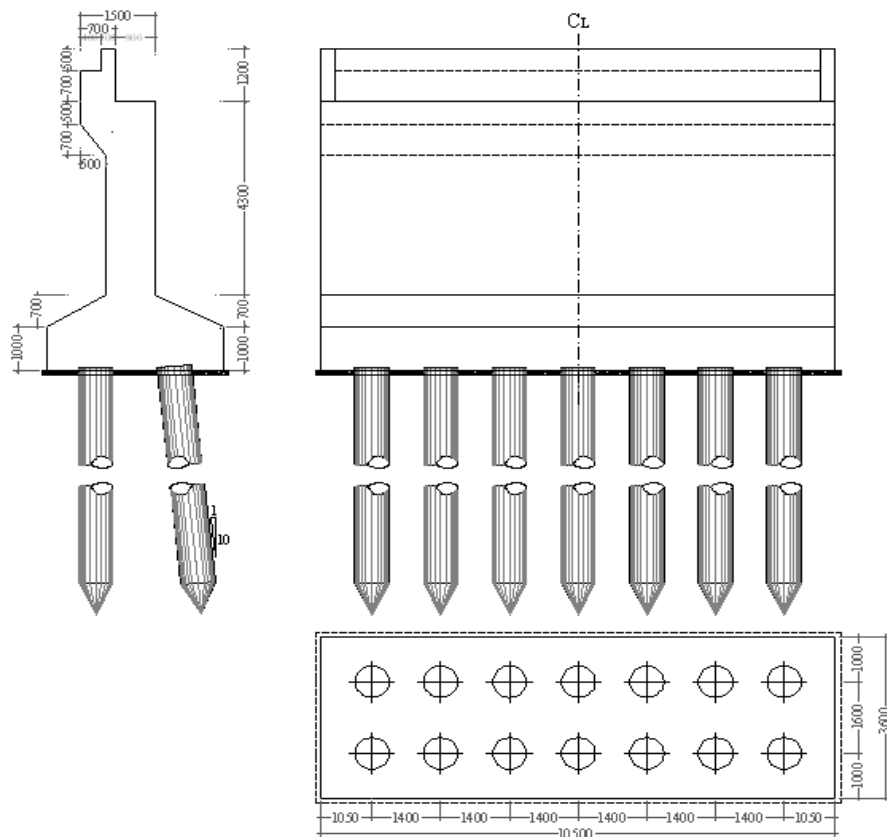
$$P = \frac{PV}{n} + \frac{M * X_{\max}}{n y * \sum X^2}$$

Tabel 5.19 Perhitungan gaya maksimum dan minimum

Komb	PV (T)	n	M <sub>V</sub> (Tm)	X <sub>MAX</sub> (m)	ΣX <sup>2</sup> (m <sup>2</sup> )	ny	P <sub>Max</sub> (T)	P <sub>Min</sub> (T)
I	638.406	14	1200.358	1.60	2.56	7	152.775	-61.574
II	599.555	14	1130.427	1.60	2.56	7	143.756	-58.106
III	638.406	14	1200.358	1.60	2.56	7	152.775	-61.574
IV	599.555	14	1130.427	1.60	2.56	7	143.756	-58.106
V	599.555	14	1130.427	1.60	2.56	7	143.756	-58.106
VI	638.406	14	1200.358	1.60	2.56	7	152.775	-61.574

Dari table perhitungan diperoleh bahwa Pmaks terjadi pada kombinasi III sebesar 152,775 T. Maka daya dukung tanah haru lebih besar dari Pmaks tersebut.

### 5.3.3.1 Perhitungan Daya Dukung Tiang Pancang



Gambar 5.58 Denah Rencana Pondasi Tiang Pancang Pada Abument

### 1. Daya dukung tiang individu

Tinjauan spesifikasi tiang pancang berdasarkan :

#### a. Kekuatan bahan tiang

Mutu beton : K – 600

$$\sigma_b : \frac{1}{3} * 600 = 200 \text{ kg/cm}^2$$

$$P_{\text{tiang}} : \sigma_b * A_{\text{tiang}} = 200 * 961,625 = 192,325 \text{ ton}$$

#### b. Daya dukung tanah

- Rumus Umum

$$P_{\text{ult}} = \frac{K_b * q_c * A + K_s * JHP * O}{SF}$$

$$K_b = 0,75$$

$$K_s = 0,5 - 0,75$$



$$A = \frac{1}{4} \pi D^2 = 961,625 \text{ cm}^2 = 0,096125 \text{ m}^2$$

$$O = \pi D = 109,90 \text{ cm} = 1,099 \text{ m}$$

Dari data tanah diperoleh :

$$q_c = \frac{1}{2} (q_{cu} + q_{cb})$$

$$q_{cu} = q_c \text{ rata - rata } 3,5 D \text{ dibawah ujung tiang} = 206,667$$

$$q_{cb} = q_c \text{ rata - rata } 8 D \text{ diatas ujung tiang} = 145$$

$$q_c = \frac{1}{2} (206,667 + 145) = 175,835$$

$$JHP = 1300$$

$$P_{ult} = \frac{0,75 * 175,835 * 0,096125 + 0,75 * 1300 * 1,099}{3}$$

$$= 361,400 \text{ T}$$

- Rumus Trofimankof

$$P_{ult} = \frac{K_b * q_c * A + JHP / D * O}{SF}$$

$$D = 1,5$$

$$P_{ult} = \frac{0,75 * 175,835 * 0,096125 + (1300 / 1,5) * 1,099}{3}$$

$$= 321,708 \text{ T}$$

- Rumus S.P.T (Standard Penetration Test) untuk tanah pasir

$$Q_u = 40 \times N \times A_b + 0,2 \times N \times A_s$$

$$Q_u = \text{Daya Dukung Batas Tiang (ton)}$$

$$N = \text{Nilai rata - rata SPT sepanjang Tiang} = 60$$

$$A_s = \text{Luas Total Selimut Tiang (m}^2\text{)}$$

$$= \text{kell } O \times H \text{ tiang} = 1,099 \times 14 = 15,386 \text{ m}^2$$

$$A_b = \text{luas penampang ujung tiang (m}^2\text{)}$$

$$= \pi \cdot r (S + r) = 3,14 \times 0,175 (0,789 + 0,175) = 0,529 \text{ m}^2$$

$$Q_u = 40 \times 60 \times 0,529 + 0,2 \times 60 \times 15,386$$

$$= 1454,232 \text{ T}$$

$$Q_a = Q_u / SF = 1454,232 / 3 = 484,744 \text{ ton.}$$

- Rumus Meyerhof

$$Q_{ult} = 40 \times N \times A_b + \frac{N_x \times A_s}{5}$$

N = Nilai SPT ujung Tiang = 60

A<sub>s</sub> = Luas Total Selimut Tiang (m<sup>2</sup>)

$$= \text{kell O} \times \text{H tiang} = 1,099 \times 14 = 15,386 \text{ m}^2$$

A<sub>b</sub> = luas penampang ujung tiang (m<sup>2</sup>) = 0,529 m<sup>2</sup>

N<sub>x</sub> = nilai rata rata SPT = 60

$$Q_{ult} = 40 \times 60 \times 0,529 + \frac{60 \times 15,386}{5}$$

$$Q_u = 1454,232 \text{ T}$$

$$Q_a = Q_u / SF = 1454,232 / 3 = 484,744 \text{ ton.}$$

◦ Rumus begemann

$$\begin{aligned} P_{ult} &= \frac{q_c \times A}{3} + \frac{JHP * O}{5} \\ &= \frac{175,835 \times 0,096125}{3} + \frac{1300 * 1,099}{5} \\ &= 291,374 \text{ T} \end{aligned}$$

Dari tinjauan diatas dipakai nilai daya dukung terkecil = 291,374 T

## 2. Daya Dukung Kelompok Tiang

Berdasarkan perumusan dari *converse labarre* :

$$\text{Eff} = 1 - \theta \left[ \frac{(n-1)m + (m-1)n}{90 * m * n} \right]$$

Dimana :

m = jumlah tiang dalam baris y = 7

n = jumlah baris = 2

θ = arc tan θ (D/S) = arc tan (0,35/1,6) = 12,298°

D = diameter tiang = 35 cm

S = jarak antar tiang = 160 cm

$$\text{Eff} = 1 - 12,298 \left[ \frac{(2-1)7 + (7-1)2}{90 * 7 * 2} \right]$$

$$= 0,815$$

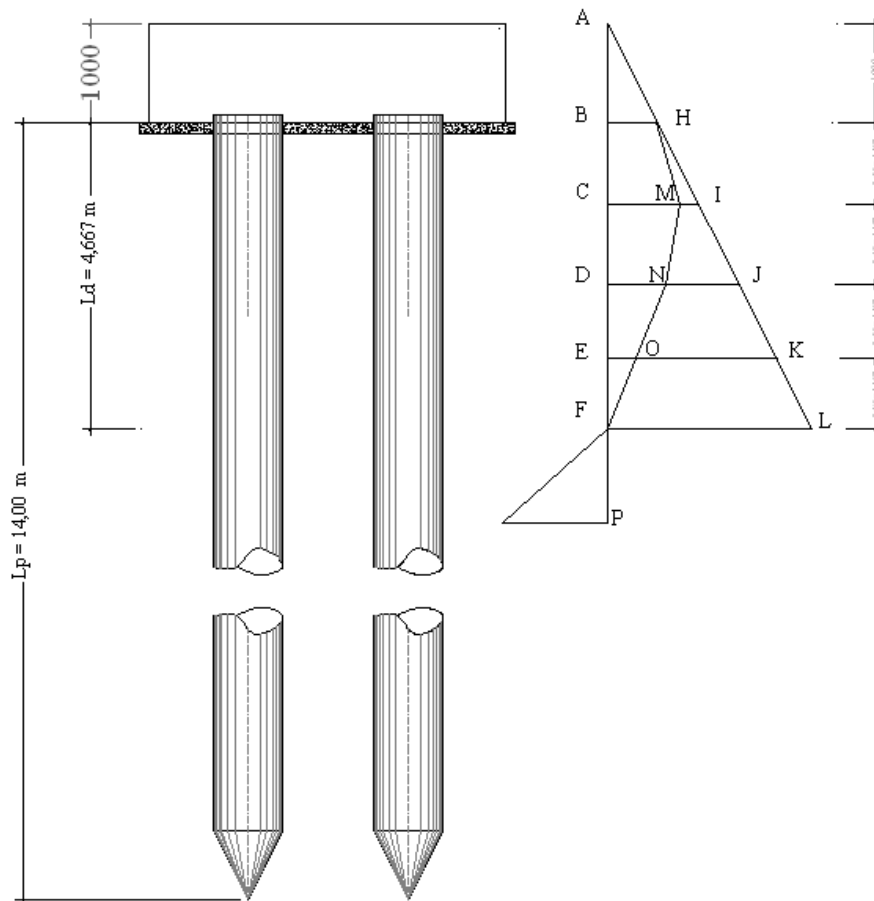
Daya dukung tiap kelompok tiang pada kelompok tiang :

$$P_{all} = 291,374 \times 0,815 = 237,469 \text{ T}$$

Kontrol  $P_{all}$  terhadap  $P_{maks}$  yang terjadi :

$$P_{all} (237,469 \text{ T}) > P_{maks} (152,775 \text{ T}) \dots\dots\dots \text{OK}$$

### 5.3.3.2 Kontrol Gaya Horisontal



Gambar 5.59 gaya horizontal tekanan pasif pada pondasi

Diketahui :

$$L_p = 14 \text{ m}$$

$$L_a = 1,0 \text{ m}$$

Panjang penjepitan :

$$L_d = 1/3 L_p = 1/3 \times 14 = 4,667 \text{ m}$$

$$L_h = L_d + L_a = 4,667 + 1,00 = 5,667 \text{ m}$$

$$\text{Lebar Poer} = 10,5 \text{ m}$$

Kedalaman 0 – 15 m :

$$\gamma = 1,566 \text{ T/m}^3$$

$$\phi = 20^\circ$$

$$C = 0,02 \text{ kg/cm}^2$$

$$K_p = \tan^2(45^\circ + \varphi/2)$$

$$= 2,039$$

Perhitungan diagram tekanan tanah pasif :

$$FL = (K_p \cdot \gamma \cdot AF) \cdot L = (2,039 \cdot 1,566 \cdot 5,667) \cdot 10,5 = 189,999 \text{ T/m}$$

$$EK = (K_p \cdot \gamma \cdot AE) \cdot L = (2,039 \cdot 1,566 \cdot 4,500) \cdot 10,5 = 150,872 \text{ T/m}$$

$$DJ = (K_p \cdot \gamma \cdot AD) \cdot L = (2,039 \cdot 1,566 \cdot 3,333) \cdot 10,5 = 111,746 \text{ T/m}$$

$$CI = (K_p \cdot \gamma \cdot AC) \cdot L = (2,039 \cdot 1,566 \cdot 2,167) \cdot 10,5 = 72,653 \text{ T/m}$$

$$BH = (K_p \cdot \gamma \cdot AB) \cdot L = (2,039 \cdot 1,566 \cdot 1,000) \cdot 10,5 = 33,527 \text{ T/m}$$

Tekanan tanah pasif efektif bekerja :

$$BH = 33,527 \text{ T/m}$$

$$CM = 0,75 \times 72,653 = 54,489 \text{ T/m}$$

$$DN = 0,5 \times 111,746 = 55,873 \text{ T/m}$$

$$EO = 0,25 \times 150,872 = 37,718 \text{ T/m}$$

Resultan tekanan pasif :

$$P1 = 0,5 \cdot BH \cdot La = 0,5 \cdot 33,527 \cdot 1,00 = 16,763 \text{ T}$$

$$P2 = 0,5 \cdot (BH + CM) \cdot BC = 0,5 \cdot (33,527 + 54,489) \cdot 1,167 = 51,357 \text{ T}$$

$$P3 = 0,5 \cdot (CM + DN) \cdot CD = 0,5 \cdot (54,489 + 55,873) \cdot 1,167 = 64,396 \text{ T}$$

$$P4 = 0,5 \cdot (DN + EO) \cdot DE = 0,5 \cdot (55,873 + 37,718) \cdot 1,167 = 54,610 \text{ T}$$

$$P5 = 0,5 \cdot (EO + 0) \cdot EF = 0,5 \cdot (37,718 + 0) \cdot 1,167 = 22,008 \text{ T}$$

Titik tangkap resultan :

$$\Sigma P \cdot L_z = P1 \cdot L1 + P2 \cdot L2 + P3 \cdot L3 + P4 \cdot L4 + P5 \cdot L5$$

$$L1 = 5,003 \text{ m}$$

$$L2 = 4,056 \text{ m}$$

$$L3 = 2,889 \text{ m}$$

$$L4 = 1,723 \text{ m}$$

$$L5 = 1,167 \text{ m}$$

$$209,134 \cdot L_z = 16,763 \cdot 5,003 + 51,357 \cdot 4,056 + 64,396 \cdot 2,889 + 54,610 \cdot 1,723 + 22,008 \cdot 1,167$$

$$209,134 \cdot L_z = 597,984$$

$$L_z = 2,859 \text{ m}$$

$$\Sigma M_s = 0$$

$$PH (1,00 + L_d + L_z) = \Sigma P \times L_z$$

$$PH = \frac{(\Sigma P \times L_z)}{(1,00 + L_d + L_z)} = \frac{209,134 \times 2,859}{1,00 + 4,667 + 2,859} = 70,128 \text{ T}$$

$$= 70,128 \text{ T} < PH_{\text{max}} \text{ yang terjadi (124,960 T) ....tidak aman}$$

Kesimpulan dari perhitungan di atas adalah diperlukannya pemasangan tiang pancang miring, ini disebabkan karena tekanan tanah pasif efektif yang terjadi masih belum dapat mengatasi gaya horisontal yang bekerja pada konstruksi.

### 5.3.4 Perhitungan Tiang Pancang Miring

Rumus :  $H_{\text{ijin}} + N1.P \sin \alpha \geq H_{\text{yang bekerja}} \times FS$

Dimana :

$H_{\text{ijin}}$  : gaya horisontal yang mampu ditahan oleh tekanaan tanah pasif

$N$  : jumlah tiang pancang miring

$P$  : daya dukung tiang pancang vertikal dalam group = 237,469 T

$H_{\text{yang bekerja}}$  : total gaya horisontal yang bekerja

Direncanakan Kemiringan tiang pancang 1 : 10 ( $\alpha = 5,71^\circ$ )

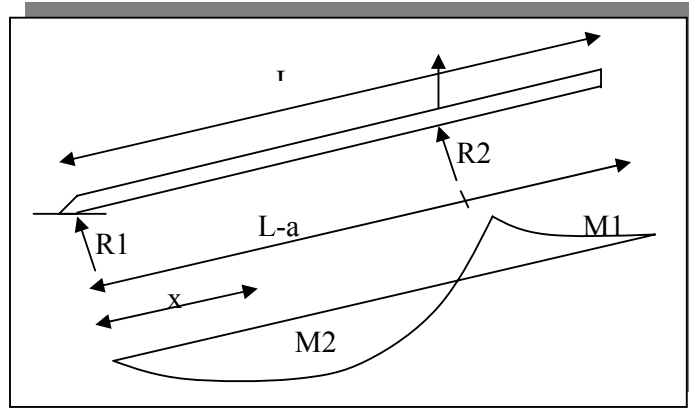
$H_{\text{ijin}} + N1.P \sin \alpha \geq H_{\text{yang bekerja}} \times FS$

$$70,128 + (237,469 N1 \sin 5,71) \geq 124,960 \times 1,5$$

$$N1 \geq 1,99 \approx 2 \text{ buah}$$

### 5.3.4.1 Perhitungan Penulangan Tiang pancang

#### a. Momen akibat pengangkatan satu titik



Gambar 5.60 Pengangkatan dengan 1 titik

$$M_1 = \frac{1}{2} \times q \times a^2$$

$$R_1 = \frac{1}{2} q (L - a) - \frac{1}{2} \times qa^2 \frac{1}{L - a} = \frac{q(L - a)}{2} - \frac{qa^2}{2(L - a)} = \frac{qL^2 - 2aq}{2(L - a)}$$

$$M_x = R_1 x - \frac{1}{2} q x^2$$

$$\text{Syarat Maksimum } \frac{dM_x}{dx} = 0$$

$$R_1 - qx = 0$$

$$x = \frac{R_1}{q} = \frac{(L^2 - 2aL)}{2(L - a)}$$

$$M_{\max} = M_2$$

$$M_{\max} = R_1 \frac{L^2 - 2aL}{2(L - a)} - \frac{1}{2} q \left( \frac{L^2 - 2aL}{2(L - a)} \right)^2$$

$$M_{\max} = \frac{1}{2} q \left( \frac{L^2 - 2aL}{2(L - a)} \right)^2$$

$$M_1 = M_2$$

$$\frac{1}{2} qa^2 = \frac{1}{2} q \left( \frac{L^2 - 2aL}{2(L - a)} \right)^2$$

$$a = \frac{L^2 - 2aL}{2(L - a)}$$

$$2a^2 - 4aL + L^2 = 0 \rightarrow L = 14 \text{ m}$$

$$2a^2 - 56a + 196 = 0$$

$$a_{1,2} = \frac{56 \pm \sqrt{(-56)^2 - 4 \cdot 1 \cdot 196}}{2 \cdot 1}$$

$$a_1 = 7 \text{ m (memenuhi)}$$

$$a_2 = 21 \text{ m (tidak memenuhi)}$$

$$WD = \frac{1}{4} \times \pi \times d^2 \times \gamma_{\text{beton}} = \frac{1}{4} \times 3,14 \times 0,35^2 \times 2500 = 240,406 \text{ kg/m}$$

$$WL = 40 \text{ kg/m}$$

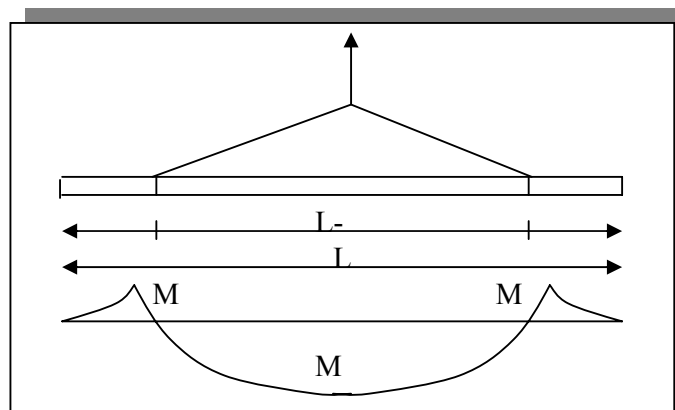
$$q_{\text{tot}} = 1,2 WD + 1,6 WL = (1,2 \times 240,406) + (1,6 \times 40) = 352,487 \text{ kg/m}$$

$$\begin{aligned} M_1 &= M_2 = M_{\text{max}} \\ &= \frac{1}{2} \times q \times a^2 = \frac{1}{2} \times 352,487 \times 7^2 \\ &= 8635,932 \text{ kgm} \\ &= 8,635 \text{ Tm} \end{aligned}$$

$$\begin{aligned} R_1 &= \frac{qL^2 - 2aq}{2(L - a)} = \frac{352,487 \cdot 14^2 - 2 \cdot 7 \cdot 352,487}{2(14 - 7)} \\ &= 4582,331 \text{ kg} = 4,582 \text{ T} \end{aligned}$$

$$R_2 = \frac{qL^2}{2(L - a)} = \frac{352,487 \cdot 14^2}{2(14 - 7)} = 4934,818 \text{ T}$$

**b. Momen akibat pengangkatan dengan dua titik**



Gambar 5.61 Pengangkatan dengan 2 titik



$$M_1 = \frac{1}{2} \times q \times a^2$$

$$M_2 = \frac{1}{8} q (L - 2a)^2 - \frac{1}{2} qa^2$$

$$M_1 = M_2$$

$$\frac{1}{2} qa^2 = \frac{1}{8} q (L - 2a)^2 - \frac{1}{2} qa^2$$

$$4a^2 + 4aL - L^2 = 0$$

$$4a^2 + 56a - 196 = 0$$

$$a = 2,899 \text{ m}$$

$$a = -16,899 \text{ m}$$

$$M_1 = M_2 = M_{\max} = \frac{1}{2} \times q \times a^2 = \frac{1}{2} \times 325,487 \times 2,899^2 = 683,864 \text{ kgm} = 0,684 \text{ Tm}$$

$$R1 = \frac{1}{2} \times q \times L = \frac{1}{2} \times 325,487 \times 2,899 = 471,793 \text{ kg} = 0,472 \text{ T}$$

Pada perhitungan tulangan didasarkan pada momen pengangkatan dengan 1 titik karena momen yang didapat dari 2 titik pengangkatan lebih kecil daripada momen pengangkatan akibat 1 titik. Pada perhitungan tulangan didasarkan pada momen pengangkatan dengan 1 titik.

$$M_{\text{design}} = 1,5 \times M_{\text{Max}} = 1,5 \times 8,635 \text{ Tm} = 12,952 \text{ Tm}.$$

Direncanakan ;

$$f'_c = 40 \text{ Mpa}$$

$$f_y = 240 \text{ Mpa}$$

$$\text{Diameter pancang (h)} = 350 \text{ mm}$$

$$\text{Tebal selimut (p)} = 50 \text{ mm}$$

$$\text{Diameter efektif (d)} = 350 - 50 - 0,5 \times 20 - 14 = 276 \text{ mm}$$

$$\rho_{\min} = \frac{1,4}{f_y} = \frac{1,4}{240} = 0,0583$$

$$\rho_{\max} = 0,75 \times \beta_1 \times \left[ \frac{0,85 \times f_c'}{f_y} \times \frac{600}{600 + f_y} \right] \text{ dimana } \beta_1 = 0,85$$

$$\rho_{\max} = 0,75 \times 0,85 \times \left[ \frac{0,85 \times 45}{240} \times \frac{600}{600 + 240} \right] = 0,0723$$

Tiang pancang berbentuk bulat, sehingga perhitungannya dikonfirmasi ke dalam bentuk bujur sangkar dengan  $b = 0,88D = 0,88 \cdot 0,35 = 0,308 \text{ m}$

$$\frac{M_u}{bxd^2} = \rho \cdot \phi \cdot f_y \left[ 1 - 0,588 \rho \times \frac{f_y}{f_c'} \right]$$

$$\frac{M_u}{bxd^2} = \rho \times 0,8 \times 240 \left[ 1 - 0,588 \rho \times \frac{240}{40} \right]$$

$$\frac{12952}{0,308 \times 0,276^2} = 192\rho - 602,112\rho^2$$

$$677,376\rho^2 - 192\rho - 551148,936 = 0$$

$$\rho = 0,262$$

$$\rho_{\min} = 0,0583$$

$$\rho_{\max} = 0,0724$$

#### Tulangan utama

$$A_{st} = \rho_{\min} \cdot b \cdot d \cdot 10^6 = 0,0583 \times 308 \times 276 = 4955,966 \text{ mm}^2$$

Dipakai tulangan 8Ø28 ( 4926 mm<sup>2</sup> )

#### 5.3.4.2 Kontrol Gaya Vertikal

$$\text{Rumus : } [(P \times N_2) + N_1 \times (P \cdot \cos \alpha)] \geq V$$

dimana :

P : kemampuan tiang pancang vertikal dalam group = 237,469 T

N1 : jumlah tiang pancang miring = 2 bh

N2 : jumlah tiang pancang vertical = 12 bh

V : beban vertikal yang bekerja pada konstruksi = 638,406 T

$$[(PxN_2) + N_1x(P.\cos\alpha)] \geq V$$

$$(237,469 \times 12) + 2 (237,469 \cos 5,71) \geq 638,406 \text{ T}$$

$$3332,209 \text{ T} \geq 638,406 \text{ T} \dots\dots\dots \text{OK}$$

#### 5.3.4.3 Kontrol terhadap Tumbukan Hammer

Jenis Hammer yang akan digunakan adalah tipe K –35 dengan berat hammer 3,5 ton.

Daya dukung satu tiang pancang = 152,775 T

Rumus Tumbukan :

$$R = \frac{Wr .H}{\Phi (s + c)}$$

Dimana :

R = Kemampuan dukung tiang akibat tumbukan

Wr = Berat Hammer = 3,5 T

H = Tinggi jatuh Hammer = 1,5 m

S = final settlement rata-rata = 2,5 cm = 0,025 m

C = Koefisien untuk double acting system Hammer = 0,1

Maka :

$$R = \frac{Wr .H}{\Phi (s + c)}$$

$$R = \frac{3,5 \times 1,5}{0,2(0,025 + 0,1)} = 210 \text{ T} < P_{\text{tiang}} = 152,775 \text{ T} \dots\dots\dots \text{OK}$$

#### ◦ Penulangan Akibat Tumbukan

Dipakai rumus New Engineering Formula :

$$P_U = \frac{eh.Wr .H}{s + c}$$

Dimana :

P<sub>U</sub> = Daya Dukung Tiang tunggal

eh = efisiensi Hammer = 0,8

H = Tinggi jatuh Hammer = 1,5 m

S = final settlement rata-rata = 2,5 cm

Maka :

$$P_U = \frac{eh.Wr.H}{s+c} = \frac{0,8 \times 3,5 \times 1,5}{0,025 + 0,1} = 33,6 \text{ T}$$

Menurut SKSNI – T – 03 – 1991 Pasal 3.3.3.5

Kuat Tekan Struktur :

$$P_{mak} = 0,8 ( 0,85 f'c ( A_g - A_{gt} ) + f_y.A_{st} )$$

$$33600 = 0,8 ( 0,85.400 ( 3,14.17,5^2 - A_{st} ) + 2400.A_{st} )$$

$$A_{st} = 1187,302 \text{ mm}^2$$

Dipakai tulangan 6 Ø 16 ( 1206 mm<sup>2</sup> )

#### 5.3.4.4 Kontrol geser

$$\tau_b = \frac{D \max}{0,9 \times 1/4 \pi . d^2} = \frac{(-q.a) + (1/2.q.L)}{0,9 \times 1/4 \pi . d^2}$$

$$\tau_b = \frac{(352,487 \times 7) + (1/2 \times 352,487 \times 14)}{0,9 \times 1/4 \times 3,14 \times 0,35^2}$$

$$= 32599,954 \text{ kg/m}^2 = 3,259 \text{ kg/cm}^2$$

$$\tau_b = 0,53\sigma \rightarrow \sigma = 2400 \text{ kg/cm}^2$$

$$= 0,53 \cdot 1600 = 1272 \text{ kg/cm}^2$$

karena  $\tau_b < \tau_{b \text{ ijin}}$  maka tidak perlu tulangan geser, maka digunakan tulangan sengkang praktis yaitu tulangan spiral.

#### Perhitungan Tulangan Spiral

Rasio penulangan spiral :

$$\rho_s = 0,45 \left( \frac{A_g}{A_c} - 1 \right) \times \frac{f_c}{f_y}$$

$$\rho_s = 0,45 \left( \frac{1/4 \cdot \pi \cdot 35^2}{1/4 \cdot \pi \cdot 25^2} - 1 \right) \times \frac{400}{2400} = 0,0721$$

$$A_s = 2 \times \rho_s \times A_c$$

$$= 2 \times 0,0721 \times 1/4 \cdot \pi \cdot 25^2$$

$$= 70,748 \text{ cm}^2$$

$$s = 2 \times \pi \times D_c \times A_{sp}/s$$

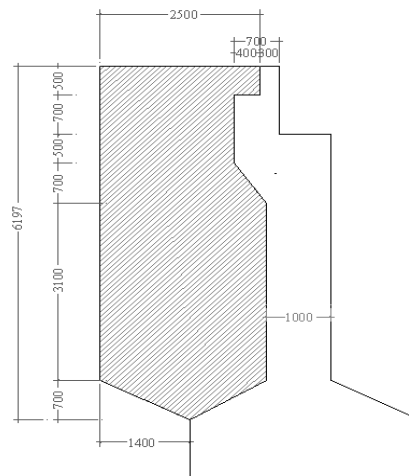
$$= 2 \times 3,14 \times 35 \times \frac{1}{4} \cdot 3,14 \cdot 1^2 / 164,85 = 1,046 \text{ cm} \rightarrow 5 \text{ cm}$$

sehingga dipakai tulangan **Ø8-50**

sengkan pada ujung tiang dipakai **Ø8-50**

sengkan pada tengah tiang dipakai **Ø8-100**

### 5.3.4 Perhitungan Wing Wall



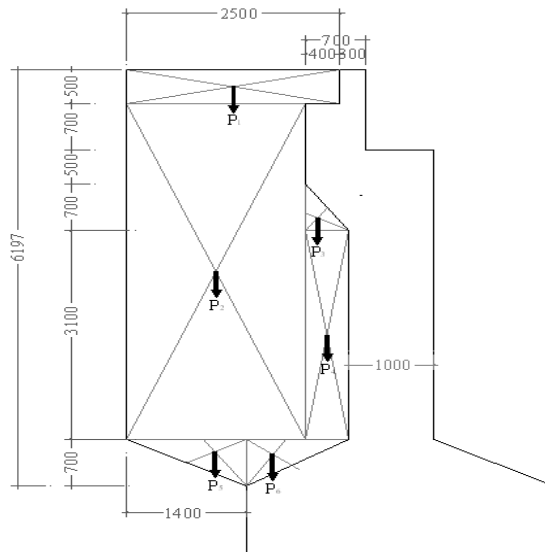
Gambar 5.62 Dimensi Wingwall

#### a. Pembebanan Wingwall

Akibat Berat Sendiri

$$\text{Tebal wingwall minimum} = 1/20 \times h_w = 1/20 \times 619,7 \text{ cm} = 30,985 \text{ cm}$$

$$\text{Direncanakan tebal wingwall} = 40 \text{ cm}$$



Gambar 5.63 Akibat Berat Sendiri *Wingwall*

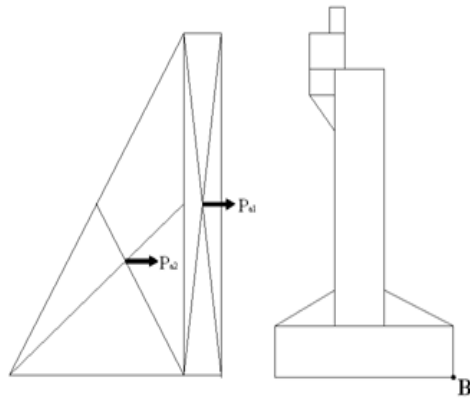
Tabel 5.20 Perhitungan Akibat Beban Sendiri *Wing wall*

	P (m)	L (m)	T (m)	V(m <sup>3</sup> )	$\gamma_{\text{beton}}$	W ( T )	X (m)	Momen (T.m)
1	2.500	0.500	0.400	0.500	2.500	1.250	1.250	1.563
2	2.100	5.000	0.400	4.200	2.500	10.500	1.050	11.025
3	0.400	0.700	0.400	0.112	2.500	0.280	0.267	0.075
4	0.400	3.100	0.400	0.496	2.500	1.240	0.200	0.248
5	1.400	0.700	0.400	0.392	2.500	0.980	1.667	1.634
6	1.200	0.700	0.400	0.336	2.500	0.840	0.600	0.504
$\Sigma$				6.036		15.090		15.048

6

Akibat Tekanan Tanah

Dari perhitungan pembebanan abutment akibat tekanan tanah aktif,  
diperoleh :



Gambar 5.64 Akibat Tekanan Tanah aktif

Diketahui :

- Tanah Lapisan 1 (tanah urugan)

$$\gamma_1 = 2,0 \text{ gr/cm}^3$$

$$\phi_1 = 28^\circ$$

$$C_1 = 1 \text{ kg/cm}^2$$

$$H_1 = 6,5 \text{ m}$$

- Koefisien tekanan tanah aktif:

$$\begin{aligned} K_{a1} &= \tan^2(45^\circ - \phi_1/2) \\ &= \tan^2(45^\circ - 28/2) \\ &= 0,360 \end{aligned}$$

Menurut pasal 1.4 P3JJR SKBI 1.3.28.1987, muatan lalu lintas dapat diperhitungkan sebagai beban merata senilai dengan tekanan tanah setinggi:  $h = 60 \text{ cm}$ , jadi beban lalu lintas ( $q_x$ ) :

$$\begin{aligned} q_x &= \gamma_1 \times h \\ &= 2,0 \times 0,6 \\ &= 1,2 \text{ t/m}^2 \end{aligned}$$

$$\begin{aligned} q_1 &= q_{\text{pelat injak}} + q_x \\ &= 1,457 + 1,2 \\ &= 2,657 \text{ T/m}^2 \end{aligned}$$

Gaya tekanan tanah aktif:

$$\begin{aligned} P1 &= Ka_1 \times q_1 \times H_1 \\ &= 0,36 \times 2,657 \times 6,5 \\ &= 6,217 \text{ Ton} \end{aligned}$$

$$\begin{aligned} P2 &= \frac{1}{2} \times \gamma_1 \times Ka_1 \times H_1^2 \\ &= \frac{1}{2} \times 2,657 \times 0,360 \times 6,5^2 \\ &= 20,206 \text{ T} \end{aligned}$$

$$M = 6,127 \times 3,600 + 20,206 \times 4,800 = 119,045 \text{ Tm}$$

#### b. Penulangan Wingwall

Direncanakan :

$$f'_c = 35 \text{ MPa}$$

$$f_y = 240 \text{ Mpa}$$

$$b = 1000 \text{ mm}, h = 1000 \text{ mm}$$

$$M_{tot} = 15,048 + 119,045 = 134,092 \text{ Tm}$$

Diameter tulangan utama dipakai D16, dan tulangan pembagi dipakai D14, sehingga :

$$d' = h - (50 + 14 + \frac{1}{2} 16) = 1000 - (50 + 14 + 8) = 928 \text{ mm}$$

$$\Phi = 0,65$$

$$M_u = M_{tot} / 0,6 = 223,486 \text{ Tm}$$

$$\frac{M_u}{bd^2} = \rho \times 0,8 \times f_y (1 - 0,588 \times \rho \times \frac{f_y}{f'_c})$$

$$\frac{223486}{1 \times 0,928^2} = \rho \times 0,8 \times 2400 (1 - 0,588 \times \rho \times \frac{2400}{35})$$

$$7741,44 \rho^2 - 1920 \rho + 459847,736 = 0, \quad \rho = 1,638$$

$$\rho_{min} = \frac{1,4}{f_y} = \frac{1,4}{240} = 0,0058$$

$$\rho_{max} = 0,75 \times \beta_1 \left( \frac{0,85 f'_c}{f_y} \times \frac{600}{600 + f_y} \right) \text{ dan } \beta_1 = 0,85$$



$$\rho_{\max} = 0,75 \times 0,85 \left( \frac{0,85 \times 350}{2400} \times \frac{600}{600 + 2400} \right) \text{ dan } \beta_1 = 0,85$$

$$\rho_{\max} = 0,015$$

$$\text{dipakai } \rho_{\min} = 0,0058$$

- Tulangan Pokok

$$A_{s_{\text{total}}} = \rho \times b \times d = 0,0058 \times 1000 \times 928 = 5382,4 \text{ mm}^2$$

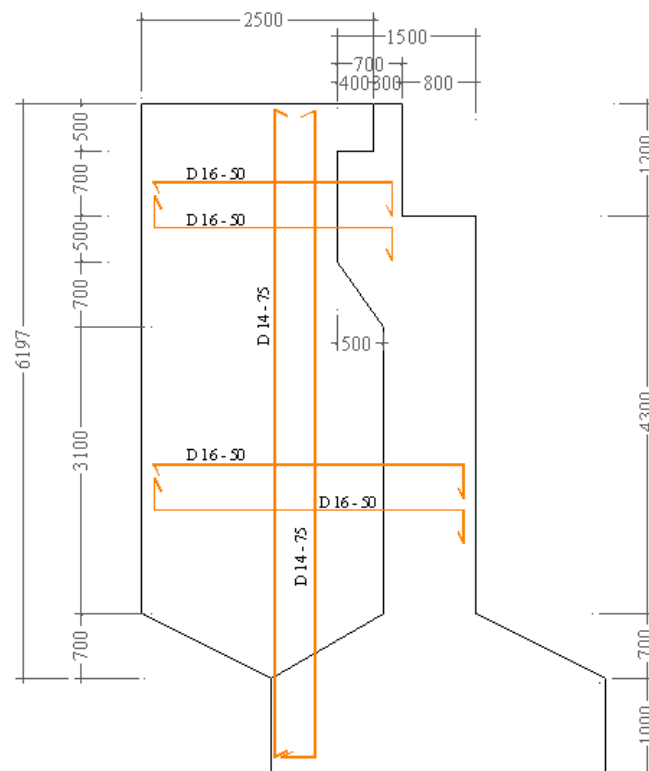
$$A_{s_{\text{kiri}}} = A_{s_{\text{kanan}}} = 0,5 A_{s_{\text{total}}} = 2691,2 \text{ mm}^2$$

Dipakai tulangan rangkap D16 – 50 ( $A_s = 4022 \text{ mm}^2$ )

- Tulangan bagi

$$\text{Diambil sebesar 20 \% dari tulangan utama} = 1076,48 \text{ mm}^2$$

Dipakai tulangan rangkap D14 – 75 ( $A_s = 1608 \text{ mm}^2$ )



Gambar 5.65 Penulangan Wingwall

### 5.3.5 Perhitungan Bearing Elastomer

Untuk perletakan jembatan direncanakan menggunakan bearing merk CPU buatan Indonesia. CPU *Elastomeric Bearing* memiliki karakteristik sebagai berikut:

#### b. Spesifikasi

- Merupakan bantalan atau perletakan elastomer yang dapat menahan beban berat, baik yang vertikal maupun horisontal.
- Bantalan atau perletakan elastomer disusun atau dibuat dari lempengan elastomer dan logam secara berlapis – lapis
- Merupakan satu kesatuan yang saling merekat kuat, diproses dengan tekanan tinggi.
- Bantalan atau perletakan elastomer berfungsi untuk merdam getaran, sehingga kepal jembatan (*abutment*) tidak mengalami kerusakan.
- Lempengan logam yang paling luar dan ujung – ujungnya elastomer dilapisi dengan lapisan elastomer supaya tidak mudah berkarat.
- Bantalan atau perletakan elastomer (*neoprene*) dibuat dari karet sintetis

#### c. Pemasangan

- Bantalan atau perletakan elastomer dipasang diantara tumpuan kepala jembatan dan gelagar jembatan.
- Untuk melekatkan bantalan atau elastomer dengan beton atau baja dapat digunakan lem *epoxy rubber*.

#### d. Ukuran

- Selain ukuran – ukuran standart yang sudah ada, juga dapat dipesan ukuran sesuai permintaan.

Gaya vertikal ditahan oleh *bearing elastomer* dan gaya horisontal ditahan oleh *seismic buffer*.

Reaksi tumpuan yang terjadi pada rangka jembatan rangka baja berdasarkan analisis SAP 2000 versi 7.02, yaitu :

- Gaya vertikal pada joint 1 =  $64,630 \text{ T} = 646,30 \text{ kN}$ .
- Gaya horisontal dihitung berdasarkan gaya rem :  

$$\text{Gaya rem} = P_{\text{RM}} = 9,658 \text{ T}$$

$$\text{Gaya gempa} = 108,786 \text{ T}$$

Total gaya horisontal = 118,444 T = 1184 kN.

Spesifikasi elastomer dapat dilihat dalam tabel sebagai berikut :

Tabel 5.21 Spesifikasi Bearing Elastomer dan Seismic Buffer

Jenis	Ukuran (mm)	Beban Max (KN)
TRB 1	480.300.87	2435
TRB 2	480.300.101	3600
TRB 3	350.280.97	540
TRB 4	350.280.117	690

Dimensi bearing elastomer

TRB 1 ukuran 480.300.87

Beban max = 2435 kN > 1184 kN

Dimensi seismic buffer

TRB 4 ukuran 350.280.117

Beban max = 690 kN > 646,30 kN .....OK

### 5.3.6 Perhitungan Angkur

Angkur berfungsi menahan gaya gesekan kesamping.

Digunakan angkur mutu baja 52

Gaya gesek =  $0,08 \times v$

$$\text{Luas penampang} = \frac{\text{gaya gesek}}{0,58 \sigma}$$

Dipakai Angkur diameter 25 mm

$$\begin{aligned} a &= \frac{1}{4} \times \pi \times d^2 \\ &= \frac{1}{4} \times 3,14 \times 25^2 \\ &= 490,625 \text{ mm}^2 \end{aligned}$$

$$\begin{aligned} \text{Jumlah angkur} &= \frac{A}{a} \\ &= \frac{A}{490,625} \end{aligned}$$

Panjang angkur max =  $40 \times d = 40 \times 2,5 = 100 \text{ cm}$

Diambil kedalaman angkur 60 cm.