DATA - DATA UMUM PERENCANAAN JEMBATAN

Diket :

Jembatan adalah kelas				=	100%	beban hidup B
Bentang jembatan beton (L1)				=	20 m	
Bentang jembatan rangka batang (L2)						
• n x λ = 9 x 4.5 m				=	40.5 m	
Lebar ruang bebas (B1)				=	34 m	
Tinggi ruang bebas (H1)				=	6 m	(TRB ≥ 5,3m
Tinggi balok (d1) dan rangka (d2):						
Muka tanah asal				=	0 m	
Muka Air Banjir Tertinggi (MAT)				=	-5 m	
Tinggi bebas (TB)				=	3 m	
BJ 52 :	fu	=	520 Mpa	=	5200	kg/cm²
	fy	=	410 Mpa	=	4100	kg/cm²
Mutu beton :	fy	=	390 Mpa	=	3900	kg/cm²
	fc'	=	35 Mpa	=	350	kg/cm²
Kekuatan angin/ letak jembatan				=	≤ 5	km dari pantai
Load Factor :	(SNI T-02-2005 pasal 5.2 Tabel 2)					
	Beton/ Aspal			=	1.3	SNI T-02 2005
	Baja			=	1.1	SNI T-02 2005
	Truk			=	1.8	SNI T-02 2005
	Kayu			=	1.4	SNI T-02 2005

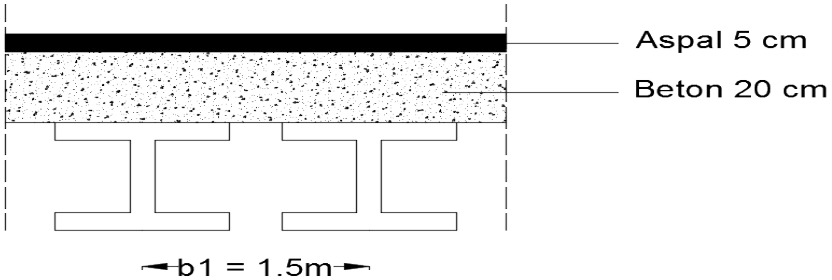
MS

Untuk Rangka Tertutup)

Ps.5.2 Tabel 2
Ps.5.2 Tabel 2
Ps 6.4 Tabel 12
Ps.5.2 Tabel 2

BAB 1
PERENCANAAN PELAT LANTAI KENDA

1.1 MODEL GAMBAR PERHITUNGAN



b1 = jarak antar balok memanjang → dianggap pelat 1 arah
b1 = lx = 1.5 m memanjang)

$$\frac{ly}{lx} \geq 2$$

λ = jarak antar rangka/balok melintang
λ = ly = 4.5 m

$$\frac{ly}{lx} = \frac{4.5}{1.5}$$

* Pelat beton dihitung per meter panjang.

Berdasarkan SNI T 14-2004 pasal 5.5.2

Pelat beton, d3 ≥ 200 mm
d3 ≥ 100 + 40 b1
≥ 100 + [40 1.5]
≥ 160 mm
Maka, dipakai d3 = 200 mm = 20 cm

Berdasarkan BMS '92 Pasal 2.2.3.2

Tebal aspal, d4 = 5 sd 8 cm
Maka, dipakai = 5 cm

1.2 PERHITUNGAN PELAT LANTAI

1.2.1 Faktor Beban

- Beton cor ditempat → K^u_{MS} = 1.3 = LF (akibat get
- Muatan truk → K^u_{TT} = 1.8

1.2.2 Pembebanan

1.2.2.1 *Beban Mati* (dihitung per 1 m panjang)

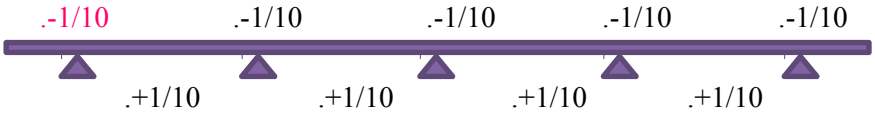
Beban sendiri pelat = d3 x bj.beton x K^u_{MS}
= 0.2 x 24 x 1.3 = 6.24
Beban aspal = d4 x bj.aspal x LF Aspal
= 0.05 x 22 x 1.3 = 1.43
q_{MU} = 7.67

1.2.2.2 *Beban Hidup*

Beban hidup "T" sebesar 112.5 kN (beban roda truk) [SNI T 02-2005 pasal 6.4.1] yai tambahan faktor kejut (DLA) sebesar 0,3.

Beban **Tu** = 112.5 x (1 + 0,3) x 1.8 = 263.25 kN

1.2.3 Perhitungan Momen (Momen-momen Arah Melintang = Mx)



Beban Mati	=	1/10 x q_{MU} x $b1^2$	=	1/10 7.67	2.25
Beban Hidup	=	$(0,8\times(S+0,6)\times Tu)/10$	=	0.8 55.283	

Total → Mu

1.2.4 Penulangan Pelat

Direncanakan :

fy	=	390 Mpa	Selimut beton	d'	=	40	
fc'	=	35 Mpa	Tebal pelat	tp	=	200	
β1	=	0.6					
φ	=	0.8	*(faktor reduksi kekuatan berdasarkan SK SNI 3.2.3-2.2.a untuk tulan dan aksial tarik dengan lentur)*				
b	=	1000 mm					

Diameter tulangan D = 10 mm

Ø = 16 mm

$$\rho b = \frac{0{,}85 \times \beta 1 \times fc'}{fy}\left(\frac{600}{600+fy}\right) \rightarrow \text{(SK SNI 3.1.4-}$$

$$= \frac{0.85 \quad 0.6 \quad 35}{390}\left(\frac{600}{600 \quad + \quad 390}\right) = 0.0277$$

$$\rho_{min} = \frac{1.4}{fy} = \frac{1.4}{390} = 0.0036$$

$$\rho_{max} = 0{,}75 \times \rho b \rightarrow \text{(SK SNI 3.3.3-3)}$$

$$= 0.75 \times 0.0277$$

$$= 0.0208$$

$$dx = h - d' - dt - 1/2Ø$$

$$= 200 - 40 - 10 - 8$$

$$= 142 \text{ mm} = 14.2 \text{ cm}$$

$$Rn = \frac{Mu}{\varphi \cdot b \cdot dx^2} = \frac{45951750}{16131200} = 2.8486 \text{ N/mm}^2 \text{ (Mpa)}$$

$$m = \frac{fy}{0{,}85 \times fc'} = \frac{390}{29.75} = 13.109$$

$$\rho_{perlu} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \quad m \quad Rn}{fy}}\right)$$

$$= \frac{1}{13.109}\left(1 - \sqrt{1 - \frac{2 \quad 13.109 \quad 2.8486}{390}}\right) =$$

ρ_{min} < ρ_{perlu} < ρ_{max}

0.004 < 0.0077 < 0.021 **OK → Dipakai ρ_{perlu}**

As_{perlu} = ρ_{perlu} x b x dx = 0.0077 x 1000 x 142

Digunakan tulangan Ø16- 120 → As = 1675 mm² /m

As_{susut} (arah Y) = ρ_{min} x b x dx = 0.0036 1000 142

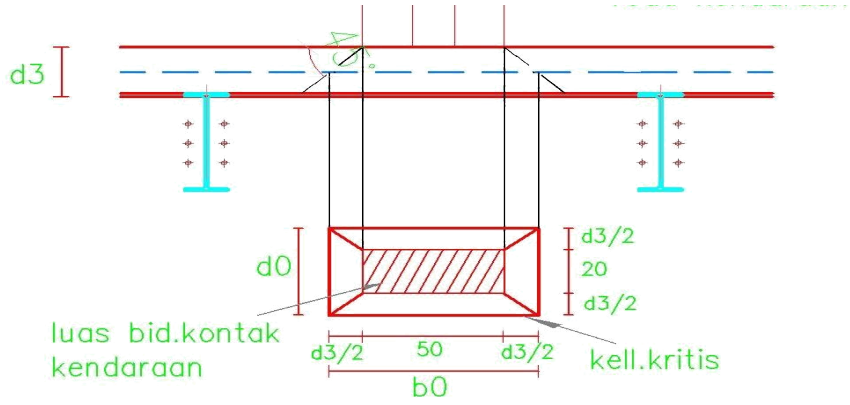
Digunakan tulangan D10- 150 → As = 523.33 mm² /m

1.2.5 Kontrol Geser Pons



bo =

do =



Keliling kritis

U = 2 x (bo

= 2200

Luas bidand k

Ak = 2 x (bo

= 44(

$$bc = \frac{bo}{do}$$

Gaya geser (V) = K^{u}_{TT} x 112.5 x (1 + 0,3)

= 1.8 x 112.5 x (1 + 0,3) =

Kemampuan geser nominal beton (Vc) *(SK SNI 3.4.11.2).(1)*

$$= \left(1 + \frac{2}{bc}\right) x \left(\frac{\sqrt{f'c}}{6}\right) U$$

$$= \left(1 + \frac{2}{1.75}\right) x \left(\frac{\sqrt{35}}{6}\right) 22$$

= 2.1429 x 0.986 x 2200

= 660065.47294 N

= 660.07 kN

φVc = 0.6 x 660.07 = 396 kN → Vuc

V < Vuc → 263.25 kN < 396 kN **OK**

RAAN

(tertumpu menerus pada balok

= 3 **OK**

aran)

kNm

kNm +
kNm

ıg dikalikan dengan K^{u}_{TT} dan

(*PBI 1971 halaman 195 poin F*)

= 1.7258 kNm
= 44.226 kNm +

= 45.952 kNm

mm
mm

ıgan yang terkena beban tarik

·3)

ρmin < ρ Perlu < ρmax
Under Reinforced
OK

0.007691983

= **0.0077**

= 1092.26 mm² /m

= 509.74 mm² /m

700 mm
400 mm

+do)
mm

ritis (yang di arsir)
+do) x d3
)00 mm²

= 1.75

263.25 kN

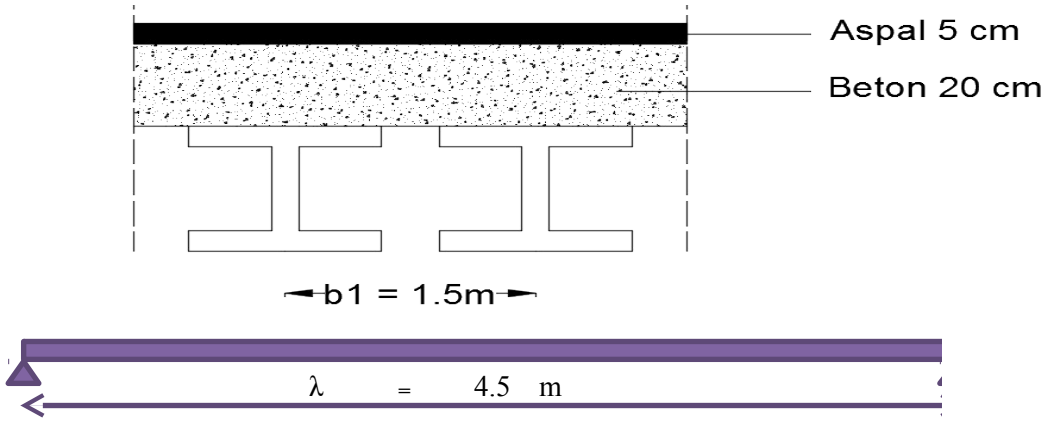
dx

00 142

x 142

BAB 2
PERENCANAAN BALOK MEMANJANG

2.1 MODEL GAMBAR PERHITUNGAN



A dan B adalah tumpuan yang diasumsikan Simple Connection ke balok melintang.

Direncanakan Profil WF : WF 350 x 350 x 159

h	=	356	mm		Ix	=	47600	cm^4
b	=	352	mm		Iy	=	16000	cm^4
tf	=	22	mm		ix	=	15.3	cm
tw	=	14	mm		iy	=	8.9	cm
r	=	20	mm					
A	=	202.00	cm^2			hw	=	312
w	=	159 Kg/m	=	1.590 kN/m		hf	=	334
Zx	=	2927.20	cm^3					
Zy	=	1378.23	cm^3		$Z_x = (t_w \cdot h_w^2)/4 + h_f \cdot t_f \cdot b_f$			.....(8)
Sx	=	2670	cm^3		$Z_y = (h_w \cdot t_w^2 + 2\, t_f \cdot b_f^2)/4$			
Sy	=	909	cm^3		$h_w = h - 2\, t_f$ $h_f = h - t_f$			.....(9)

2.2 PERHITUNGAN BALOK MEMANJANG

2.2.1 Pembebanan

2.2.2.1 *Beban Mati*

Aspal = d4 x bj.aspal x LF x b1
= 0.05 22 1.3 1.5

Pelat beton = d3 x bj.beton x K^u_{MS} x b1
= 0.2 24 1.3 1.5

Beban sendiri profil = berat profil x LF
= 1.590 x 1

Beban bekisting (asumsi) = 0,5 kN/m x LF
= 0.5 x 1.4

q_{MU}

M_D = 1/8 q_{MU} λ 2

$= \quad 0.125 \quad 13.10 \quad 4.5 \quad 2$

$= \quad 33.15 \quad \text{kN m}$

$V_{Aqm} = \quad 0.5 \quad q_{MU} \quad \lambda$

$= \quad 0.5 \quad 13.10 \quad 4.5$

$= \quad 29.46 \quad \text{kN}$

2.2.2.2 ***Beban Hidup***

Beban hidup merata (BTR) : *(SNI T 02-2005 pasal 3.7)*

untuk λ = 4.5 m < L = 30 m

maka digunakan q = 9 Kpa = 9 kN/m²

q_{BTR} = q b1 K^{u}_{TD}

= 9 1.5 2 = 27 kN/m

Beban hidup garis (BGT): *(berdasarkan RSNI T 02-2005 pasal 6.6)*

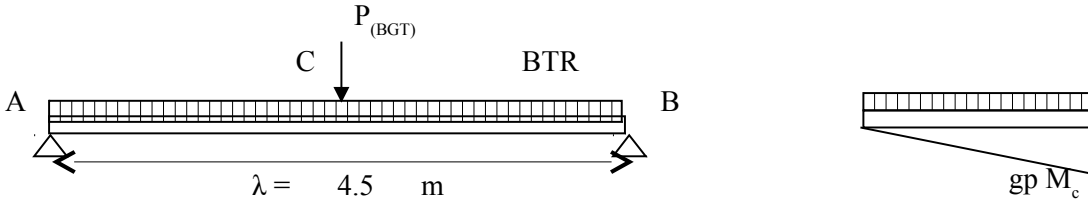
p = 49 kN/m

P_{BGT} = p { 1 + DLA } b1 K^{u}_{TD}

→ DLA = 40% untuk *L* = 40.5 *m*

P_{BGT} = 49 { 1 + 40% } 1.5 2

= 205.80 kN



M_{L1} = { P_{BGT} 1/4 λ } + { q_{BTR} 1/4 λ 1/2

= { 205.80 0.25 4.5 } + { 27 0.25 4.5 0.5

= 299.87 kNm

V_{L1} = P_{BGT} + q_{BTR} 1/2 λ

= 205.8 + 27 0.5 4.5

= 266.55 kN

Akibat beban truk "T"

T = 112.5 kN *(SNI T 02-2005 pasal 6.4.1)*

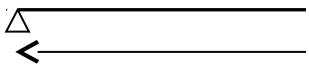
Tu = T x (1+0,3) x K^{u}_{TT} *(0,3 = DLA)*

= 112.5 1.3 1.8 = 263.25 kN

M_{L2} = Tu 1/4 λ

= 263.25 0.25 4.5

= 296.16 kNm



Momen Total yang harus diperhitungkan adalah :

M_{T1} = M_D + ML1 = 33.15 + 299.869 =

M_{T2} = M_D + ML2 = 33.15 + 296.156 =

Jadi, momen total adalah yang terbesar antara

Momen Total (M_T) = 333.02 kNm

2.2.2 Kontrol Penampang Profil

Pelat Sayap :

$$\lambda = \frac{bf}{2.tf} = \frac{352}{44} = 8$$

$$\lambda p = \frac{170}{\sqrt{fy}} = \frac{170}{410} = 8.3957$$

$\lambda < \lambda p \rightarrow$ Penampang Kompak **OK**

Pelat Badan :

$$\lambda = \frac{h}{tw} = \frac{356}{14} = 25.429$$

$$\lambda p = \frac{1680}{\sqrt{fy}} = \frac{1680}{410} = 82.97$$

$\lambda < \lambda p \rightarrow$ Penampang Kompak **OK**

Karena Penampang Kompak, maka Mn = Mp

2.2.3 Kontrol Tegangan Lentur (Tekuk Lateral)

Lb = 0

$$Lp = 1{,}76\ iy \left(\frac{E}{fy}\right)$$

$$= 1.76 \quad 8.9 \quad \left(\frac{200000}{410}\right)^{0.5}$$

$$= 345.96 \text{ cm}$$

Lb < Lp → Bentang Pendek

Karena bentang pendek, maka :

Mn = Mp

Mp = fy x Zx

= 410 x 2927200 = 1200152000 Nmm

φMp = 0.9 x 1200152000

= 1080136800 Nmm

= 1080.14 kNm

Mu = MD + MT

= 33.15 + 333.02

= 366.16 KNm

φMp ≥ Mu

→ 1080.14 kNm > 366.16 kNm **OK**

2.2.4 Kontrol Lendutan

$$f\ ijin = \frac{\lambda}{800} = \frac{4500}{800} = 5.625$$ *(SNI T 03-2005 4.7.2)*

$$f^{\circ} = \frac{5}{384} \times \frac{q_{L1}.\lambda^4}{E.Ix} + \frac{1}{48} \times \frac{P_{BGT}.\lambda^3}{E.Ix}$$

$$= \frac{5}{384} \times \frac{27 \times 4500^4}{200000 \times 476000000} + \frac{1}{48}$$

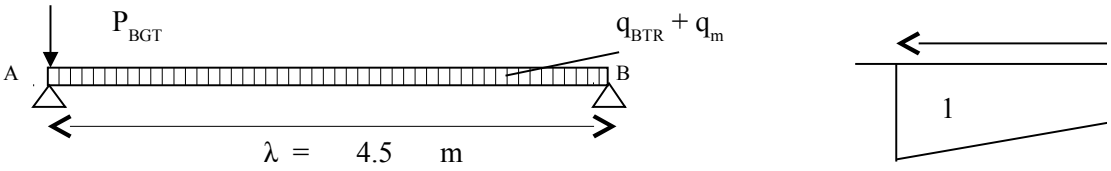
$= \quad 5.62 \quad mm$

$f° \quad < \quad f\,ijin \quad = \quad 5.62 \quad < \quad 5.625 \quad \rightarrow$ **OK**

2.2.5 Kontrol Tegangan Geser

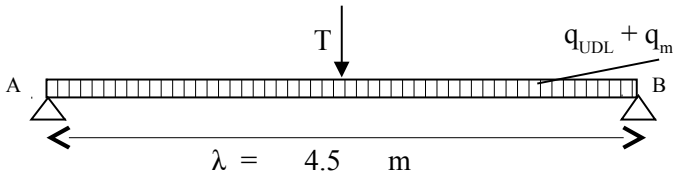
Gaya geser maksimum terjadi pada saat beban hidup berada dekat perletakan

1. Akibat beban mati + BTR + BGT



V_A	=	P_{BGT}	+	.1/2	q_{BTR} +	λ	+	.1/2	q_{MU}
	=	205.80	+	0.5	27	4.5	+	0.5	13.095
	=	461.84	kN						

2. Akibat beban mati + beban truk T



V_A	=	T	+	.1/2	q_{BTR} +	λ	+	.1/2	q_{MU}
	=	112.5	+	0.5	27	4.5	+	0.5	13.095
	=	312.56	kN						

Jadi, V_A yang digunakan adalah V_{AT} = 461.84 kN

hw	=	312 mm		hw/tw	=	22.285714	<
tw	=	14 mm					
kn	=	5					
Aw	=	4984 mm2		Vn	=	0.6 Aw fy	
fy	=	410 MPa		Vn	=	1226064	
				Vn	=	1226.06 kN	

V_{AT} = 461.84 kN < ØVn = 1103 kN →

Aspal 5 cm

Beton 20 cm

Baja Bj 52

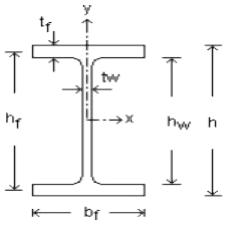
fy = 410 MPa

fu = 520 MPa

E = 200000 MPa

mm

mm



= 2.145 kN/m

= 9.36 kN/m

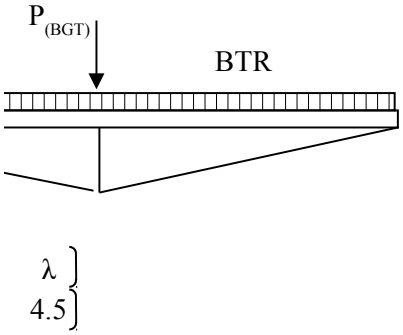
= 1.59 kN/m

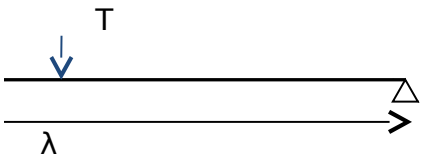
= 0.7 kN/m +

= 13.10 kN/m

Shelvy Elvina Santoso - 3113041059

(gambar 8 RSNI T 02-2005)





333.02 kNm

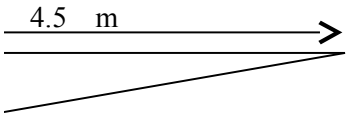
329.30 kNm

M_{T1} dgn M_{T2}

Shelvy Elvina Santoso - 3113041059

$$x \quad \frac{205800.00 \quad x \quad 4500\,3}{200000 \quad x \quad 476000000}$$

Shelvy Elvina Santoso - 3113041059



λ
4.5

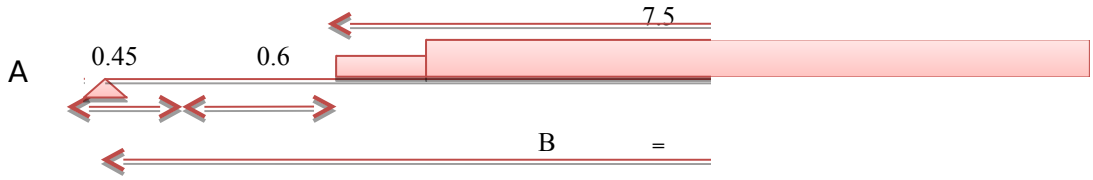
λ
4.5

1.1 √(kn E/fy) = 54.33

N

OK

3.1 MODEL GAMBAR PERHITUNGAN



Direncanakan Profil WF :

h	=	708	mm			
b	=	302	mm			
tf	=	28	mm			
tw	=	15	mm			
r	=	28	mm			
A	=	273.60	cm^2			
w	=	215	Kg/m	=	2.150	
Zx	=	7344.22	cm^3			
Zy	=	1313.531	cm^3			
Sx	=	6700	cm^3			
Sy	=	853	cm^3			

3.2 PERHITUNGAN BALOK MELINTANG

3.2.1 Pembebanan

- **Sebelum Komposit**

Beban Mati

Balok memanjang	=	(W x λ /
	=	1.590
Balok melintang	=	berat pro
	=	2.150
Pelat beton	=	d3 x γbe
	=	0.2
Bekisting (asumsi)	=	0,5 kN/n
	=	0.5

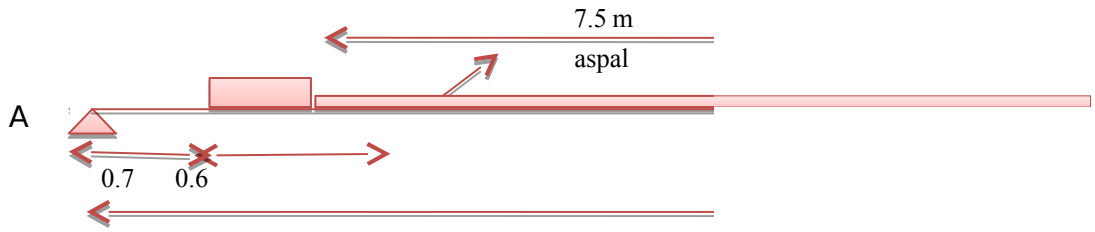
M_{qD1}	=	1/8	q_{D1}	B	2
	=	0.125	36.59	9.6	2
	=	421.53984	kNm		
	=	4215398.4	kgcm		

V_{qD1}	=	1/2	q_{D1}	B
	=	0.5	36.59	9.6

= 175.6416 kN
= 17564.16 kg

- **Setelah Komposit**

Beban Mati



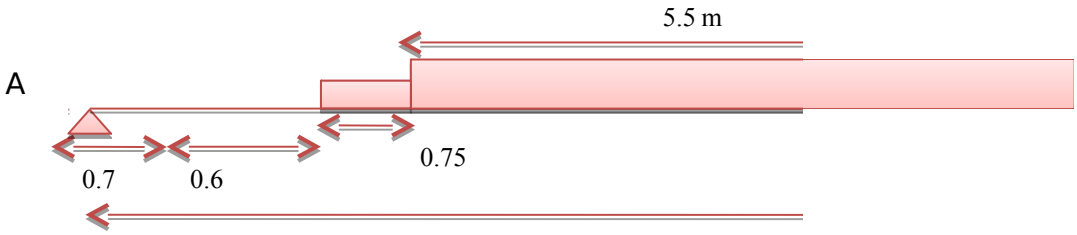
B = 9.6 m

Beban Aspal = d4 x γasp
= 0.05
Berat Kerb = dk x γbe
= 0.5

- Σ MB = 0
  9.6 Ra = 70.2 0.6
  9.6 Ra = 362.2 +
  9.6 Ra = 627.02
  Ra = 65.31434 kN

- M_{qD2} = (Ra.B/2) -
  = 65.31434 4.8
  = 97.68

Beban Hidup



B = 9.6 m

Beban terbagi rata (BTR) :

untuk B = 9.6 m

maka digunakan q =

q_{BTR} = q

= 9

Beban hidup garis (BGT):

p = 49 kN/m

P_{BGT} = p (1 +

→ DLA = 40%

P_{BGT} = 49 (1 +

= 137.20 kN

"D" = BTR + BGT

= 81 + 137.2

= 218.2 kN/m

- q1 = 100% x "D"
- q2 = 50% x "D"

- Σ MB = 0

9.6 Va = 218.2 5.5

9.6 Va = 5760.5

9.6 Va = 6022.32

Va = 627.33

- Mmax = (Va.B/2) -

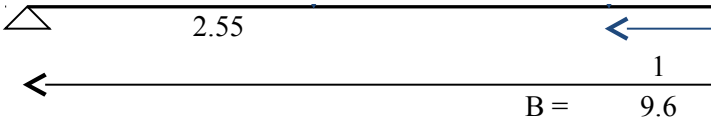
= [627.325 4.8]

= 2100.86

Beban Truck "T" :

Kondisi 1





T = 112.5 kN

Tu = T x (1+0,3) x K^{u}_{TT}

= 112.5 1.3 1.8

- Σ MB = 0

  9.6 Va = Tu ⎛ 7.05

  9.6 Va = 263 x

  9.6 Va = 5054.40

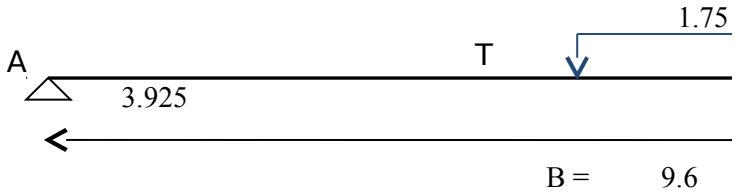
  Va = 526.50

- M_{max} L1 = (Va.B/2) -

  = ⎡ 526.5 4.8⎤

  = 1803.26

Kondisi 2



- Σ MB = 0

  9.6 Va = Tu ⎛ 5.675

  9.6 Va = 263 x

  9.6 Va = 2527.20

  Va = 263.25

- M_{max} L2 = (Va.B/2) -

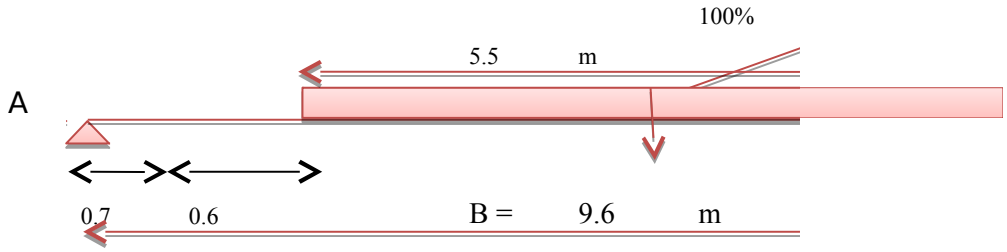
  = ⎡ 263.25 4.8⎤

  = 1033.26

Untuk Beban hidup, dipakai momen terbesar, yaitu

3.2.2 Kontrol Geser

Gaya geser maksimum terjadi jika BTR + BGT tidak simetris.



"D"	BTR	100%	=	81	kN/m
	BGT	100%	=	137.2	kN/m

- ***Untuk BTR***

Σ MB	=	0	
9.6 Va	=	(81	5.5
9.6 Va	=	1804	+
9.6 Va	=	1928.81	
Va	=	200.92	

- ***Untuk BGT***

Σ MB	=	0	
81 Va	=	(137	5.5
81 Va	=	3056	+
81 Va	=	3267.08	
Va	=	40.33	

Maka, Vu = $V_{qD1}MU$
= 175.64
= 416.89

3.2.3 Kontrol Penampang Profil

Pelat Sayap :

$$\lambda = \frac{bf}{2.tf} = \frac{302}{56} =$$

$$\lambda p = \frac{170}{\sqrt{fy}} = \frac{170}{\sqrt{360}} =$$

$\lambda < \lambda p \rightarrow$ Penampang Kompa

Pelat Badan :

$$\lambda = \frac{h}{tw} = \frac{652}{15} =$$

$$\lambda p = \frac{1680}{\sqrt{fy}} = \frac{1680}{\sqrt{360}} =$$

$\lambda < \lambda p \rightarrow$ Penampang Kompa

Karena Penampang Kompak, maka Mn = Mp

3.2.4 Kontrol Tegangan Lentur (Tekuk Lateral)

Lb = 1.5

Lp = 1,76 iy $\sqrt{\frac{E}{fy}}$

= 1.76 294 $\sqrt{\quad}$

= 11428.338 mm

Lb < Lp → Bentang Pendek

Karena bentang pendek, maka :

Mn = Mp

Mn = fy x Zx

= 410 x 7344220

φMn = 0.9 x

= 2710017180 Nmm

= 2710.0 kNm

Mu = Mq_{D1} +

= 421.54 +

= 2522.40 kNm

φMn ≥ Mu

→ 2710.0 kNm >

3.2.5 Menentukan Lebar Efektif Pelat Beton

- be1 ≤ S

≤ 4500 cm

- be2 ≤ $\frac{L}{4}$

≤ $\frac{9600}{4}$ = 2400

3.2.6 Cek Kreteria Penampang

$\frac{h}{tw}$ = $\frac{652}{15}$ = 43.46667

$\frac{1100}{\sqrt{fy}}$ = $\frac{1100}{\sqrt{240}}$ = 54.32513

Kapasitas momen penampang dianalisa dengan **distribusi tegan**

3.2.7 Menentukan "c"

Luas beton :

Ac = b_{eff} x tb

Mencari "c" :

c1 = As x fy

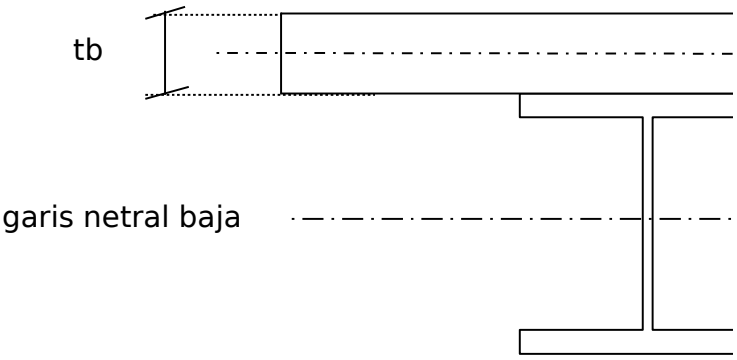
c2 = 0,85 x f'c x Ac

c3 = $\sum_{n-1}^{N}$ Qn

Maka, c diambil yang tekecil yaitu

3.2.8 Menentukan "c"

$$a = \frac{c}{0.85 \quad f'c \quad b_{eff}} =$$



d1 = tb - a/2 =

d2 = 0 → profil ba

d3 = D/2 = 708

3.2.9 Perhitungan Momen Positif

Mn = c (d1+d2) + Py (d3-d2)

= c (d1+d2) + (As x fy).(d3-d2)

= 11217600 (121.4454 +

= 5333356073.94958 Nmm

φMn = 0.9 x 533

= 4800020466.6 Nmm

= 4800.0 kNm

3.2.10 Kontrol Lendutan

$$f\,ijin = \frac{B}{800} = \frac{9600}{800} =$$

$$f° = \frac{5}{384} \times \frac{q_{L1}.\lambda^4}{E.Ix}$$

$$= \frac{5}{384} \times \frac{109.1}{200000}$$

= 1.23 mm

f ° < f ijin → 1.23

3.3 PERHITUNGAN SHEAR CONNECTOR

Untuk jarak perhitungan shear connector (BMS pasal 7.6.8.3) tid

- 600 mm
- 2 x tebal lantai
- 4 x tebal shear connector

Tinggi minimum dari paku shear connector adalah 75 mm dan j
diameter paku shear connector tidak boleh melebihi :

- 1,5 x tebal plat flens bila plat memikul tegangan tarik.
- 2,0 x tebal plat flens bila tidak terdapat tegangan tarik.

Digunakan shear connector jenis paku/ stud dengan data-data se

- Diameter
- Asc
- Tinggi total
- Jarak melintang antar stud
- Kuat beton (fc')

→ σ_c

- Mutu baja stud (fu)

3.3.1 Kekuatan Stud Connector (Q)

Ec = $W^{1,5}$ 0.041 fc'

= 2400 1.5 0.041

= 168720.85 Mpa

Qn = 0.5 Asc ⎛ fc' Ec

= 0.5 641.077 ⎛ 35

= 778930.609306888 N

Asc x fu = 641.0772453425 x 520 =

Qn ≥ Asc x fu →

Vh = c = 11217600

3.3.2 Jumlah stud connector :

$$n = \frac{Vh}{Asc\ fu} = \frac{\quad}{\quad}$$

Jadi, jumlah shear connector stud yang dibutuhkan disepanjang l

→ n x 2 = 33.7

Jarak pemasangan stud connector :

→ $\frac{1000}{68}$ = 14.7059 ≈ 15

3.4 PERHITUNGAN SAMBUNGAN BALOK MEMANJANG I

Sambungan balok memanjang dan melintang menggunakan baut

3.4.1 Sambungan siku pada balok memanjang (2 bidang geser)

- Gaya geser max. : Vu
- Profil :
- Rencana :
  - → Baut ØM$_{22}$ =
    Ab =
  - → Pelat Penyambung

Baut mutu tinggi tipe gesek

- Plat bersih μ =
- Lubang baut Ø =
- Bidang geser m =

Vh = 1.13 μ

= 1.13 0.35

Vd = Ø Vh

$$n = \frac{Vu}{Vd} =$$

Pu	≤	n	Vd
416893.8		≤	22
416893.8		<	

3.4.2 Sambungan siku pada balok melintang (1 bidang geser)

- Gaya geser max. : Vu
- Profil : WF
- Rencana :
  - → Baut $\text{Ø}M_{22}$ =
    Ab =
  - → Pelat Penyambung

Baut mutu tinggi tipe gesek

- Plat bersih μ =
- Lubang baut Ø =
- Bidang geser m =

Vh	=	1.13	μ
	=	1.13	0.35

Vd = Ø Vh

$$n = \frac{Vu}{Vd} =$$

Pu	≤	n	Vd
416893.8		≤	43
416893.8		<	

3.4.3 Kontrol Pelat Siku

Dipakai : L 50 x 50

Luas Geser = Lmv t_L

Anv = (L - n d1) t_L

= [400 - 3

= 1670 mm^2

Kuat rencana :

Ø Pn = Ø 0.6 fu

= 0.75 0.6 520

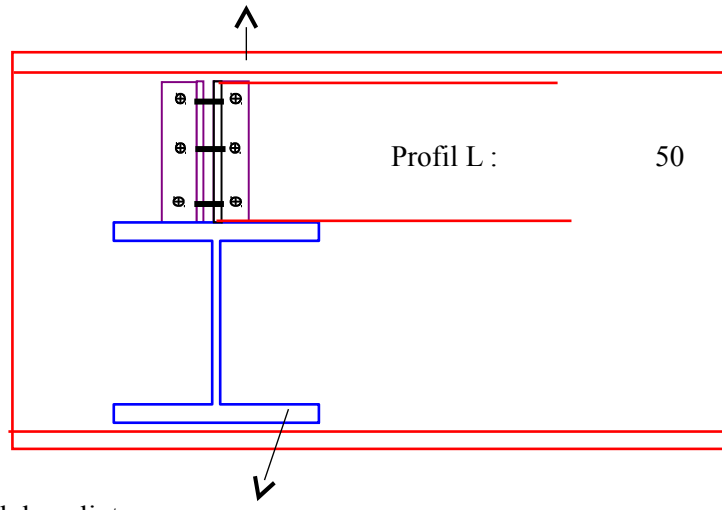
= 390780 N

Ada 2 siku :

Ø Pn = 2

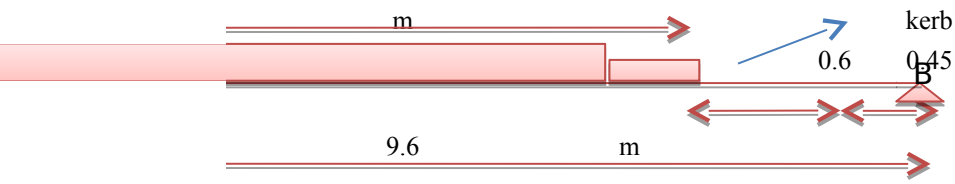
= 781560

Balok memanjang =



Balok melintang =

BAB 3
PERENCANAAN BALOK MELINTANG

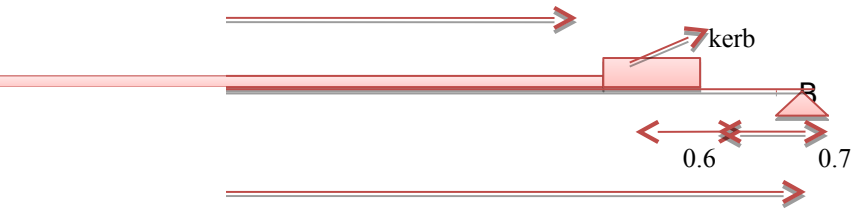


WF 700 x 300 x 215

	Ix	=	237000	cm⁴	Baja
	Iy	=	12900	cm ⁴	fy =
	ix	=	29.4	cm	fu =
	iy	=	6.86	cm	E =
	hw	=	652	mm	
kN/m	hf	=	680	mm	

' b1) x LF

4.5	1.7	x	1.1	=	5.247 kN/m	
ɔfil x LF						
x	1.1			=	2.365 kN/m	
ton x λ x LF						
24	4.5	1.3		=	28.08 kN/m	
ɲ x LF						
x	1.8			=	0.9 kN/m	+
	q_{D1}			=	36.59 kN/m	



pal x λ x LF

22 4.5 1.3 = 6.435 kN/m

ton x λ x LF

24 4.5 1.3 = 70.2 kN/m +

q_{D2} = 76.635 kN/m

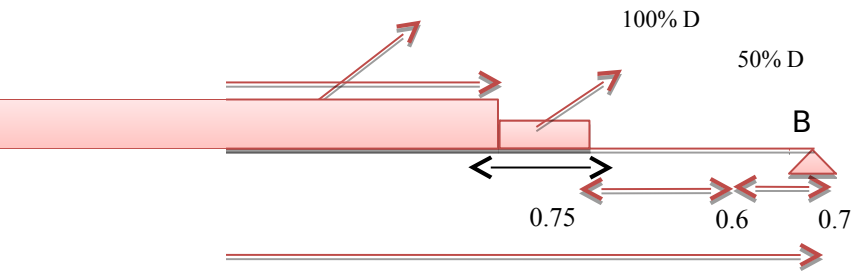
8.6 + 6.435 7.5 5.05 + 70.2 0.3 1

243.7 + 21.06

) − [70.2 0.6 4.05] − [6.435 3.75 1.875]

) − 170.586 − 45.2461

kNm



7 = lebar 2 jalur kendaraan. (RSNI T

Distribusi beban (RSNI T 02-2005 Pasal

(RSNI T 02-2005 pasal 3.7)

<	L	=	30	m	2.9	1.45
9	Kpa	=	9	kN/m²		
λ	K^{u}_{TD}					
4.5	2	=	81	kN/m		

(RSNI T 02-2005 pasal 6.6)

DLA	) x	K^{u}_{TD}			
untuk	*L*	=	40.5	*m*	(gambar 8 RSNI T 02-2005)
40%	) x	2			

34.3

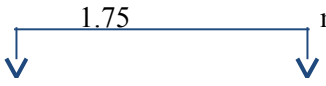
=	100 % x	218.2	=	218.2	kN/m		
=	50% x	218.2	=	109.1	kN/m		

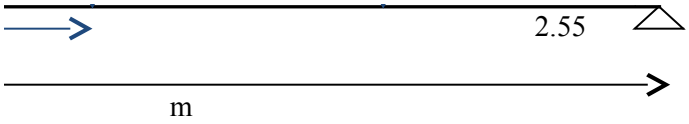
4.8	+	109.1	0.25	7.925	+	109.1	0.25	1.675
+	216.15		+	45.686				

kN

[	109.1	0.25	3.125]	–	[218.2	2.75	1.375]	
–	85.2344	–	825.069					

kNm





(RSNI T 02-2005 pasal 6.4.1)

(0,3 = DLA)

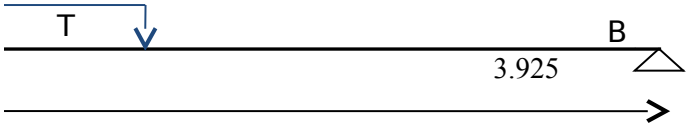
= 263.25 kN

+ 5.3 + 4.3 + 2.55]
19.2

kN

[Tu 2.3 + 0.5]
– [263.25 2.8]
kNm

m

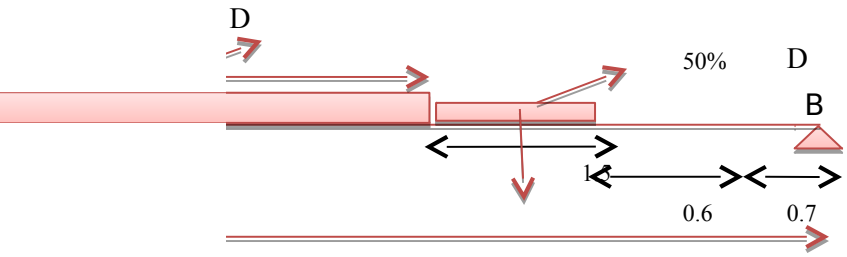


+ 3.925]
9.6

kN

Tu [0.9]
– [263.25 0.9]
kNm

= ***2100.86***



BTR	50%	=	40.5	kN/m
BGT	50%	=	68.6	kN/m

4.05] + [40.5 1.5 2.05]

124.5

kN

4.05] + [68.6 1.5 2.05]

210.9

kN

+	Va (BTR)	+	Va (BGT)
+	200.92	+	40.33

kN

5.39

8.40

ak **OK**

43.4666666667

82.97

ak **OK**

$$\frac{200000}{410}$$

= 3011130200 Nmm

3011130200

Mc
2101

2522.40 kNm **OK**

Dimana :

S = Jarak antar gelagar melintang

L = Lebar jembatan

mm

$$\frac{h}{tw} < \frac{1100}{\sqrt{fy}} \rightarrow$$ Plastis

gan plastis.

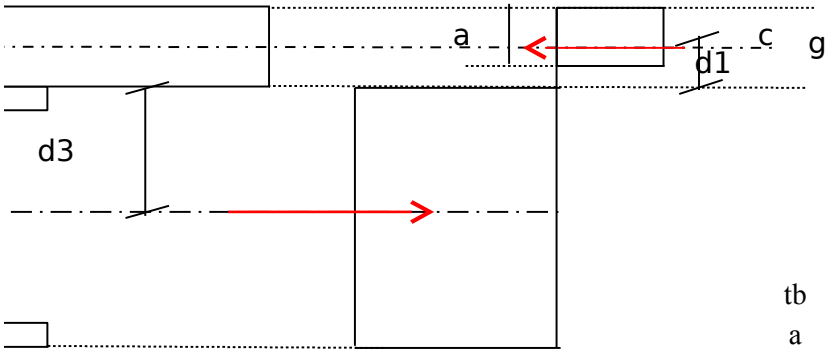
= 2400 x 200 = 480000 mm²

= 27360 x 410 = 11217600.00 N
= 0.85 35 480000 = 14280000.00 N

→ untuk komposit penuh c3 tidakmenentukan.

= ***11217600.00*** ***N***

$$\frac{11217600}{0.85 \quad 35 \quad 2400} = 157.11 \text{ mm}$$



tb = 200 mm
a = 157.1092437 mm

200 − 157.109 0.5 = 121.45 mm

ja tidak mengalami tekan

x 0.5 = 354 mm

0] + 11217600 [354 − 0]

3356073.94958

12 *(SNI T 03-2005 4.7.2)*

$$\frac{x \quad 4500^4}{x \quad 2370000000}$$

< 12 **OK**

dak boleh melebihi nilai sebagai berikut :

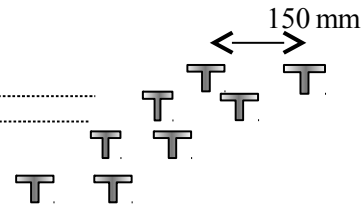
arak antara paku shear connector dengan ujung flens gelagar tidak boleh kurang dari 25 mm. Untuk

bagai berikut :

=	28.57 mm	<	1,5 x	0	=		0 mm	
=	641.077 mm²							
=	100 mm							
=	150 mm							
=	35 Mpa							
=	0,4 x fc'	=	0.4	35	=	14	Mpa	
=	520 Mpa							

35

100 mm



0.5

168720.85 0.5

333360.167578118 N

778930.609306888 N > **333360.167578** N **OK**

N

$$\frac{11217600}{333360.2} = 33.6501$$

balok adalah :

x 2 = 67.3002 ≈ 68 buah

cm

DAN MELINTANG

: mutu tinggi tipe gesek.

= 416893.83 N

WF 350 x 350 x 159

22 cm → Tb = 25000 N

183984.232164833 mm^2

: t = 20 mm

0.35

1

2

m Tb

2 25000 = 19775 N

= 1 x 19775 = 19775 N

$$\frac{416893.8}{19775} = 21.0819 \rightarrow 22 \text{ buah}$$

19775

435050 **OK**

= 416893.83 N

WF 700 x 300 x 215

22 mm → Tb = 25000 N

183890.96 mm^2

: t = 20 cm

0.35

1

1

m Tb

1 25000 = 9887.5 N

= 9887.5 N

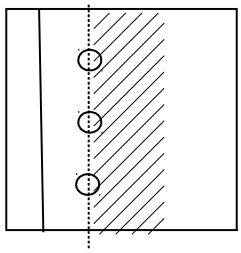
$$\frac{416893.8}{9887.5} = 42.1637 \rightarrow 43 \text{ buah}$$

9887.5

425162.5 **OK**

x 5

] 22 5



400

Anv
1670

390780 ≥ Vu
N > 416893.83 N **OK**

WF 350 x 350 x 159



x 50 x 5

WF 700 x 300 x 215

Bj 52

410 MPa

520 MPa

200000 MPa

Shelvy Elvina Santoso - 3113041059

02-2005 pasal 6.2)

6.3.2)

4.0571

kNm

Shelvy Elvina Santoso - 3113041059

Shelvy Elvina Santoso - 3113041059

Shelvy Elvina Santoso - 3113041059

Shelvy Elvina Santoso - 3113041059

Shelvy Elvina Santoso - 3113041059

Shelvy Elvina Santoso - 3113041059

Shelvy Elvina Santoso - 3113041059

BAB 4
SAMBUNGAN BALOK MEMANJANG DAN MELINTANG

Sambungan balok memanjang dan melintang menggunakan baut mutu tinggi tipe gesek.

4.1. Sambungan siku pada balok memanjang (2 bidang geser)

- Gaya geser max. : Vu = 17,564.16 kg
- Profil : WF588.300
- Rencana :
  - → Baut ØM$_{22}$ = 2.2 cm → Tb =
    Ab = 3.7994 cm^2
  - → Pelat Penyambung : t = 2

4.2. *Baut mutu tinggi tipe gesek*

- Plat bersih μ = 0.35
- Lubang baut Ø = 1
- Bidang geser m = 2

Vh = 1.13 μ m Tb
= 1.13 0.35 2 27270 = 2157

Vd = Ø Vh = 1 x 21570.57

$$n = \frac{Vu}{Vd} = \frac{17564.2}{21570.57} = 0.81426 \rightarrow$$

Pu ≤ n Vd
17564.2 ≤ 1 21570.57
17564.2 < 21570.57 **OK**

4.3. Sambungan siku pada balok melintang (1 bidang geser)

- Gaya geser max. : Vu = 17564.16 kg
- Profil : WF900.300
- Rencana :
  - → Baut ØM$_{22}$ = 2.2 cm → Tb =
    Ab = 18.3891 cm^2
  - → Pelat Penyambung : t = 2

4.4. *Baut mutu tinggi tipe geser*

- Plat bersih μ = 0.35
- Lubang baut Ø = 1
- Bidang geser m = 1

Vh = 1.13 μ m Tb
= 1.13 0.35 1 27270 = 1078

Vd = Ø Vh = 10785.285 kg

$$n = \frac{Vu}{Vd} = \frac{17564.2}{10785.285} = 1.62853 \rightarrow$$

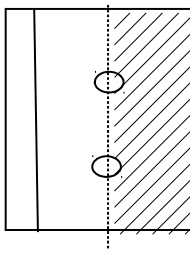
$$Pu \leq n \; Vd$$

$$17564.2 \leq 2 \; 10785.285$$

$$17564.2 < 21570.57 \quad \textbf{OK}$$

4.5. Kontrol Pelat Siku

Dipakai : L 50 x 50 x 5



Luas Geser = Lmv t_L

Anv = (L - n d1) t_L

= [50 - 2 2.2] 0.5

= 22.8 cm^2

Kuat rencana :

Ø Pn = Ø 0.6 fu Anv

= 0.75 0.6 5200 22.8

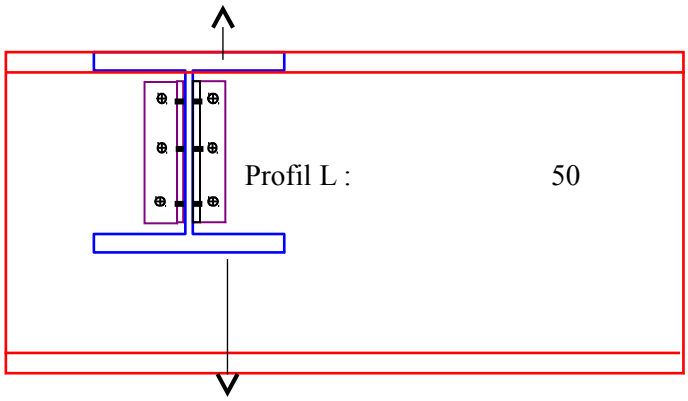
= 53352 kg

Ada 2 siku :

Ø Pn = 2 53352 ≥ Vu

= 106704 kg ≥ 17564.16 kg

Balok memanjang = WF 350 x 350 x 159



Balok melintang = WF 700 x 300 x 215

27270 kg

cm

'0.57 kg
= 21570.6 kg

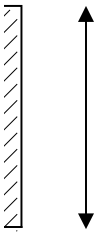
1 buah

27270 kg

cm

5.285 kg

2 buah



500

OK

BAB 5
PERENCANAAN IKATAN ANGIN

5.1. PERHITUNGAN IKATAN ANGIN (BMS 2.4.6)

Diketahui bahwa letak jembatan ≤ 5 km dari pantai.

Maka kecepatan angin rencana, Vw = 35 m/s

Angin bekerja pada bangunan atas jembatan (rangka) dan bidang samping kendaraan.

5.1.1. Gaya Nominal Ultimit pada Bangunan Atas

$T_{EW} = 0.0006\ Cw\ Vw^2\ Ab$

Dimana :

Cw = Koefisien seret ...*BMS*

= 1.2 (rangka batang)

Vw = Kecepatan angin rencana

Ab = Luas jembatan rangka yang terkena angin

= 30% luas total bagian yang masif dalam arah tegak lurus sumbu memanjang jembatan. .....*BMS*

$$Ab = 30\% \left(8\ \lambda + 7\ \lambda \right) \frac{d2}{2}$$

$$= 30\% \left(8\ 4.5 + 7\ 4.5 \right) \frac{6}{2} = 60.75\ m^2$$

$T_{EW1} = 0.0006\ Cw\ Vw^2\ Ab$

$= 0.0006\ \ 1.2\ \ 35^2\ \ 60.75 = 53.582$ kN/m

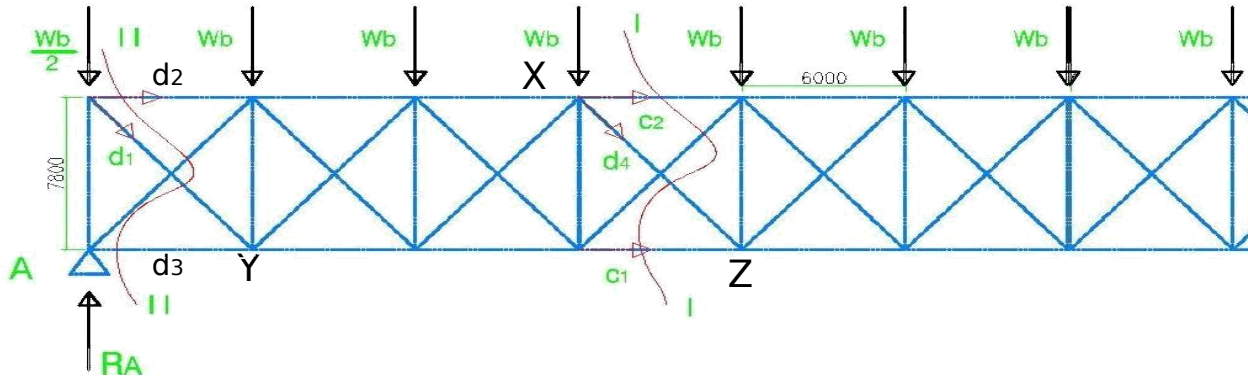
5.1.2. Gaya Nominal Ultimit pada Kendaraan Sepanjang Jembatan

Bekerja pada LK --> hanya dipikul oleh ikatan angin bawah sebagai beban merata sepanjang je

$T_{EW2} = 0.0012\ Cw\ Vw^2$

$= 0.0012\ \ 1.2\ \ 35^2 = 1.764$ kN/m

5.2. IKATAN ANGIN ATAS



5.2.1. Beban pada titik simpul :

$$Wb = \frac{1}{n} T_{EW1} + \lambda\ T_{EW2} = \frac{1}{8} 53.582 + 4.5$$

= 14.64 kN = 1464 kg

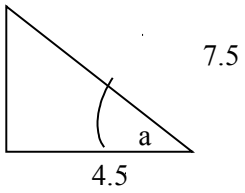
5.2.2. Reaksi Perletakan :

R_A = 3.5 Wb

= 3.5 14.636

= 51.225 kN

= 5227.031 kg



Mencari Gaya Batang dengan metode Ritter

- **c1** (c2, d4) Berpotongan di X → Σ MX = 0

→ RA 14 − 731.78 14 − 1464 9 − 1464 4.5 −

→ 70564.921875 − 9879.09 − 13172.11875 − 6586.059375 =

Maka, **c1** = $\frac{40927.65}{6.0}$ = 6821.27578125 kg **(Tarik)**

- **c2** (d4, c1) Berpotongan di Z → Σ MZ = 0

→ RA 20 − 731.78 18 − 1464 14 − 1464 9 −

c2 6.0 = 0

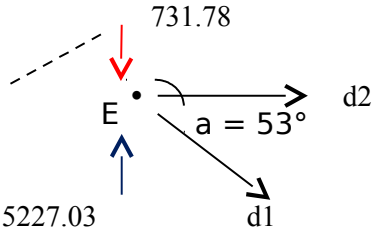
Maka, **c2** = $\frac{49238.63}{-6.0}$ = -8206.44 kg **(Tekan)**

- **d4** (c2, c1) Tidak berpotongan → Σ V = 0

→ RA − 731.78 − 1464 − 1464 − 1464 − d4 sin a

Maka, **d4** = $\frac{104.54}{0.8}$ = 130.90 kg **(Tarik)**

Mencari Gaya Batang d1 dan d2 dengan metode Titik Simpul



→ Σ V = 0

5227.03 − 731.78 − d1

d1 = $\frac{4495.25}{0.80}$ = 562

→ Σ H = 0

d1 cos a + d2 = 0

d2 = -5628.66 0.6018

= -3387.41 kg **(Tekan**

5.2.3 Dimensi Ikatan Angin Atas

Syarat Kelangsingan

$\frac{Lk}{\quad}$ → Lk = $\sqrt{6^2 + 4.5^2}$

i_{min} < 300

$$i_{min} = \frac{Lk}{300} = \frac{750}{300} = 2.5 \rightarrow \mathbf{i_{min} > 2.50}$$

Perencanaan Profil

Direncanakan Profil L : 250 x 175 x 11

A	=	56.20	cm^2			fy	=	410 Mpa
w	=	44.1	Kg/m			Beban tarik Pu		→
Ix	=	Iy	=	6120	cm^4			=
ix	=	iy	=	10.4	cm	Ø baut		=
b	=	175	mm			Ø lubang		=
d	=	32	mm					=
e	=	5.02	cm			Tb		=

Kontrol Kelangsingan

$$\lambda_{max} = \frac{Lk}{iy} = \frac{750.00}{10.4} = 72.115 < 300 \quad \mathbf{OK}$$

Kontrol Kekuatan Batang Tarik

- **Batas Leleh**

Pu = Ø x fy x Ag

= 0.9 x 4100 x 56.20

= 207378 kg > 6821.28 kg **OK**

- **Batas Putus**

$An = Ag - \left[1 \; d_{baut} \; t_f \right]$

$= 56.20 - 1 \; 1.6 \; 1.1 = 54 \; cm^2$

$$U = 1 - \frac{e}{d} = 1 - \frac{5.02}{32} = 0.8431$$

Ae = U An

= 0.84 54 = 45.9 cm^2

Pu ≤ Ø fu Ae = 0.75 520 45.9

= 17900.9 kg > 682

Baut mutu tinggi tipe gesek

- Plat bersih μ = 0.35
- Lubang baut Ø = 1
- Bidang geser m = 1

Vh = 1.13 μ m Tb

= 1.13 0.35 1 6000 = 2373

Vd = Ø Vh = 2373 kg

$$n = \frac{Vu}{Vd} = \frac{6821.3}{2373} = 2.8745 \rightarrow 3$$

Pu ≤ n Vd

6821.3 ≤ 3 2373

6821.3 < 7119 **OK**

Kontrol Pelat Siku L 250 175 11

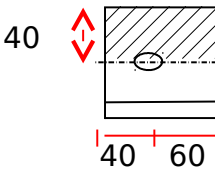
Luas Geser = Lmv t_L

Anv = (L - n d1) t_L

= [11 - 2.5 1.6] 1.1

= 7.7 cm^2

n =



Kuat rencana

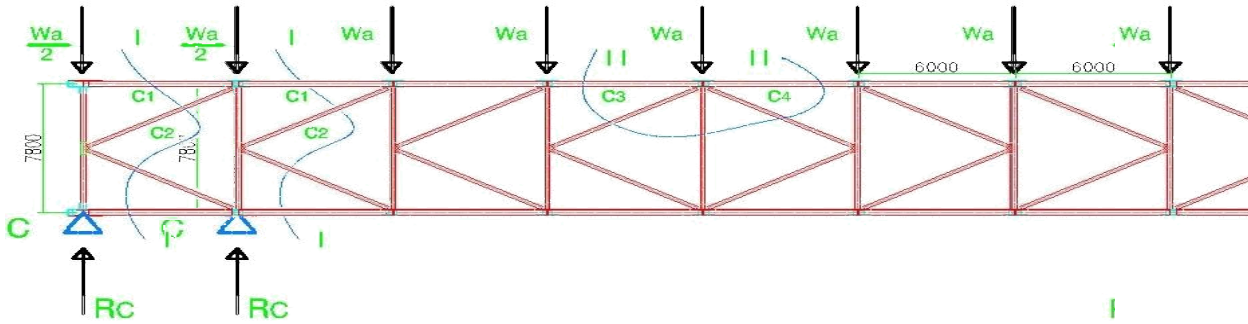
Ø Pn = Ø 0.6 fu Anv

= 0.75 0.6 520 7.7

= 1802 kg

Ø Pn = 1802 kg < Pu = 6821.28 kg

5.3. IKATAN ANGIN BAWAH



5.3.1 Beban pada titik simpul :

$$Wa = \frac{1}{n} T_{EW1} = \frac{1}{8} 53.582 = 6.70 \text{ kN}$$

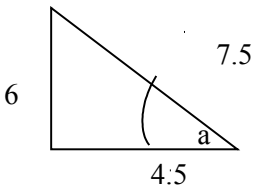
4.3.2 Reaksi Perletakan :

R_C = 3 Wa

= 3 6.70

= 20.09 kN

= 2009.306 kg



Potongan I - I

ΣVo = 0

0 = c1 sin a — c2 sin a + R_C — Wa/2

0 = c1 0.80 — c2 0.7986 + 20.09 — 3.3488

-16.74 = c1 0.80 — c2 0.7986 ……..persamaan 1

`

ΣHo = 0

0 = c1 cos a c2 cos a

0 = c1 0.60 c2 0.6018

c1 = (-) c2 ……..persamaan 2

Substitusi persamaan 1 dan 2

-16.74 = c1 0.80 - c2 0.7986

-16.74 = 1.60 C1

c1 = -10.48302 kN **(Tekan)**

c2 = 10.483017 kN **(Tarik)**

5.3.3. Dimensi Ikatan Angin Bawah

Syarat Kelangsingan

$\frac{Lk}{i_{min}} < 300$ → $Lk = \sqrt{3^2 + 4.5^2}$

$i_{min} = \frac{Lk}{300} = \frac{540.8}{300} = 1.8028$ → **i_{min} > 1.80**

Perencanaan Profil

Direncanakan Profil L : 180 x 180 x 16

A	=	55.40	cm^2			fy	=	410	Mpa
w	=	43.5	Kg/m			Beban tarik Pu			→
Ix	=	Iy	=	1680	cm^4				=
ix	=	iy	=	5.51	cm	Ø baut			=
b	=	180	mm			Ø lubang			=
d	=	16	mm						=
e	=	5.02	cm			Tb			=

Kontrol Kelangsingan

$\lambda_{max} = \frac{Lk}{Iy} = \frac{540.83}{1680} = 0.3219 < 300$ **OK**

Kontrol Kekuatan Batang Tarik

- **Batas Leleh**

Pu = Ø x fy x Ag

= 0.9 x 4100 x 55.40

= 204426 kg > 1048.30 kg **OK**

- **Batas Putus**

$An = Ag - \{ 1 \; d_{baut} \; t_f \}$

$= 55.40 - \{ 1 \; 1.6 \; 1.6 \} = 53 \; cm^2$

$U = 1 - \frac{e}{L} = 1 - \frac{5.02}{180} = 0.9721$

$Ae = U \; An = 0.97 \; 53 = 51.366 \; cm^2$

$Pu \leq Ø \; fu \; Ae = 0.75 \; 520 \; 51.366$

$= 20032.9 \; kg > 104$

Baut mutu tinggi tipe gesek

- Plat bersih μ = 0.35
- Lubang baut Ø = 1
- Bidang geser m = 1

$Vh = 1.13 \; \mu \; m \; Tb$

$= 1.13 \; 0.35 \; 1 \; 6000 = 2373$

$Vd = Ø \; Vh = 2373 \; kg$

$n = \frac{Vu}{Vd} = \frac{1048.3}{2373} = 1.44 \rightarrow 2$

$Pu \leq n \; Vd$

$c1 \leq 2 \; 2373$

$c1 < 4746$ **OK**

Kontrol Pelat Siku L 180 180 16

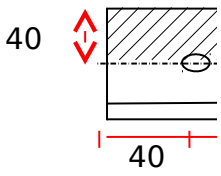
Luas Geser = Lmv t_L

Anv = (L - n d1) t_L

= { 16 - 1.5 1.6} 1.6

= 21.76 cm^2

n =



Kuat rencana

Ø Pn = Ø 0.6 fu Anv

= 0.75 0.6 520 21.76

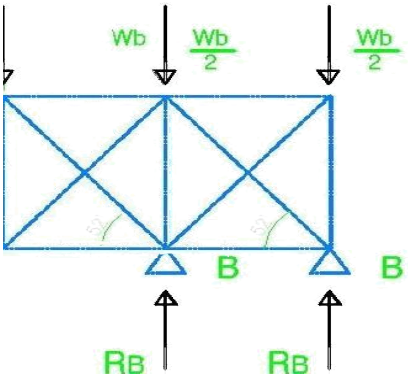
= 5092 kg

Ø Pn = 5092 kg ≥ 1048 = c1 kg

Tabel 2.9

S 2.4.6

mbatan.



1.764

a = 53 °
sin a = 0.7986
cos a = 0.6018

c1 6.0 = 0
c1 6.0

1464 4.5 +

= 0

sin a = 0
8.66 kg **(Tarik)**

)

= 7.5 m

= 750 cm

cm

= 4100 kg/cm²

c1

6821.28 kg

16 mm

16 + 1.6

17.6 mm

6000

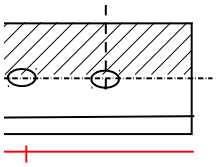
< 0.9

1.28 kg **OK**

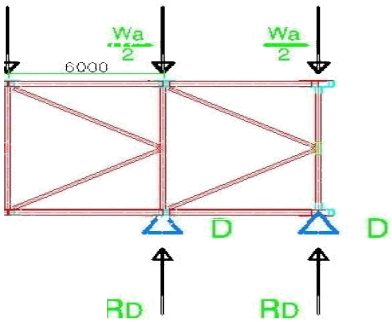
kg

buah

2.5



Ok



a	=	53	1
sin a	=	0.7986	
cos a	=	0.6018	

= 5.408 m

= 540.8 cm

cm

= 4100 kg/cm²

c1

1048.30 kg

16 mm

16 + 1.6

17.6 mm

6000

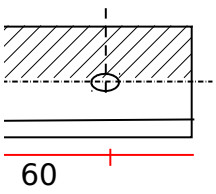
> 0.90

8.30 kg **OK**

kg

buah

1.5



OK

BAB 6. PERENCANAAN RANGKA BATANG UTAMA

6.1. Data Perencanaan

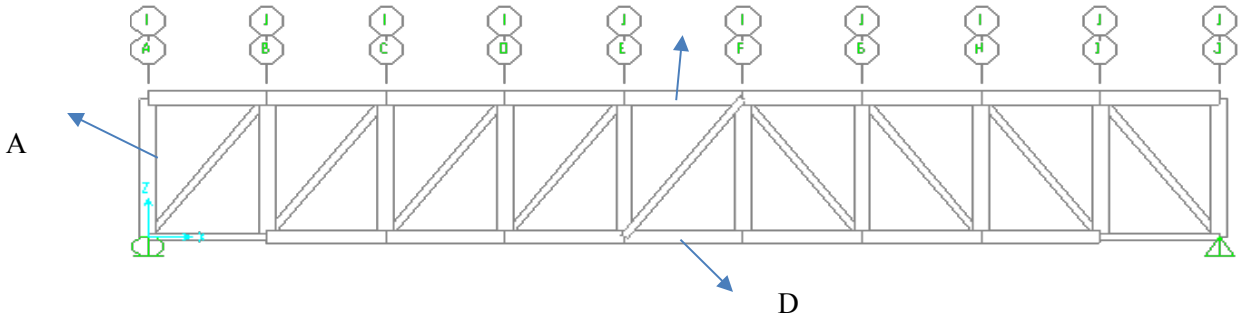
Mutu Bahan Baja Bj 52

fy = 410 MPa fu = 520 MPa E = 200000 MPa

Mutu Baja yang digunakan : Bj 52

fy = 410 MPa fu = 520 MPa E = 200000 MPa

Rangka Batang yang ditinjau : B



λ = jarak horizontal antar titik buhul = 4.5 m

h = tinggi rangka = 6 m

tinggi rangka diagonal = 7.5 m

Besar Gaya Batang yang Ditinjau

Seperti yang terlihat pada gambar model SAP di atas, gaya-gaya terbesar berada pada batang yang berlabel. (hasil SAP terlampir)

Batang A:	(Horizontal Bawah)			
	Nu_{aksial}	= 2839.53	kN	(tarik)
Batang B:	(Horizontal Atas)			
	Nu_{aksial}	= 2048.49	kN	(tekan)
Batang C:	(Diagonal Tepi)			
	Nu_{aksial}	= 1772.34	kN	(tekan)
Batang D:	(Vertikal Tepi)			
	Nu_{aksial}	= 1254.65	kN	(tarik)

6.2. Kontrol Kekuatan Batang

6.2.1. Batang Horizontal bawah, batang A

Digunakan Profil : WF 400 x 400 x 605

h	b	tw	tf	r	A	W	Ix	Iy	ix	iy	Sx
mm	mm	mm	mm	mm	cm^2	kg/m	cm^4	cm^4	cm	cm	cm^3
498	432	45	70	22	770.1	605	298000	94400	19.7	11.1	12000

Nu_{aksial} = 2839.53 kN

Panjang Batang : 5500 mm

Kontrol Batang Tarik

a. Kontrol Kelangsingan

Faktor tekuk rangka batang : k = 1

L = 5500 mm

$\lambda = \frac{L}{r} = \frac{5500}{197} = 27.9188 < 240$ (RSNI T - 03 - 2005 Ps 5.3

b. Kekuatan Nominal

Mutu baut: BJ 50

fy = 290 MPa diameter baut, d =

fu = 500 MPa diameter lubang =

- Kondisi Leleh

ØNn = 0.9 Ag fy

= 0.9 77010 410

= 28416690 N

ØNn > Nu

28416690 N > 2839530 N ***OK***

- Kondisi Putus

profil disampbung pada pelat setebal : 70 mm

Menentukan Anet

Anet 1 = Ag - n d t

= 77010 - 1 22 70 = 754

Anet = 75470 mm^2 > 85%Ag = 6545

$U = 1 - \frac{x}{L} \leq 0.9$

Dimana ,

x = eksentrisitas sambungan

L = panjang sambungan = 432 mm

$U = 1 - \frac{61.19 \text{ mm}}{432}$

U = 0.8584

Ae = U Anet = 0.8583624 75470 = 64780.613

ØNn = 0.75 Ae fu = 24292729.96332 N

= 24292.72996332 KN ***OK***

6.2.2. Batang Horizontal atas, batang B

Digunakan Profil : WF 400 x 300 x 94,3

h	b	tw	tf	r	A	W	Ix	Iy	ix	iy	Sx
mm	mm	mm	mm	mm	cm^2	kg/m	cm^4	cm^4	cm	cm	cm^3
386	299	9	14	22	120.1	94.3	33700	6240	16.7	7.21	1740

Nu_{aksial} = 2048.49 kN

Panjang Batang = 4500 mm

Kontrol Batang Tekan

a. Kontrol Penampang

- Sayap

$$\frac{bf}{2tf} < \lambda_p \qquad \frac{299}{28} < \frac{250}{fy^{0.5}} \qquad 10.7 < 12.35$$

- Badan

$$\frac{h}{tw} < \lambda_p \qquad \frac{386}{9} < \frac{665}{fy^{0.5}} \qquad 42.9 < 32.84$$

Jadi penampang profil kompak, maka Mn= Mp

b. Kelangsingan Batang

Faktor tekuk rangka batang : k = 1

- Arah Sumbu X

Lkx = k L = 4500 mm

$$\lambda x = \frac{Lx}{rx} = \frac{4500}{167} = 26.9461 < 140$$

(RSNI T - 03 - 20(

- Arah Sumbu Y

Lkx = k L = 4500 mm

$$\lambda y = \frac{Ly}{ry} = \frac{4500}{72.1} = 62.4133 < 140$$

(RSNI T - 03 - 20(

c. Kekuatan Nominal Terfaktor Batang

- Arah Sumbu X

$$\lambda c = \frac{\lambda x}{\pi} \times \sqrt{\frac{fy}{E}} = \frac{26.946108}{\pi} \times \sqrt{\frac{410}{200000}} =$$

ω = 1.06732

$$Nn = Ag \frac{fy}{\omega}$$

$$Nn = 12010 \frac{410}{1.0673} = 4613522.3 \text{ N} = 4613.$$

- Arah Sumbu Y

$$\lambda c = \frac{\lambda y}{\pi} x \sqrt{\frac{fy}{E}} = \frac{62.413315}{\pi} x \sqrt{\frac{410}{200000}} =$$

$$\omega = 1.43383$$

$$Nn = Ag \frac{fy}{\omega}$$

$$Nn = 12010 \frac{410}{1.4338} = 3434234.8411628 \text{ N} = 3434.$$

d. Kontrol Kapasitas Batang

Nn yang menentukan adalah, Nn = 3434.23 kN (Arah Sun

Ø = 0.85

ØNn = 2919.099614988 kN

Nu = 2048.49 kN < ØNn = 2919.09961499 kN

6.2.3 Batang diagonal, batang C

Digunakan Profil : WF 400 x 400 x 197

h	b	tw	tf	r	A	W	Ix	Iy	ix	iy	Sx
mm	mm	mm	mm	mm	cm^2	kg/m	cm^4	cm^4	cm	cm	cm^3
400	408	21	21	22	250.7	197	70900	23800	16.8	9.75	3540

Nu_{aksial} = 1772.34 kN

7800 mm

Kontrol Batang Tekan

a. Kontrol Penampang

- Sayap

$$l \pounds lp$$

$$\frac{bf}{2tf} < l_p$$

$$\frac{408}{42} < \frac{250}{fy^{0.5}}$$

$$9.7 < 12.35$$

- Badan

$$l \pounds lp$$

$$\frac{h}{tw} < l_p$$

$$\frac{400}{21} < \frac{665}{fy^{0.5}}$$

$$19.0 < 32.84$$

Jadi penampang profil kompak, maka Mn= Mp

b. Kelangsingan Batang

Faktor tekuk rangka batang : k = 1

- Arah Sumbu X

Lkx = k L = 7500 mm

$\lambda x = \frac{Lx}{rx} = \frac{7500}{168} = 44.6429 < 140$ (RSNI T - 03 - 20(

- Arah Sumbu Y

Lkx = k L = 7800 mm

$\lambda y = \frac{Ly}{ry} = \frac{7800}{97.5} = 80 < 140$ (RSNI T - 03 - 20(

c. Kekuatan Nominal Terfaktor Batang

- Arah Sumbu X

$\lambda c = \frac{\lambda x}{\pi} \times \sqrt{\frac{fy}{E}} = \frac{44.642857}{\pi} \times \sqrt{\frac{410}{200000}} =$

$\omega = 1.22335$

$Nn = Ag \frac{fy}{\omega}$

$Nn = 25070 \frac{410}{1.2233} = 8402111.3$ N = 8402

- Arah Sumbu Y

$\lambda c = \frac{\lambda y}{\pi} \times \sqrt{\frac{fy}{E}} = \frac{80}{\pi} \times \sqrt{\frac{410}{200000}} =$

$\omega = 1.72807$

$Nn = Ag \frac{fy}{\omega}$

$Nn = 25070 \frac{410}{1.7281} = 5948073.88810615$ N = 5948

d. Kontrol Kapasitas Batang

Nn yang menentukan adalah, Nn = 5948.07 kN (Arah Sun

Ø = 0.85

ØNn = 5055.86280489 kN

Nu = 1772.34 kN < ØNn = 5055.86280489 kN

6.2.4 Batang Vertikal Tepi, batang D

Digunakan Profil : WF 400 x 300 x 94,3

h	b	tw	tf	r	A	W	Ix	Iy	ix	iy	Sx
mm	mm	mm	mm	mm	cm^2	kg/m	cm^4	cm^4	cm	cm	cm^3

386	299	9	14	22	120.1	94.3	33700	6240	16.7	7.21	1740

Nu_{aksial} = 1254.65 kN

Panjang Batang : 4500 mm

Kontrol Batang Tarik

a. Kontrol Kelangsingan

Faktor tekuk rangka batang : k = 1

L = 5500 mm

$\lambda = \frac{L}{r} = \frac{5500}{167} = 32.9341 < 240$ (RSNI T - 03 - 2005 Ps 5.3

b. Kekuatan Nominal

Mutu baut: BJ 50

fy = 290 MPa diameter baut, d =

fu = 500 MPa diameter lubang =

- Kondisi Leleh

ØNn = 0.9 Ag fy

= 0.9 12010 410

= 4431690 N

ØNn > Nu

4431690 N > 1254650 N ***OK***

- Kondisi Putus

profil disampbung pada pelat setebal : 14 mm

Menentukan Anet

Anet 1 = Ag - n d t

= 12010 - 1 22 14 = 117(

Anet = 11702 mm^2 > 85%Ag = 1020

$U = 1 - \frac{x}{L} \leq 0.9$

Dimana ,

x = eksentrisitas sambungan

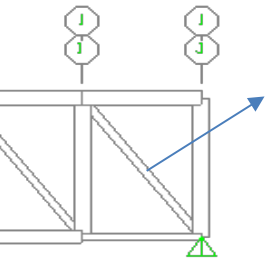
L = panjang sambungan = 299 mm

$U = 1 - \frac{33.82 \text{ mm}}{299}$

U = 0.8869

Ae = U Anet = 0.8868977 11702 = 10378.477

ØNn	=	0.75	Ae	fu	=	3891928.973541	N
					=	3891.928973541	KN **OK**



C

Sy
cm^3
4370

3.5)

20 mm
22 mm

70 mm^2

8.5 mm^2

3235518 mm^2

Sy
cm^3
418

)5 Ps 6.1)

)5 Ps 6.1)

0.3883

.52 kN

0.8995

.23 kN

ɹbu Y)

OK

Sy
cm^3
1170

)5 Ps 6.1)

)5 Ps 6.1)

0.6434

.11 kN

1.153

.07 kN

ɪbu Y)

OK

Sy
cm^3

418

).5)

20 mm
22 mm

)2 mm^2

8.5 mm^2

'262775 mm^2

Jenis Baja	fu	fy	peregangan minimum (%)
Bj 34	340	210	22
BJ 37	370	240	20
Bj 41	410	250	18
Bj 50	500	290	16
Bj 52	520	410	
Bj 55	550	410	13

1	2	3	4	5	6	7	8	9	10
Profil WF	Weight (kg/m')	h (mm)	b (mm)	tw (mm)	tf (mm)	r	A (cm²)	Ix (cm⁴)	Iy (cm⁴)
WF 900 x 300 x 286	286	912	302	18	34	28	364	498000	15700
WF 900 x 300 x 243	243	900	200	16	28	28	309.8	411000	12600
WF 900 x 300 x 213	213	890	299	15	23	28	270.9	345000	10300
WF 800 x 300 x 241	241	808	302	16	30	28	307.6	339000	13800
WF 800 x 300 x 210	210	800	300	14	26	28	267.4	292000	11700
WF 800 x 300 x 191	191	792	300	14	22	28	243.4	254000	9930
WF 700 x 300 x 215	215	708	302	15	28	28	273.6	237000	12900
WF 700 x 300 x 185	185	700	300	13	24	28	235.5	201000	10800
WF 700 x 300 x 166	166	692	300	13	20	28	211.5	172000	9020
WF 600 x 300 x 175	175	594	302	14	23	28	222.4	137000	10600
WF 600 x 300 x 151	151	588	300	12	20	28	192.5	118000	9020
WF 600 x 300 x 137	137	582	300	12	17	28	174.5	103000	7670
WF 600 x 200 x 134	134	612	202	13	23	22	107.7	103000	3180
WF 600 x 200 x 120	120	606	201	12	20	22	152.5	90400	2720
WF 600 x 200 x 106	106	600	200	11	17	22	134.4	77600	2280
WF 600 x 200 x 94,6	94.6	596	199	10	15	22	120.5	68700	1980
WF 500 x 300 x 128	128	488	300	11	18	26	163.5	71000	8110
WF 500 x 300 x 114	114	482	300	11	15	26	145.5	60400	6760
WF 500 x 200 x 103	103	506	201	11	19	20	131.3	56500	2580
WF 500 x 200 x 89,6	89.6	500	200	10	16	20	114.2	47800	2140
WF 500 x 200 x 79,5	79.5	496	199	9	14	20	101.3	41900	1840
WF 450 x 300 x 124	124	440	300	11	18	24	157.4	56100	8110
WF 450 x 300 x 106	106	434	299	10	15	24	135	46800	6690
WF 450 x 200 x 76	76	450	200	9	14	18	96.76	33500	1870
WF 450 x 200 x 66,2	66.2	446	199	8	12	18	84.3	28700	1580
WF 400 x 400 x 605	605	498	432	45	70	22	770.1	298000	94400
WF 400 x 400 x 415	415	458	417	30	50	22	528.6	187000	60500
WF 400 x 400 x 283	283	428	407	20	35	22	360.7	119000	39400
WF 400 x 400 x 232	232	414	405	18	28	22	295.4	92800	31000
WF 400 x 400 x 200	200	406	403	16	24	22	254.9	78000	26200

WF 400 x 400 x 197	197	400	408	21	21	22	250.7	70900	23800
WF 400 x 400 x 172	172	400	400	13	21	22	218.7	66600	22400
WF 400 x 400 x 168	168	394	405	18	18	22	214.4	59700	20000
WF 400 x 400 x 147	147	394	398	11	18	22	186.8	56100	18900
WF 400 x 400 x 140	140	388	402	15	15	22	178.5	49000	16300
WF 400 x 300 x 107	107	390	300	10	16	22	136	38700	7210
WF 400 x 300 x 94,3	94.3	386	299	9	14	22	120.1	33700	6240
WF 400 x 200 x 66	66	400	200	8	13	16	84.1	23700	1740
WF 400 x 200 x 56,6	56.6	396	199	7	11	16	72.1	20000	1450
WF 350 x 350 x 159	159	356	352	14	22	20	202	47600	16000
WF 350 x 350 x 156	156	350	357	19	19	20	198.4	42800	14400
WF 350 x 350 x 137	137	350	350	13	19	20	173.9	40300	13600
WF 350 x 350 x 131	131	344	354	16	16	20	166.6	35300	11800
WF 350 x 350 x 115	115	344	348	10	16	20	146	33300	11200
WF 350 x 350 x 106	106	338	351	13	13	20	135.3	28200	9380
WF 350 x 250 x 79,7	79.7	340	250	9	14	20	101.5	21700	3650
WF 350 x 250 x 69,2	69.2	336	249	8	12	20	88.1	18500	3090
WF 350 x 175 x 49,6	49.6	350	175	7	11	14	63.1	13600	984
WF 350 x 175 x 41,4	41.4	346	174	6	9	14	52.6	11100	792
WF 300 x 300 x 106	106	304	301	11	17	18	134.8	23400	7730
WF 300 x 300 x 106	106	300	305	15	15	18	134.8	21500	7100
WF 300 x 300 x 94	94	300	300	10	15	18	119.8	20400	6750
WF 300 x 300 x 87	87	298	299	9	14	18	110.8	18800	6240
WF 300 x 300 x 84,5	84.5	294	302	12	12	18	107.7	16900	5520
WF 300 x 200 x 65,4	65.4	298	201	9	14	18	83.3	13300	1900
WF 300 x 200 x 56,8	56.8	294	200	8	12	18	72.4	11300	1600
WF 300 x 150 x 36,7	36.7	300	150	6.5	9	13	46.8	7210	508
WF 300 x 150 x 32	32	298	149	5.5	8	13	40.8	6320	442
WF 250 x 250 x 82,2	82.2	250	255	14	14	16	104.7	11500	3880
WF 250 x 250 x 72,4	72.4	250	250	9	14	16	92.2	10800	3650
WF 250 x 250 x 66,5	66.5	248	149	8	13	16	84.7	9930	3350
WF 250 x 250 x 64,4	64.4	244	252	11	11	16	82.1	8790	2940
WF 250 x 175 x 44,1	44.1	244	175	7	11	16	56.2	6120	984
WF 250 x 125 x 29,6	29.6	250	125	6	9	12	37.6	4050	294
WF 250 x 125 x 25,7	25.7	248	124	5	8	12	32.7	3540	255
WF 200 x 200 x 65,7	65.7	208	202	10	16	13	83.7	6530	2200
WF 200 x 200 x 56,2	56.2	200	204	12	12	13	71.5	4980	1700
WF 200 x 200 x 49,9	49.9	200	200	8	12	13	63.5	4720	1600
WF 200 x 150 x 30,6	30.6	194	150	6	9	13	39	2690	507
WF 200 x 100 x 21,3	21.3	200	100	5.5	8	11	27.16	1840	134
WF 200 x 100 x 18,2	18.2	198	99	4.5	7	11	23.18	1580	114
WF 175 x 175 x 40,2	40.2	175	175	7.5	11	12	51.21	2880	984
WF 175 x 125 x 23,3	23.3	169	125	5.5	8	12	29.65	1530	261
WF 175 x 90 x 18,1	18.1	175	90	5	8	9	23.04	1210	97.5

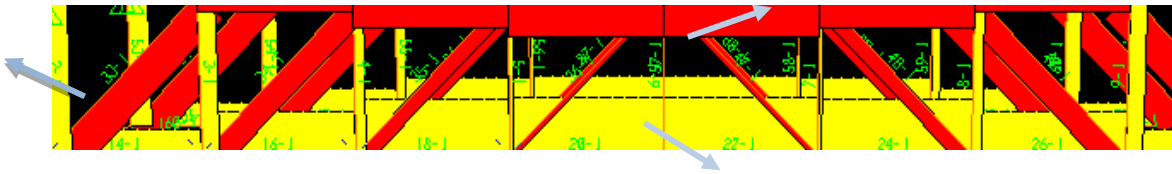
WF 150 x 150 x 31,5	31.5	150	150	7	10	11	40.14	1640	563
WF 150 x 100 x 21,1	21.1	148	100	6	9	11	26.84	1020	151
WF 150 x 75 x 14	14	150	75	5	7	8	17.85	666	49.5
WF 125 x 125 x 23,8	23.8	125	125	6.5	9	10	30.31	847	293
WF 125 x 60 x 13,2	13.2	125	60	6	8	9	16.84	413	29.2
WF 100 x 100 x 17,2	17.2	100	100	6	8	10	21.9	383	134
WF 100 x 50 x 9,3	9.3	100	50	5	7	8	11.85	187	14.8

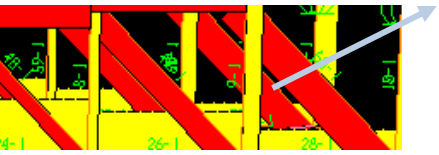
0.615

11	12	13	14
ix = rx (cm)	iy = ry (cm)	Sx (cm^3)	Sy (cm^3)
37	6.56	10900	1040
36.4	6.39	9140	843
35.7	6.16	7760	688
33.2	6.7	8400	915
33	6.62	7290	782
32.3	6.39	6410	662
29.4	6.86	6700	853
29.3	6.78	5760	722
28.6	6.53	4980	602
24.9	6.9	4620	701
24.8	6.85	4020	601
24.3	6.63	3530	511
24.6	4.31	3380	314
24.3	4.22	2980	271
24	4.12	2590	228
23.9	4.05	2310	199
20.8	7.04	2910	541
20.4	6.82	2500	451
20.7	4.43	2230	257
20.5	4.33	1910	214
20.3	4.27	1690	185
18.9	7.18	2550	541
18.6	7.04	2160	448
18.6	4.4	1490	187
18.5	4.33	1290	159
19.7	11.1	12000	4370
18.8	10.7	8170	2900
18.2	10.4	5570	1930
17.7	10.2	4480	1530
17.5	10.1	3840	1300

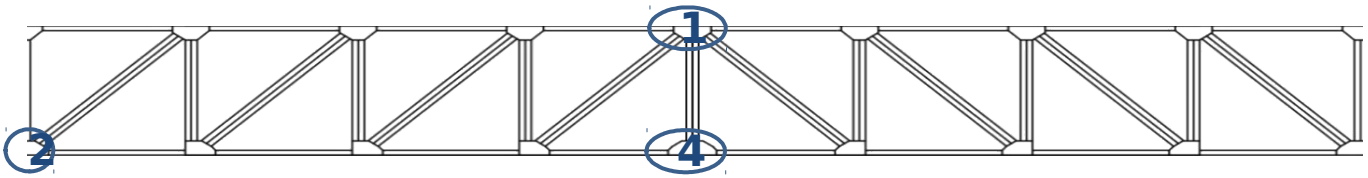
16.8	9.75	3540	1170
17.5	10.1	3330	1120
16.7	9.65	3030	985
17.3	10.1	2850	951
16.6	9.54	2520	809
16.9	7.28	1980	481
16.7	7.21	1740	418
16.8	4.54	1190	174
16.7	4.48	1010	145
15.3	8.9	2670	909
14.7	8.53	2450	809
15.2	8.84	2300	776
14.6	8.43	2050	669
15.1	8.78	1940	646
14.4	8.33	1670	534
14.6	6	1280	292
14.5	5.92	1100	248
14.7	3.95	775	112
14.5	3.88	641	91
13.2	7.57	1540	514
12.6	7.26	1440	466
13.1	7.51	1360	450
13	7.51	1270	417
12.5	7.16	1150	365
12.6	4.77	893	189
12.5	4.71	771	160
12.4	3.29	481	67.7
12.4	3.29	424	59.3
10.5	6.09	919	304
10.8	6.29	867	292
10.8	6.29	801	269
10.3	5.98	720	233
10.4	4.18	502	113
10.4	2.79	324	47
10.4	2.79	285	41.1
8.83	5.13	628	218
8.35	4.88	498	167
8.62	5.02	472	160
8.3	3.61	277	67.6
8.24	2.22	184	26.8
8.26	2.21	160	23
7.5	4.38	330	112
7.18	2.97	181	41.8
7.26	2.06	139	21.7

6.39	3.75	219	75.1
6.17	2.37	138	30.1
6.11	1.66	88.8	13.2
5.29	3.11	136	47
4.95	1.32	66.1	9.73
4.18	2.47	76.5	26.7
3.98	1.12	37.5	5.91





BAB 7. PERENCANAAN SAMBUNGAN



	BJ baut						BJ Material	
Baja	50	Fu	=	500	Mpa	Baja	52	Fu
		Fy	=	290	Mpa			Fy

7.1. Sambungan Batang Horizontal Tepi Atas

Data : > GayaBatang : S = 204849

> Profil : Direncanakan Profil WF :

bf = 400 mm A g

h = 400 mm

tw = 13 mm

tf = 21 mm

Rencana :

> Baut Æ = 2 cm Ab

> Pelat Penyambung ; t

a. Kekuatan Baut

> Kekuatan geser Baut

ØRn	=	0.75	r1	fu_{baut}	m	Ab
ØRn	=	0.75	0.5	5000	2	3.142
	=	11781	kg	*.....(Menentukan)*		

> Kekuatan tumpu Baut

ØRn	=	0.75	2.4	d	tp	$fu_{material}$
ØRn	=	0.75	2.4	2	2	5200
	=	37440	kg	*.....(Tidak Menentukan)*		

b. Jumlah Baut

Baut diletakkan pada kedua flens dari rangka batang,
maka jumlah baut untuk tiap sisi flens :

$$n = \frac{S}{2 \quad ÆRn}$$

$$= \frac{204849.0}{2 \quad 11780.9725}$$

= 8.69406 baut

= 10 baut

c. Jarak Pemasangan Baut

> Jarak Horizontal

->	1.5Ø	≤	s	≤	4tp + 100	atau	200 mm
	30	≤	50	≤	180	atau	200
	mm		mm		mm		mm

maka, s dipasang = 50 mm

->	3Ø	≤	s	≤	15tp	atau	200 mm
	60	≤	150	≤	300	atau	200
	mm		mm		mm		mm

maka, s dipasang = 150 mm

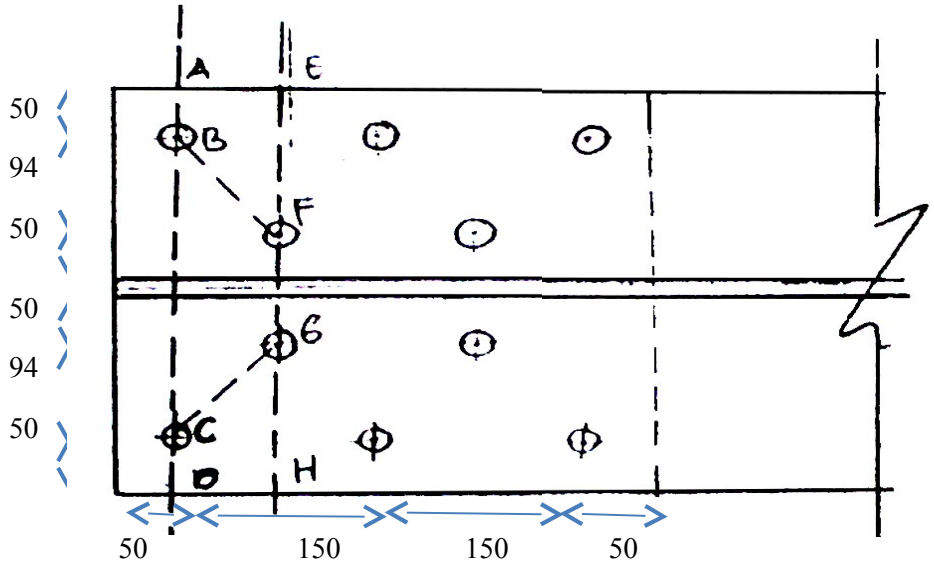
> Jarak Vertikal

->	1.5Ø	≤	s	≤	4tp + 100	atau	200 mm
	30	≤	50	≤	180	atau	200
	mm		mm		mm		mm

maka, s dipasang = 50 mm

->	3Ø	≤	s	≤	15tp	atau	200 mm
	60	≤	94	≤	300	atau	200
	mm		mm		mm		mm

maka, s dipasang = 94 mm



d. Cek Luasan Kritis (berlaku sesuai gambar: baut diletakkan selang seling)

ABCD Anetto	=	Ag - n Øp tf				
	=	218.7	-	9.66		
	=	209.04	cm²			
ABFGH Anetto	=	Ag - n Øp tf + n (S / 4ɥ) tf				
	=	218.7	-	0	+	3.14
	=	221.84	cm²			
ABFGHCD Anetto	=	Ag - n Øp tf + n (S / 4ɥ) tf				
	=	218.7	-	0	+	0.60
	=	219.30	cm²			
Anetto kritis	=	209.04	cm²			
Anetto kritis	≥	0.85 Ag				
209.04 cm²	>	185.895 cm²	...(OK)			

7.2. Sambungan Batang Vertikal

Data : > Gaya Batang : S = 125465

> Profil : Direncanakan Profil WF :

bf = 300 mm A g

h = 300 mm

tw = 10 mm

tf = 15 mm

Rencana :

> Baut Æ = 1.6 cm Ab

> Pelat Penyambung ; t

a. Kekuatan Baut

> Kekuatan geser Baut

ØRn	=	0.75	r1	fu_{baut}	m	Ab
ØRn	=	0.75	0.5	5000	2	2.011
	=	7539.8	kg	*.....(Menentukan)*		

> Kekuatan tumpu Baut

ØRn	=	0.75	2.4	d	tp	$fu_{material}$
ØRn	=	0.75	2.4	1.6	2	5200
	=	29952	kg	*.....(Tidak Menentukan)*		

b. Jumlah Baut

Baut diletakkan pada kedua flens dari rangka batang, maka jumlah baut untuk tiap sisi flens

$$n = \frac{S}{2 \quad ÆRn}$$

$$= \frac{125465.0}{2 \quad 7539.82237}$$

= 8.32016 baut

= 10 baut

c. Jarak Pemasangan Baut

> Jarak Horizontal

->	1.5Ø	≤	s	≤	4tp + 100	atau	200 mm
	24 mm	≤	50 mm	≤	180 mm	atau	200 mm
	maka, s dipasang =		50	mm			
->	3Ø	≤	s	≤	15tp	atau	200 mm
	48 mm	≤	80 mm	≤	300 mm	atau	200 mm
	maka, s dipasang =		80	mm			

> Jarak Vertikal

->	1.5Ø	≤	s	≤	4tp + 100	atau	200 mm
	24 mm	≤	70 mm	≤	180 mm	atau	200 mm

maka, s dipasang = 70 mm

-> 3Ø ≤ s ≤ 15tp atau 200 mm

48 ≤ 155 ≤ 300 atau 200

mm mm mm mm

maka, s dipasang = 155 mm

d. Cek Luasan Kritis

ABCD Anetto = Ag - n Øp tf

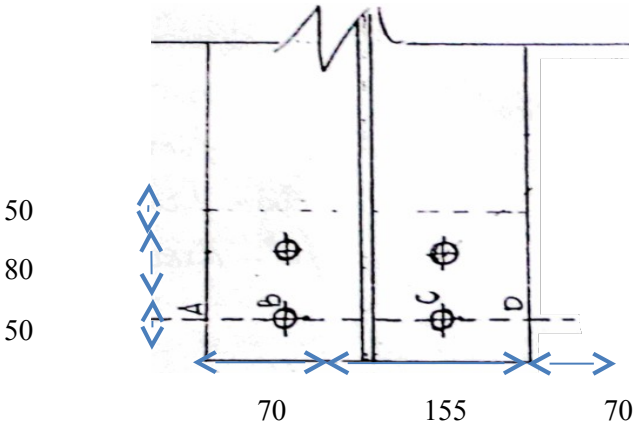
= 119.8 - 5.7

= 114.1 cm²

Anetto kritis ≥ 0.85 Ag

114.10 > 101.83 ...(OK)

cm² cm²



7.3. Sambungan Batang Diagonal

Data : > Gaya Batang : S = 177234

> Profil : Direncanakan Profil WF :

bf = 300 mm A g

h = 390 mm

tw = 10 mm

tf = 16 mm

Rencana :

> Baut Æ = 1.6 cm Ab

> Pelat Penyambung ; t

a. Kekuatan Baut

> Kekuatan geser Baut

ØRn = 0.75 r1 fu_{baut} m Ab

ØRn = 0.75 0.5 5000 2 2.011

= 7539.8 kg *.....(Menentukan)*

> Kekuatan tumpu Baut

ØRn = 0.75 2.4 d tp $fu_{material}$

ØRn = 0.75 2.4 1.6 2 5200

= 29952 kg *.....(Tidak Menentukan)*

b. Jumlah Baut

Baut diletakkan pada kedua flens dari rangka batang,

maka jumlah baut untuk tiap sisi flens :

$$n = \frac{S}{2 \text{ ÆRn}} = \frac{177234.0}{2 \quad 7539.82237} = 11.7532 \text{ baut} = 12 \text{ baut}$$

c. Jarak Pemasangan Baut

> Jarak Horizontal

->	1.5Ø	≤	s	≤	4tp + 100	atau	200 mm
	24 mm	≤	50 mm	≤	180 mm	atau	200 mm
	maka, s dipasang =		50	mm			
->	3Ø	≤	s	≤	15tp	atau	200 mm
	48 mm	≤	80 mm	≤	300 mm	atau	200 mm
	maka, s dipasang =		80	mm			

> Jarak Vertikal

->	1.5Ø	≤	s	≤	4tp + 100	atau	200 mm
	24 mm	≤	70 mm	≤	180 mm	atau	200 mm
	maka, s dipasang =		70	mm			
->	3Ø	≤	s	≤	15tp	atau	200 mm
	48 mm	≤	155 mm	≤	300 mm	atau	200 mm
	maka, s dipasang =		155	mm			

d. Cek Luasan Kritis

ABCD Anetto	=	Ag - n Øp tf				
	=	136	-	2	*	1.9
	=	129.92	cm²			
Anetto kritis	≥	0.85 Ag				
129.92 cm²	>	115.6 cm²	...(OK)			

7.4. Sambungan Batang Horizontal Bawah Tengah

Data : > GayaBatang : S = 283953

> Profil : Direncanakan Profil WF :

bf = 400 mm A g

h = 400 mm

tw = 13 mm

tf = 21 mm

Rencana :

> Baut Æ = 2 cm Ab

> Pelat Penyambung t

a. Kekuatan Baut

> Kekuatan geser Baut

ØRn	=	0.75	r1	fu_{baut}	m	Ab
ØRn	=	0.75	0.5	5000	2	3.142
	=	11781.0	kg	*.....(Menentukan)*		

> Kekuatan tumpu Baut

ØRn	=	0.75	2.4	d	tp	$fu_{material}$
ØRn	=	0.75	2.4	2	2	5200
	=	37440	kg	*.....(Tidak Menentukan)*		

b. Jumlah Baut

Baut diletakkan pada kedua flens dari rangka batang, maka jumlah baut untuk tiap sisi flens

$$n = \frac{S}{2 \quad ÆRn}$$

$$= \frac{283953.0}{2 \quad 11780.9725}$$

= 12.0513 baut

= 14 baut

c. Jarak Pemasangan Baut

> Jarak Horizontal

->	1.5Ø	≤	s	≤	4tp + 100	atau	200 mm
	30 mm	≤	50 mm	≤	180 mm	atau	200 mm
	maka, s dipasang =		50	mm			
->	3Ø	≤	s	≤	15tp	atau	200 mm
	60 mm	≤	150 mm	≤	300 mm	atau	200 mm
	maka, s dipasang =		150	mm			

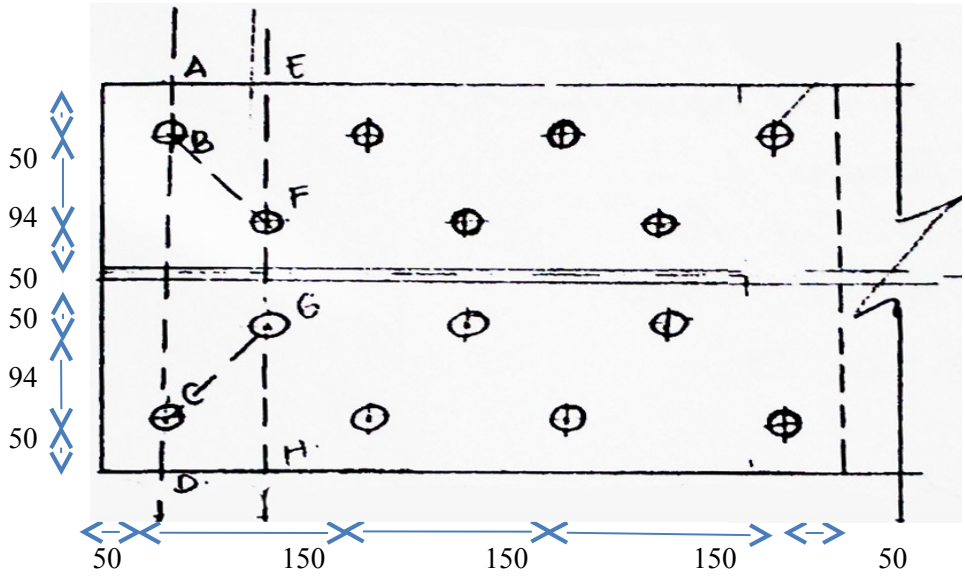
> Jarak Vertikal

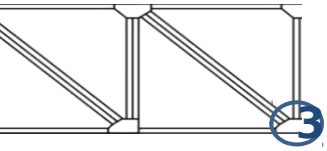
->	1.5Ø	≤	s	≤	4tp + 100	atau	200 mm
	30 mm	≤	50 mm	≤	180 mm	atau	200 mm
	maka, s dipasang =		50	mm			
->	3Ø	≤	s	≤	15tp	atau	200 mm
	60 mm	≤	94 mm	≤	300 mm	atau	200 mm
	maka, s dipasang =		94	mm			

d. Cek Luasan Kritis

ABCD Anetto	=	Ag - n Øp tf				
	=	218.7	-	9.66		
	=	209.04	cm²			
ABFGH Anetto	=	Ag - n Øp tf + n (S / 4ų) tf				
	=	218.7	-	0	+	3.14
	=	221.84	cm²			
ABFGHCD Anetto	=	Ag - n Øp tf + n (S / 4ų) tf				
	=	218.7	-	0	+	0.60

	=	219.30	cm²
Anetto kritis	=	209.04	cm²
Anetto kritis	≥	0.85 Ag	
209.04	>	185.895	...(OK)
cm²		cm²	





= 520 Mpa
= 360 Mpa

kg
WF 400x400x21x13
= 218.7 cm^2

= 3.14159 cm^2
= 2 cm

kg
WF 300x300x15x10

= 119.8 cm^2

= 2.01062 cm^2

= 2 cm

:

kg
WF 390x300x16x10
= 136

= 2.01062
= 2

* 1.6

kg
WF 400x400x21x13
= 218.7 cm^2

= 3.14159 cm^2
= 2 cm

:
:

Data Tanah :

Kedalaman	N-SPT
16	27
17	31
18	33
19	37
20	39
21	40
22.5	40
23	38
24	30
25	20

Diameter T.Pancang = 60 cm

Untuk Tanah Lempung :

1 D = 60 cm

1 D = 60 cm

Ujung Tiang ada di kedalaman 20 m

$Qp=400.n.Ap$ = 961.33 Kn

$Qf=2.N.As$ = 2601.24 Kn

$Qu=Qp+Qf$ = 3562.57

N umtuk Qf 34.50

$Qult=Qu/Sf=$ 890.64152 kn

P = 4085.21

$N=P/Q=$ 4.58681739 = 5 Buah pada setiap pilar

$Ap = \pi/4 \times [\![Diameter\ T.Pancang]\!]$ ^2= 0.283 m2

$As = \pi.D.kedalaman\ T.Pancang$= 37.70 m2

maka	nilai N pada kedalaman 1+D	20.6	=
	nilai N pada kedalaman 1-D	19.4	=
	N Rata-rata		=

10
7
8.5

Penulangan Pilecap

> Data perencanaan :

f'c =	35	MPa				
fy =	410	MPa				
V =	5 x P					
=	5	700.77	=	3503.85	ton	
Mu =	V x 1,5 m					
=	3503.85	1.5	=	5255.775	ton.m	
			=	52557750000	Nmm	
Tebal Pelat =	1	m				
D. Tul. Utama =	32	mm				
D. Tul. Mem =	32	mm				
Selimut Beton =	50	mm				
Dimensi Pilecap =	b =	8000	mm			
	h =	5000	mm			

> Perhitungan :

d = t - selimut beton - 0,5 tul. Utama - tul. Memanjang

= 902 mm

$$\rho \text{ balance} = \frac{0.85}{fy} f'c \; \beta 1 \times$$

$$= \frac{0.85}{410} 35 \; 0.85 \times$$

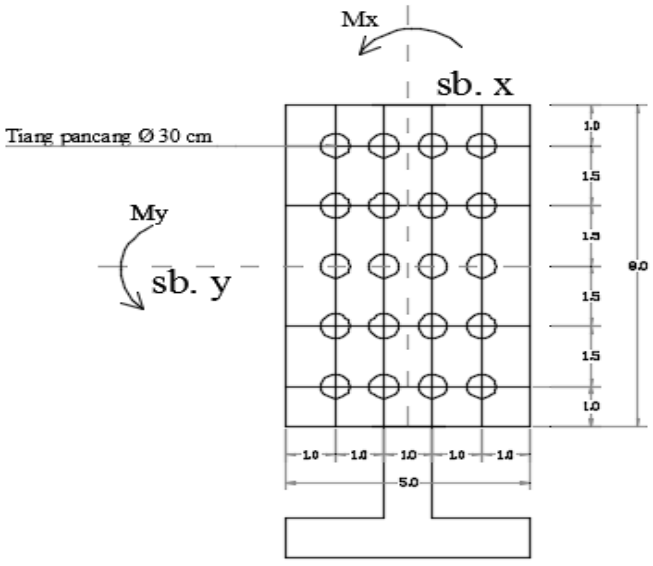
= 0.0366

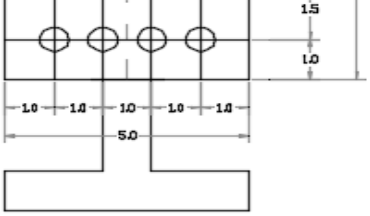
ρ max = 0.75 x ρ balance

= 0.75 x 0.0366

= 0.0275

$$\rho \text{ min} = \frac{1.4}{fy} = \frac{1.4}{410} =$$





>> Koefisien Ketahanan

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

$$= \frac{52557750000}{0.85 \quad 8000 \quad 813604}$$

$$= 9.500 \text{ N/mm2}$$

$$m = \frac{fy}{0{,}85\ f'c} = \frac{410}{29.75} =$$

$$\rho \text{ perlu} = \frac{1}{m}\left(1-\sqrt{1-\frac{2\,m\,Rn}{fy}}\right)$$

$$= \frac{1}{13.782} \qquad 1 \quad - \quad \sqrt{\qquad\qquad 1}$$

$$= 0.029$$

Syarat :	ρ min	<	ρ perlu	<	ρ max
	0.003	<	0.029	>	0.0275

Dipakai : ρ min = 0.003

Luas Tulangan

As perlu = ρ x b x
= 0.003 x 8000 x
= 24640 mm2

Digunakan Tulangan 36 D 32 As =

Untuk Tulangan memanjang

As perlu = ρ x b x
= 0.003 x 5000 x
= 15400 mm2

Digunakan Tulangan D 32 -200 As =

>> Kontrol Geser Poer

Gaya geser yang terjadi

Vu = 3503.85 ton
= 35038.5 kN

Kekuatan Beton

ϕ Vc = 0.6 x 1/6√(f^′c) bw
= 0.6 x 0.986013297 8000
= 4269043.17 N
= 4269 kN

Syarat :	Vu	<	ϕ Vc	
	35038.5	>	4269	**perlu tulangan geser**

Pasang Tulangan geser praktis D 18 - 400 mm

$$\frac{600}{600 + fy}$$

$$\frac{600}{1010}$$

0.003

13.782

$$-\frac{261.8434}{410}$$

d
902

28938.24

d
902

803.84

d
902

PENULANGAN DINDING ABUTMENT

Untuk perencanaan dinding abutment direncanakan berdasarakan momen maksimum.

Mx max = 790.21 tm
= 7902050000 Nmm

maka akan direncanakan tulangan abutmen :

Tebal dinding abutment	=	100 cm
Diameter tul. Utama	=	32 mm
Diameter tul. Memanjang	=	32 mm
Selimut beton	=	50 mm

dx = t - selimut beton - 0,5φutama - φmemanjang
= 1000 - 50 - 16 - 32
= 902 mm

$$\rho_{balance} = \frac{0.85 \times fc' \times \beta_1}{fy} \times \frac{600}{(600 + fy)}$$

$$= \frac{0.85 \times 35 \times 0.81}{410} \times \frac{600}{(600 + 410)}$$

= 0.0349

$$\rho_{max} = 0.75 \times \rho_{balance}$$
$$= 0.75 \times 0.035$$
= 0.0262

$$\rho_{min} = \frac{1.4}{fy}$$
$$= \frac{1.4}{410}$$
= 0.0034146341

a. Koefisien Ketahanan

$$Rn = \frac{Mu}{0.85 \times b \times d^2}$$
$$= \frac{Mu}{0.85 \times b \times d^2}$$
$$= \frac{7902050000}{0.85 \times 8000 \times 813604}$$
= 1.4283 N/mm²

$$m = \frac{fy}{0.85\, fc'}$$
$$= \frac{410}{}$$

$$= \frac{0.85 \quad 35}{13.782}$$

$$\rho perlu = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \; m \; Rn}{fy}}\right)$$

$$= \frac{1}{13.782}\left(1 - \sqrt{1 - \frac{2 \; 13.78 \; 1.428}{410}}\right)$$

$$= 0.003571543$$

Syarat :

$\rho min < \rho perlu < \rho max$

$0.0034 < 0.0036 < 0.0262$

Dipakai $\rho perlu = 0.0036$

b. Luas Tulangan

As perlu $= \rho \times b \times d$

$= 0.0036 \times 8000 \times 902$

$= 25772 \; mm^2$

Digunakan tulangan 36 D 32 ($As = 28953 \; mm^2$)

Digunakan tulangan D 32 - 220

Untuk tulangan memanjang :

As perlu $= \rho \times b \times d$

$= 0.0036 \times 1000 \times 902$

$= 3222 \; mm^2$

Digunakan tulangan 5 D 32 ($As = 4021 \; mm^2$)

Digunakan tulangan D 32 - 180

]



BAB 8. PERENCANAAN ELASTOMER

8.1. Perletakan Jembatan Rangka Baja

(Berdasarkan Bridge Design Code section 2).

Rencana Keadaan batas kelayanan beban hidup load faktor

K = 1.2 untuk angin

K = 1 untuk beban mati dan beban gempa

8.2. Pembebanan

Beban Mati

- Aspal	=	d4	Laspal	Baspal	γaspal	
	=	0.05	45	9.6	22	
	=	475.2	kN	=	47520	
- Lantai Kendaraan	=	d3	Lbeton	Bbeton	γbeton	
	=	0.2	45	9.6	25	
	=	2160	kN	=	216000	
- Balok Memanjang	=	berat profil x		Pbalok	x	n
	=	1.59	45	9		
	=	643.95	kN	=	64395	
- Balok Melintang	=	berat profil x		Bbalok	x	n
	=	2.15	7.8	5		
	=	83.85	kN	=	8385	
- Batang Horizontal	=	berat profil x		Pbatang	x	n
	=	0.94	5.5	32		
	=	165.968	kg			
- Batang Vertikal	=	berat profil x		Pbatang	x	n
	=	6.05	5.5	18		
	=	598.95	kg			
- Batang Diagonal	=	berat profil x		Pbatang	x	n
	=	1.97	7.78	8		
	=	122.6128	kg			
- Beban Ikatan Angin	=	W	Lk	0.25	Σλ	
	=	0.441	7.5	0.25	10	
	=	8	kg			
Beban total	=	**337195.80**	**kg**			
- Sambungan & pelat simpul	=	Øbatang	x	4%	x	

= 0.25 0.04 337195.8
= 3371.96 kg

Jadi Beban mati total: = **340567.76** **kg** →

Beban Hidup

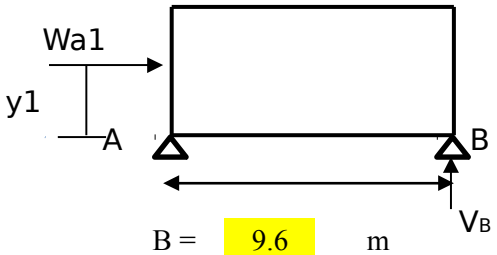
- Beban Hidup (UDL+KEL)

WUDL = q x L x B = 9 45 5.5
= 222750 kg (4 perletakkan)

WKEL = p x L = 44 45
= 198000 kg (2 perletakkan)

Beban Angin

- Beban Angin



B = 9.6 m

$$VB = \frac{Wa1 \cdot y1}{B} = \frac{7.50 \quad 2.25}{9.6} = 1.7578 \text{ kN} = 175.78 \text{ kg}$$

Beban Gempa

- ***Koefisien geser dasar*** **C**

Dimensi Pilar taksiran : 1 x 8 x 8.5

W_{TP} = Wtot + 0.5 Wpilar
= 2 Wd + 0.5 Wpilar
= 2 340567.8 + 0.5 [2500 1.00 8 8.5]
= 681135.515091 + 85000.0 = 766135.5 kg

E = 4700 $\sqrt{fc'}$ = 4700 $\sqrt{35}$ = 27805.574980568 Kg/cm^2

I1 = 60% 1/12 b h 3
= 60% 0.08 8 1 3 = 0.4 m^4

I2 = 60% 1/13 h b 3
= 60% 0.08 1 8 3 = 25.6 m^4

a. Arah Memanjang ***(Berdasarkan BMS 2.4.7.1 (2.10) - on page 2-46***

$$Kp = \frac{3 \quad E \quad I1}{L \quad 3} = \frac{3 \quad E \quad 0.4}{8.5 \quad 3} =$$

$$Tmj = 2\pi \sqrt{\frac{WTP}{g.Kp}} = 2\pi \sqrt{\frac{766135.5}{9.81 \quad Kp}}$$

dimana :

T = waktu getar arah memanjang (dalam detik)
g = percepatan graviatsi (m/dt^2)
W_{TP} = berat total nominal bangunan atas termasuk beban mati ditambah 1/2 berat pilar (kN)
Kp = kekakuan gabungan sebagai gaya horisontal yang diperlukan untuk menimbulkan satu-satuan lendutan pada bagian atas

Untuk arah memanjang : Tmj = 0.9
Zone gempa = 2
Dari gambar 2.14 - on page 2-47, didapatkan C = 0.

b. Arah Melintang (Berdasarkan BMS 2.4.7.1 (2.10) - on page 2-46

$$Kp = \frac{3 \quad E \quad I2}{L \quad 3} = \frac{3 \quad E \quad 25.6}{8.5 \quad 3} =$$

$$Tmj = 2\pi \sqrt{\frac{WTP}{g.Kp}} = 2\pi \sqrt{\frac{766135.5}{9.81 \quad Kp}}$$

dimana :
T = waktu getar arah memanjang (dalam detik)
g = percepatan graviatsi (m/dt^2)
W_{TP} = berat total nominal bangunan atas termasuk beban mati ditambah 1/2 berat pilar (kN)
Kp = kekakuan gabungan sebagai gaya horisontal yang diperlukan untuk menimbulkan satu-satuan lendutan pada bagian atas

Untuk arah memanjang : Tmj = 0.0
Zone gempa = 2
Dari gambar 2.14 - on page 2-47, didapatkan C = 0.

- ***Faktor type bangunan*** **S**

(Berdasarkan BMS pasal 2.4.7.3 tabel 2.14 - on page 2-51)
Digunakan tipe A.4 yaitu : jembatan dengan daerah sendi beton/baja.

S = 1.0 F
F = 1.25 s/d 0.025 n

dengan n = jumlah sendi plastis yang manahan deformasi arah lateral pada masing-masing bagian yang monolit dari jembatan (misal: bagian yang dipisahkan untuk expantion joint yang memberikan keleluasaan bergerak)

S = 0.25 4 = 1.0

- ***Faktor kepentingan*** **I**

(Berdasarkan BMS pasal 2.4.7.3 tabel 2.13 - on page 2-50)
Digunakan I = 1.2

yaitu : jembatan memuat lebih dari 2000 kendara
utama atau arteri, dan jembatan dimana tidak ada ru

- ***Perhitungan Gaya Geser Total***

  (Berdasarkan BMS pasal 2.4.7.1 - on page 2-45)

  T_{EQ} = Kh I Wt dimana : Kh

  T_{EQ} = C S I 2 Wd

  dengan :

C	=	Koefisien dasar geser gempa
S	=	faktor type bangunan
I	=	faktor keutamaan
Wt	=	berat total nominal bangunan
T_{EQ}	=	gaya geser dasar untuk arah yang ditinjau

- ***Gaya Geser Total arah Memanjang*** ***dibagik***

  T_{EQ} = C S I 4 Wd 0.25

  = 0.2 1.0 1.2 4 340567.76 0.25

  = 81736.26 kg

  $F_{(x)Q}$ = 0.5 T_{EQ}

  = 0.5 81736.26 = 40868.13 kg

- ***Gaya Geser Total arah Melintang*** ***dibagik***

T_{EQ} = C S I 4 Wd 0.25

= 0.21 1.0 1.2 4 340567.76 0.25

= 85823.07 kg

$F_{(y)Q}$ = T_{EQ} = 85823.07 kg

- ***Kombinasi Beban Vertikal dan Horisontal***

1. Kombinasi beban Vertikal, yaitu :

Vmax = 1 PDL + 2 PLL + 1.2 Wangin

= 1 340567.8 + 2 420750.0 +

= **1182278.70** kg

Vmax = 1/4 Wd + 1/4 WUDL + 1/2 WKEL

= 85141.94 + 55687.5 + 99000

= **239917.33** kg

2. Kombinasi beban horizontal terbesar arah memanjang yaitu:

Ha = Pgempa longitudinal

= $F_{(x)Q}$ = 40868.13 kg

3. Kombinasi beban horizontal terbesar arah melintang yaitu:

Hb = Pgempa transversal

= $F_{(y)Q}$ = 85823.07 kg

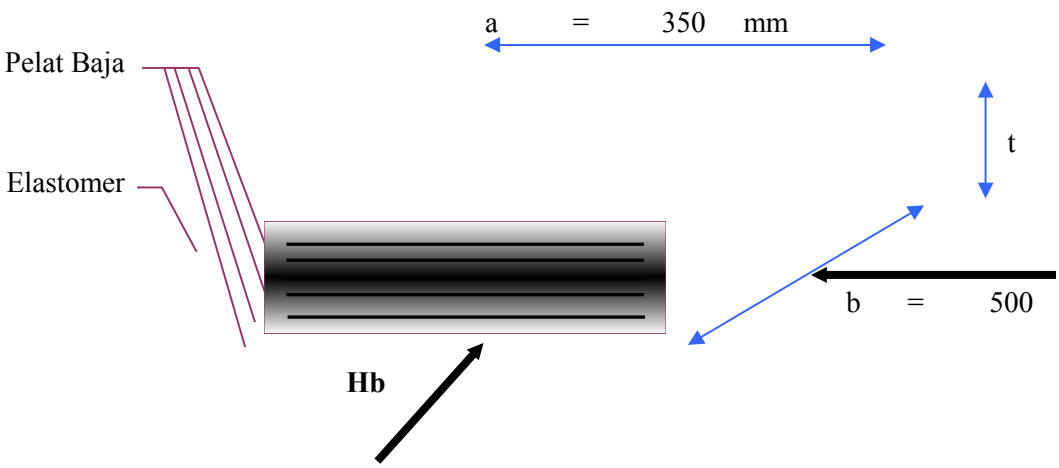
→ Hb = 85823.1 kg > 6250 kg →

Jadi, dipakai Hb = 85823.1 kg

8.3. PERENCANAAN DIMENSI ELASTOMER

8.3.1. Perencanaan Bahan ElasTomer

- Durometer hardness IRHD 70
- Shear modulus, G = 1.2 Mpa
- Bulk Modulus, B = 2000 Mpa
- Panjang Perletakan, a = 350 mm
- Lebar Perletakan, b = 500 mm
- Tebal selimut, tc = 12 mm
- Tebal Lapis Dalam, t1 = 12 mm
- Tebal Pelat Baja ts = 3 mm n
- Tebal total elastomer, t = 87 mm
- Side Cover thickness, tsc = 10 mm (berdasarkan BM
- Luas denah total karet, Ar = 158400 mm^2 { (a - 2tsc)* (b -


a = 350 mm
Pelat Baja
Elastomer
t
b = 500
Hb

8.3.2. Kontrol Elastomer

Faktor Bentuk *(Berdasarkan BMS pasal 8.3.5)*

$$S = \frac{Ar}{P \quad te}$$

dimana :

Ar = Luas permukaan terikat

P = Keliling Permukaan terikat

te = tebal efektif lapisan elastomer

= t1 = 12 mm .....untu

= 1.4 tc .....untu

= 1.4 12 = 16.8 mm

Perletakan Laminasi, $4 < S < 12$

Maka,

$$S = \frac{158400}{2 \left[330 + 480 \right] 12}$$

$$= 8.15 \quad \textbf{.....Ok!}$$

Persyaratan Perencanaan *(Berdasarkan BMS pasal 8.3.6)*

- **Regangan Geser tekan (εsc)**

$$\delta a = \frac{Ha \quad t}{Ar \quad G}$$

$$\delta b = \frac{Hb \quad t}{Ar \quad G}$$

dimana : δa = δb =

simpangan geser max. tangensial pada j dimensi a dan b akibat gerakan struktur dan

Ar = seluruh luas daerah untuk lapis tak terikat

G = modulus geser

t = tebal total elastomer

Ha = P gempa longitudinal

Hb = P gempa transversal

$$\delta a = \frac{40868.13 \times 87}{158400 \times 1.2} = 18.7054 \text{ mm}$$

$$\delta b = \frac{85823.1 \times 87}{158400 \times 1.2} = 39.2814 \text{ mm}$$

Aeff = Luas daerah efektif perletakan *(Berdasarkan BMS pasal 8*

$$= Ar \left(1 - \frac{\delta a}{a} - \frac{\delta b}{b} \right)$$

$$= 158400 \left(1 - \frac{18.705}{350} - \frac{39.28139}{500} \right) = 1374$$

$$\varepsilon c = \frac{Vmax}{3 \quad Aeff \quad G \left[1 + 2 \quad S \quad 2 \right]}$$

$$= \frac{1182278.70}{3 \quad 1.E+05 \quad 1.2 \left[1 + 2 \quad 8.15 \quad 2 \right]} =$$

Maka, $\varepsilon sc = G \; S \; ec = 1.2 \quad 8.15 \quad 0.0179$

$= \mathbf{0.17}$

- **Regangan Geser torsi (εsr)**

Gaya vertikal Vmax bekerja pada pusat luasan Elastomer dan momen =

maka $\alpha a = \alpha b = 0$

$\varepsilon sr = 0$ *(Berdasarkan BMS pasal 8.3.6.3)*

- **Regangan Geser tangensial (εsh)**

$$\varepsilon sh = \frac{\delta a}{t} = \frac{18.7}{87} = 0.22$$

* Untuk membatasi distorsi tangensial dan agar ujung perletakan menggelinc kecenderungan pelat baja untuk melentur, syarat yang harus dipenuhi adalah pasal maksimum ijin :

→ $Aeff \geq 0.9 \; Ar$

$137490 \geq 0.9 \quad 158400$

$137490 < 142560$ ***Nok! Cari ukuran elaston***

dan $\varepsilon sh \leq 0.7$

$0.22 < 0.7$ ***OK!***

* Syarat untuk menjamin bahwa regangan geser total yang berkembang tidak berle adalah:

→ $\varepsilon sh + \varepsilon sr + \varepsilon sc \leq \frac{2.4}{\sqrt{G}} = \frac{2.4}{\sqrt{1.2}}$

→ $0.22 + 0 + 0.17 \leq 2.19089$

$0.39 < 2.19089$ ***..OK!***

- **Persyaratan Tegangan Tekan rata-rata**

(Berdasarkan BMS pasal 8.3.6.2)

$\frac{Vmax}{Ar} \leq 15$ Mpa ⟶ Perletakan Laminasi

$\frac{239917.33}{158400} = 1.51463 \leq 15$ Mpa ***..OK!***

- **Persyaratan Stabilitas Perletakan**

(Berdasarkan BMS pasal 8.3.6.5)

$$\frac{Vmax}{Aeff} \leq \frac{2\ b\ G\ S}{3\ t} = \frac{2\ 500\ 1.2}{3\ 87}$$

$$\frac{239917.33}{137490.1} = 1.74498 \leq 37.463 \text{ Mpa} \quad \textit{..OK!}$$

- **Persyaratan Tebal Minimum Pelat Baja**

(Berdasarkan BMS pasal 8.3.6.6)

Tebal baja ts = 3 mm

dengan BJ 52 fy = 4100 kg/cm2

Syarat 1 :

ts ≥ te = 3

3 ≥ 3 Jadi yang menentukan adalah ts = 3

$$ts \geq \frac{3\ Vmax\ t1}{Ar\ fy} = \frac{3\ 239917.33}{158400\ 41}$$

→ 3 ≥ 0.133 ***..OK!***

Jadi Elastomer dipakai → 350 x 500 mm²

K = 2

kg

kg

kg

kg

beban total

Wd

= 2227.5 kN

= 1980 kN

= 2.78E+08 Kg/m^2

543320.82193 Kg/m

= 0.903

(kN/m)

)03
(tanah lunak)
.2 *liat BMS*

34772532.603 Kg/m

= 0.014

(kN/m)

)14
(tanah lunak)
21

an/hari, jembatan pada jalan raya
ıte alternatif.

= C S

an pada 4 elastomer

an pada 4 elastomer

1

1.2 175.8

\+ 1/2 VB
\+ 87.891

gaya rem pada 1 elastomer (diperkirakan)

= 5 lapis

MS tabel K.8)
2tsc) }

= 87 mm

Ha

mm

uk tebal lapis dalam
uk lapis selimut

permukaan tumpuan dalam arah
gaya tangensial.

3.3.6.1.d)

90.11 mm^2

0.018

0

ding seminimum mungkin atas
(8.3.6.3) : nilai regangan geser

ıer yang lebih besar

bihan berdasarkan pasal 8.3.6.1

8.15

mm

$$\frac{12}{10} = 0.133$$