

## **BAB V**

### **KESIMPULAN**

#### **5.1 Kesimpulan**

Dari hasil analisis dan perhitungan serta juga membandingkan dengan Proyek yang sedang berjalan saat ini yaitu pada Proyek Jalan Tol Porong – Gempol Seksi 1 dan dari uraian serta pembahasan yang telah disajikan pada bab-bab sebelumnya, maka dapat ditarik kesimpulan, yaitu sebagai berikut :

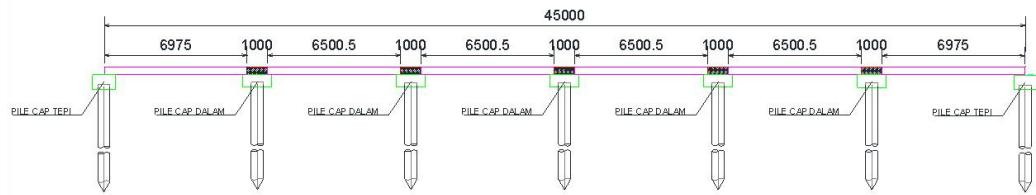
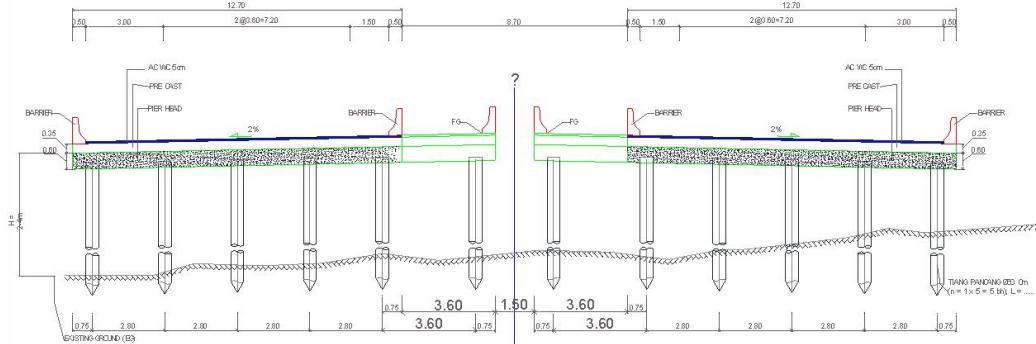
1. Biaya Konstruksi untuk *Piled Slab* adalah sebesar Rp. 9,800,079,791,- ( Sembilan Miliar Delapan Ratus Juta Tuju Puluh Sembilan Ribu Tuju Ratus Sembilan Puluh Satu Rupiah ). Dan untuk Cakar Ayam Modifikasi adalah sebesar Rp. 8,962,917,075,- ( Delapan Miliar Sembilan Ratus Enam Puluh Dua Juta Sembilan Ratus Tuju Belas Ribu Tuju Puluh Lima Rupiah ).
2. Waktu pelaksanaan Konstruksi yang dibutuhkan untuk pekerjaan *piled slab* pada Sta. 36+900 s/d 37+000 adalah selama 14 minggu hari kalender sedangkan Cakar Ayam Modifikasi membutuhkan waktu pelaksanaan selama 18 minggu hari kalender.
3. Sehingga dari uraian point 1 dan 2 diatas bahwa *Piled Slab* mempunyai biaya konstruksi lebih mahal daripada Cakar Ayam Modifikasi akan tetapi waktu pelaksanaan lebih cepat dan juga *Piled Slab* merupakan struktur yang mempunyai penurunan konsolidasi terkecil dibandingkan dengan Cakar Ayam Modifikasi, sehingga biaya *Operation Maintenance (OM)* dapat ditekan sekecil mungkin.

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# LAMPIRAN

**PILED SLAB PORONG CARAT SEKSI 1 (H 3 M)**  
**PILED SLAB 6 BENTANG**  
**Bentang antar Slab 7.5 M REVISI PELEBARAN**



### 1.1. PEMBEBANAN BALOK SLAB :

#### PERHITUNGAN BERAT STRUKTUR ATAS

- Berat beton	: $W_c$	=	2.50	t/m'
- Berat aspal	: $W_{as}$	=	2.20	t/m'
- Berat air hujan	: $W_w$	=	1.00	t/m'
- Tebal aspal	: $t_{aki}$	=	0.06	m
- Tebal air hujan	: $t_{wki}$	=	0.03	m

#### A. Beban Mati

1. Berat Sendiri Slab : By program

2. Berat Aspal + Air

- Berat Aspal	: $t_{aki} \times W_{as}$	=	0.13200	t/m <sup>2</sup>
- Berat Air	: $t_{wki} \times W_w$	=	0.03000	t/m <sup>2</sup>
Berat total		=	0.16200	t/m <sup>2</sup>

3. Berat tambahan

- Berat barrier	=	0.776	t/m
		0.776	t/m

#### B. Beban Hidup

Beban merata "D" :  $Q_l$  = 0.80 t/m

Beban Garis "D" :  $P_l$  = 4.40 ton

Beban terpusat "T" : = 12.5 ton

Faktor Dinamis = 0.40

Koefisien Kejut = 1.40

$$\text{Beban Merata "D"} = Q1 \cdot 1 = 0.8 \text{ Ton}$$

$$\text{Beban "KEL" "P"} = P \times K = 6.2 \text{ Ton}$$

$$\text{Beban hidup (LL)} = 7.0 \text{ Ton}$$

Beban Hidup Slab bentang 1:

$$\text{Luas Slab} = 5.625 \text{ m}^2$$

$$Qs1 = 1.237 \text{ Ton/m}^2 \quad 1237.33333 \text{ Kg/m}^2$$

Beban Hidup Slab bentang 2:

$$\text{Luas Slab} = 21.00 \text{ m}^2$$

$$Qs2 = 0.331 \text{ Ton/m}^2 \quad 331.428571 \text{ Kg/m}^2$$

Beban Hidup Slab bentang 3:

$$\text{Luas Slab} = 21.00 \text{ m}^2$$

$$Qs3 = 0.331 \text{ Ton/m}^2 \quad 331.428571 \text{ Kg/m}^2$$

Beban Hidup Slab bentang 4:

$$\text{Luas Slab} = 21.00 \text{ m}^2$$

$$Qs4 = 0.331 \text{ Ton/m}^2 \quad 331.428571 \text{ Kg/m}^2$$

Beban Hidup Slab bentang 5:

$$\text{Luas Slab} = 21.00 \text{ m}^2$$

$$Qs5 = 0.331 \text{ Ton/m}^2 \quad 331.42857 \text{ Kg/m}^2$$

Beban Hidup Slab bentang 6:

$$\text{Luas Slab} = 27 \text{ m}^2$$

$$Qs6 = 0.258 \text{ Ton/m}^2 \quad 257.77778 \text{ Kg/m}^2$$

Beban Hidup Slab bentang 7:

$$\text{Luas Slab} = 5.625 \text{ m}^2$$

$$Qs6 = 1.237 \text{ Ton/m}^2 \quad 1237.3333 \text{ Kg/m}^2$$

### C. Beban Spring pada Tiang Pancang

Berdasarkan Data BH VI : dimana : Cu adalah Shear Undrained

Kh = 67 . Cu/d nh adalah Typical Values

Kh = nh . z/d Softer soil

nh = 1 lb/in<sup>3</sup> = 27679.9 kg/m<sup>3</sup>

0.0002768 N/mm<sup>3</sup>

d = 0.6 m

z adalah kedalaman komulatif per 1 m

d adalah diameter tiang pancang

Kedalaman (z)	Cu Ton/m <sup>2</sup>	Kh Kg/m <sup>3</sup>	Spring kg/m
0 m	*	0.00	0.00
1 m	*	46133.17	27679.90
2 m	*	92266.33	55359.80
3 m	*	138399.50	83039.70
4 m	*	184532.67	110719.60
5 m	0.53	59183.33	35581.00
6 m	*	276799.00	166079.40
7 m	*	322932.17	0.00
8 m	*	369065.33	0.00
9 m	*	415198.50	0.00
10 m	0.43	48016.67	0.00
11 m	*	507464.83	0.00
12 m	*	553598.00	0.00
13 m	*	599731.17	0.00
14 m	*	645864.33	0.00
15 m	0.63	70350.00	42210.00
16 m	*	738130.67	442878.40
17 m	*	784263.83	470558.30
18 m	*	830397.00	498238.20
19 m	*	876530.17	525918.10
20 m	*	922663.33	553598.00

Kedalaman	Cu Ton/m <sup>2</sup>	Kh Kg/m <sup>3</sup>	Spring kg/m
21 m	*	968796.50	581277.90
22 m	*	1014929.67	608957.80
23 m	*	1061062.83	636637.70
24 m	*	1107196.00	664317.60
25 m	*	1153329.17	691997.50
26 m	*	1199462.33	719677.40

### D. Perhitungan Buckling

$$E = 4700 * (fc')^{0.5} = 27081.1373 \text{ N/mm}^2$$

$$I = \pi d^4 / 64 = 3293763548 \text{ mm}^4$$

$$nh = \text{buku poulous+davis hal} = 0.0002768 \text{ N/mm}^3$$

$$T = (EI/nh)^{1/5} = 3174.23415 \text{ mm}$$

$$Z_{max} = L/T = 9.45109861 \text{ Long Pile}$$

$$P_{cr} = \frac{\pi^2 E I_p}{4(ST + JT)^2 T^2} \text{ dimana: } ST = Ls/T \quad JT = Lu/T$$

$$p \text{ load max} = 10000 \text{ kg}$$

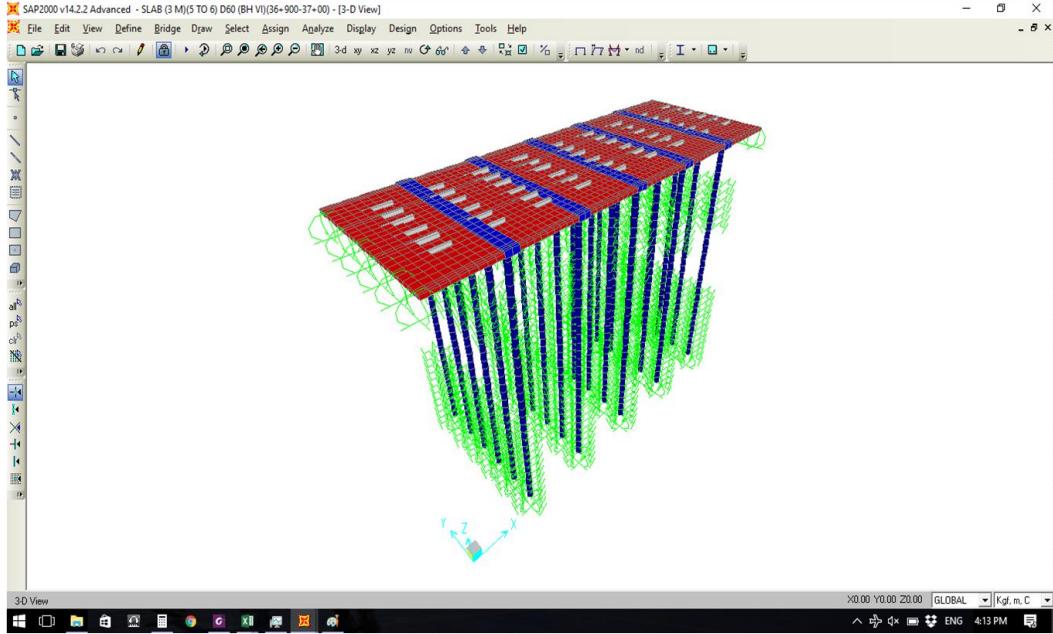
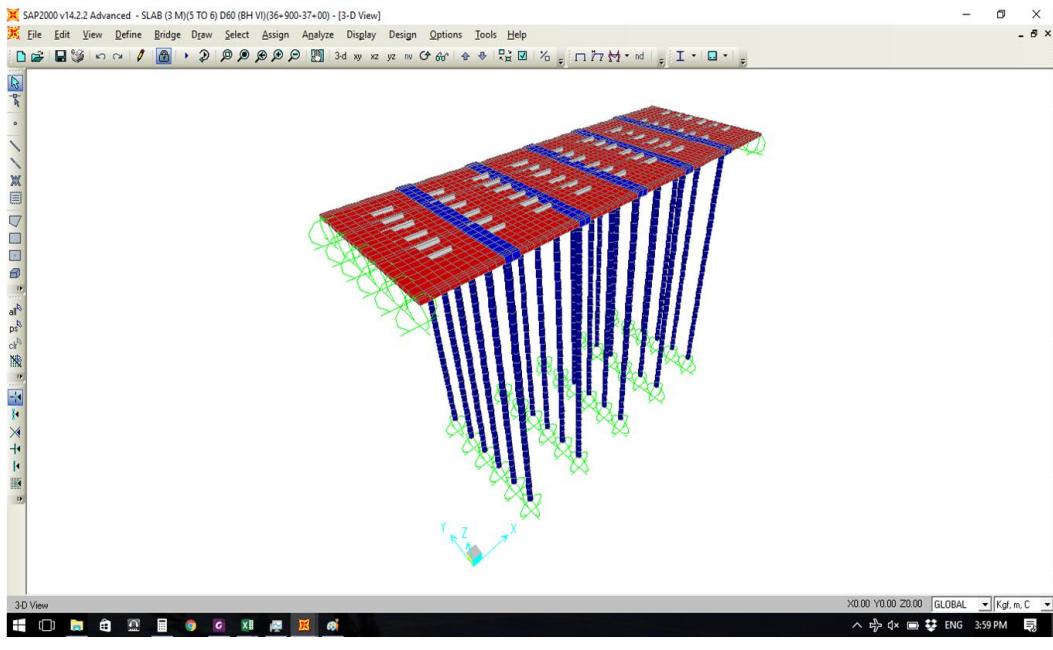
$$P_{cr} = 1072700 \text{ kg} \quad 10727000 \text{ N}$$

$$Ls + Lu = 4529.606 \text{ mm} \quad 4.52960567 \text{ m}$$

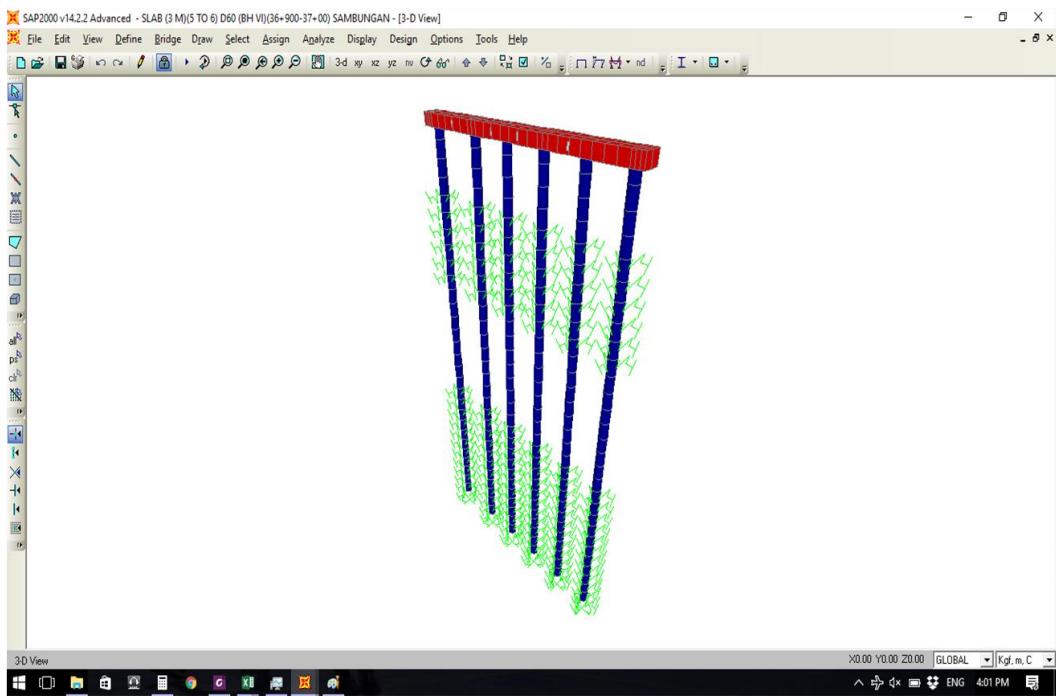
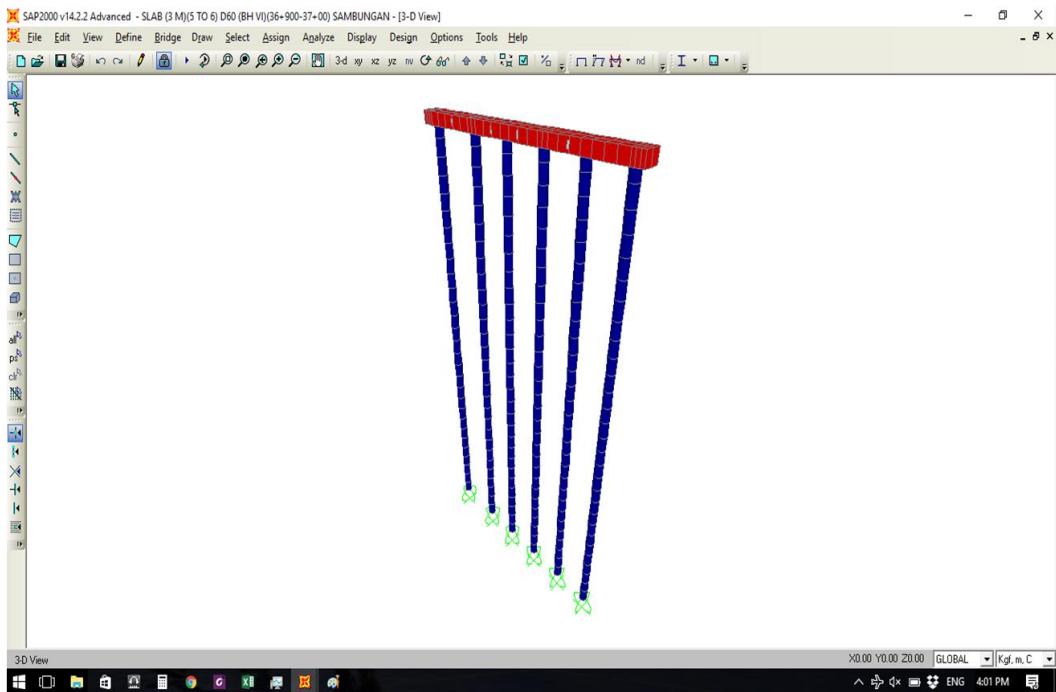
**PERHITUNGAN PILED SLAB SEKSI 1**  
**RUAS PORONG - CARAT**  
**PROYEK JALAN TOL PORONG - KEJAPANAN (ISI 6)(STA 36+900 - 37+00) FREE STANDING 3 M**

Struktur tersebut dianalisa statika dengan menggunakan program SAP2000 Ver. 14.2.2

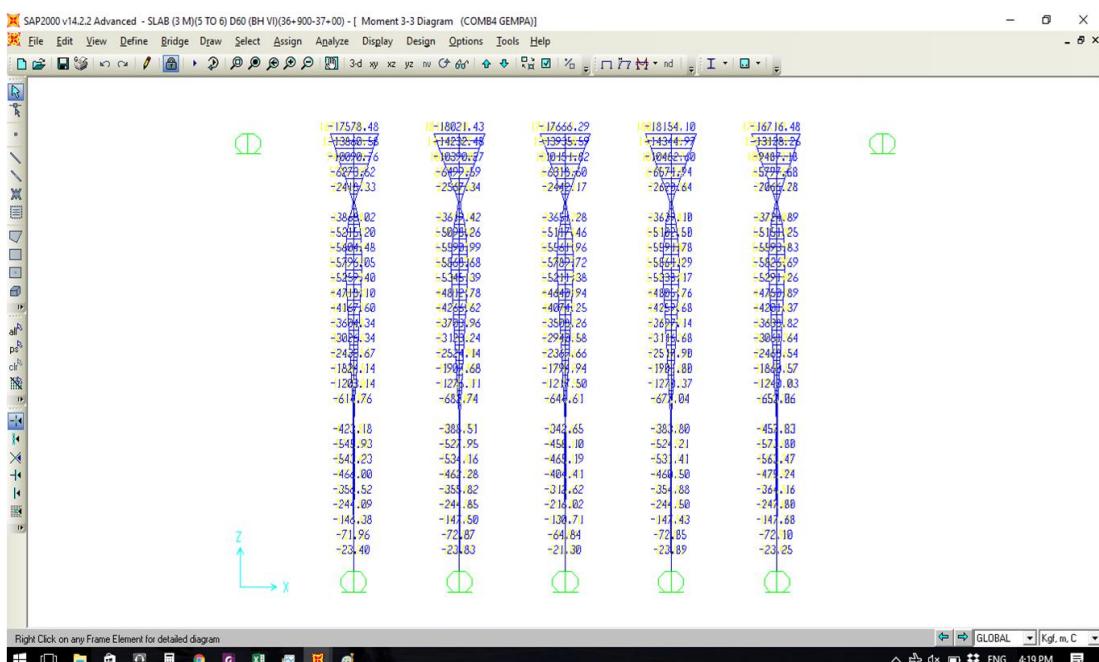
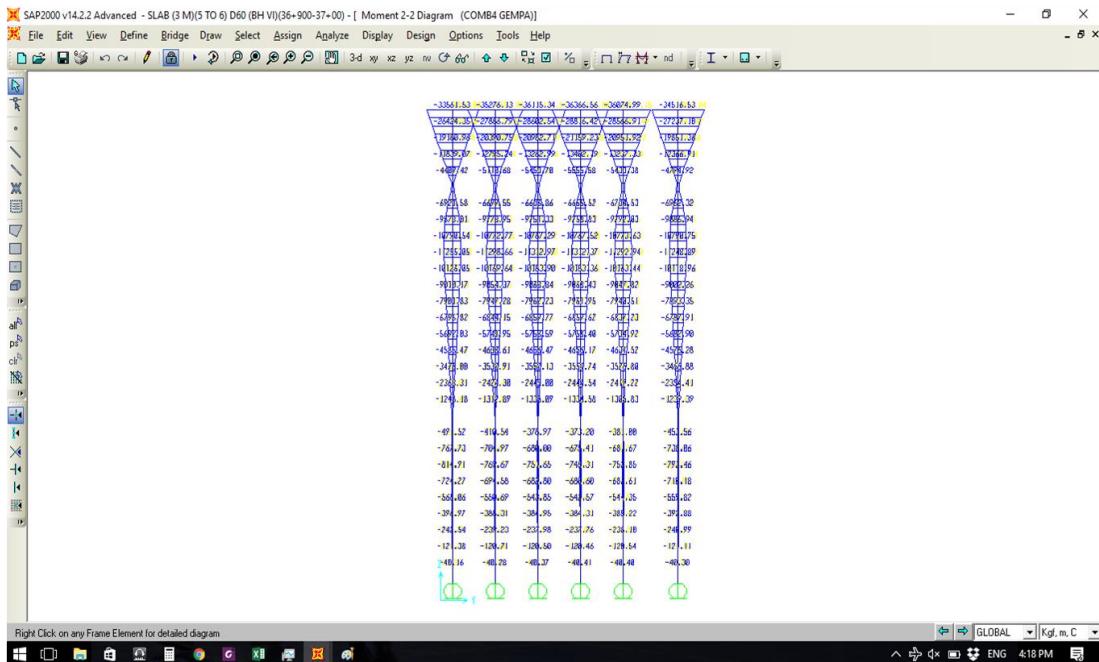
Berikut merupakan tampilan permodelan struktur pile slab:



Tampilan permodelan struktur pada sambungan pile slab:



#### 4. TIANG PANCANG D 60 CM



Dari hasil output SAP 2000 diperoleh Gaya-gaya pada TP D60 cm :

Mult = **58702.98** Kg.m

Pult = **73351.29** Kg

Vult = **12900.40** Kg

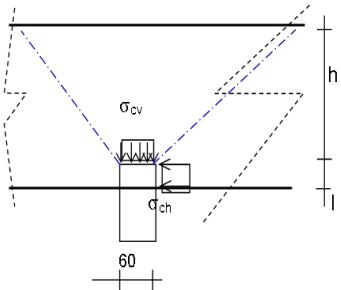
- Kepala tiang terjepit :

$$\begin{aligned} M &= 58.7030 \quad \text{ton.m} \\ N &= 73.351 \quad \text{ton} \end{aligned}$$

\*. Pengecekan Footing terhadap kepala tiang pancang :

- Tegangan dukung vertikal pada footing :

$$\begin{aligned}\sigma_{cv} &= \frac{P}{1/4 \cdot \pi \cdot D^2} = \frac{73351.290}{2827.4} \\ &= 25.943 \text{ Kg/cm}^2 < 130.725 \text{ Kg/cm}^2 \\ &\quad \text{OK}\end{aligned}$$



- Tegangan geser pons vertikal pada footing :

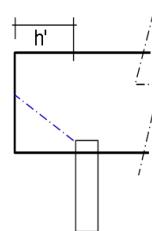
$$\begin{aligned}\tau_{cv} &= \frac{P}{(\pi \cdot D + h) \cdot h} \\ &= \frac{73351.29}{13865.02} = 5.290 \text{ Kg/cm}^2 < 8.983 \text{ Kg/cm}^2 \\ &\quad \text{OK}\end{aligned}$$

- Tegangan dukung horisontal pada footing :

$$\begin{aligned}\sigma_{ch} &= \frac{H}{D \cdot l} = \frac{12900.4000}{50 \times 10} \\ &= 25.801 \text{ Kg/cm}^2 < 87.15 \text{ Kg/cm}^2 \\ &\quad \text{OK}\end{aligned}$$

- Tegangan geser pons horisontal pada footing :

$$\begin{aligned}\tau_{ch} &= \frac{H}{(2l + D + 2h') \cdot h'} \\ &= \frac{12900.40}{8500.00} \\ &= 1.518 \text{ Kg/cm}^2 < 6.737 \text{ Kg/cm}^2 \\ &\quad \text{OK}\end{aligned}$$



\*. Pengecekan jumlah tulangan pada kepala tiang

$$\begin{aligned}M &= 58.7030 \text{ Ton.m} = 587029800 \text{ N.mm} & F_y &= 390 \text{ Mpa} \\ N &= 73.351 \text{ Ton} = 733512.9 \text{ N} & f'_c &= 33.2 \text{ Mpa}\end{aligned}$$

$$\begin{aligned}Pu/\emptyset A_{gr} \cdot 0.85 f'_c \cdot (ef/h) &= 0.0187 \xrightarrow{\text{( Dari BMS '92 diagram 5.19 )}} \\ Pu/\emptyset A_{gr} \cdot 0.85 f'_c &= 0.1192 \quad d'/h = 0.18 \\ &\quad f'_c = 32 \text{ Mpa}\end{aligned}$$

$$\text{Diperoleh } \rho = r \beta \xrightarrow{\text{Maka dipakai } \rho \text{ sebesar}} = 0.01 \times 1.287 = 0.01287$$

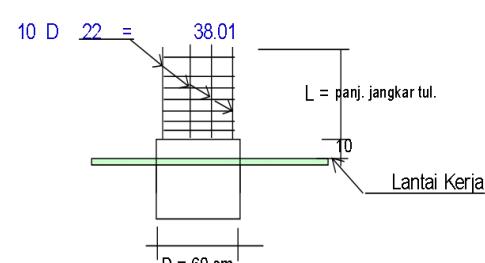
$$As \text{ total} = \rho \times A_{gr} = 36.39 \text{ cm}^2$$

Dipakai tulangan total 10 D22 ( 38.01 cm<sup>2</sup> )

$$\begin{aligned}L &= \sigma_s \cdot As / \tau_a \cdot u \\ &= 3900 \cdot 28.35 \\ &\quad 8.98 \cdot \pi \cdot 50 \\ &= 105.10\end{aligned}$$

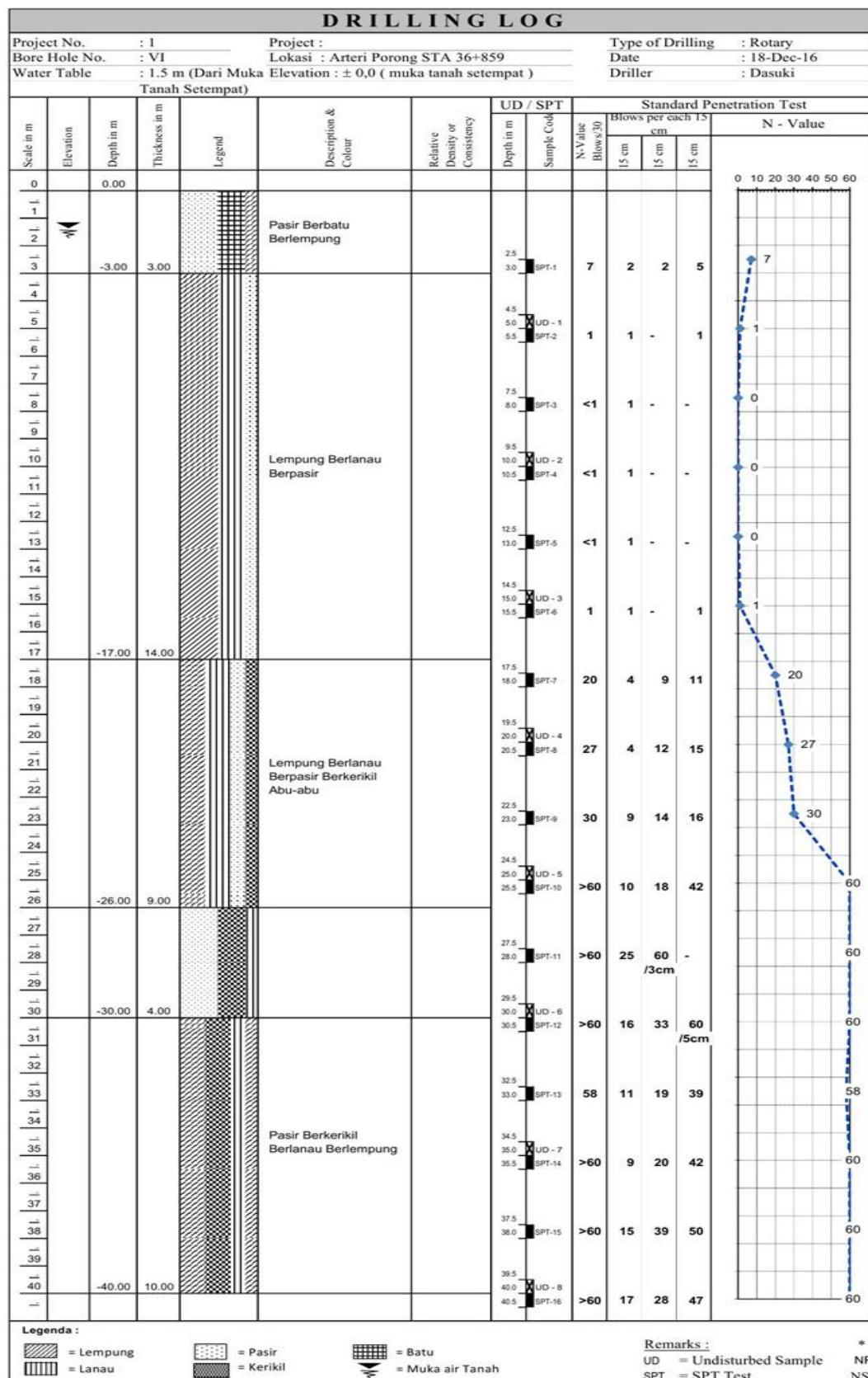
Jadi L diambil = 110 cm

$$Mult = 48.919 \text{ Ton.M}$$



$$M_{crack} = 19.568 \text{ Ton.M}$$

Dipakai Tiang Pancang Spun Pile  $\emptyset 60$  cm Type B Panjang Tiang 30,0 m



### GEMPA RESPON SPEKTRUM

Berdasarkan RSNI 2833:201X → Gempa 7% dalam 75 tahun (Gempa periode 1000 tahun)

#### KLASIFIKASI TANAH

Struktur : Pile Slab (Porong Gempol)

Lokasi : Sidoarjo

Lapisan ke-i	Tebal lapisan	Jenis tanah	Nilai N-SPT
1	3	Pasir berbatu berlempung	3.50
2	14	Lempung berlanau berpasir	1.50
3	9	Lempung berlanau berpasir berkerikil abu-abu	34.25
4	4	Pasir berkerikil berlanau	60.00
5	10	Pasir berkerikil berlanau berlempung	59.60

$$\sum d_i = 40$$

$$\sum N_i = 10.69$$

$$N = \frac{\sum d_i}{\sum N_i}$$

$$= 3.743 \rightarrow \text{SE (tanah lunak)}$$

#### PERHITUNGAN RESPON SPEKTRAL

Berdasarkan data tanah SPT tersebut, tanah masuk dalam klasifikasi SE (tanah lunak)

Lokasi : Sidoarjo

Nilai  $S_s = 0.400$

$S_1 = 0.200$

PGA = 0.250

} didapat dari Peta Zonasi Gempa Indonesia

#### **Koefisien Situs, Fa**

Kelas Situs	Parameter Respons Spektral Percepatan Gempa MCE <sub>R</sub> Terpetakan pada Periode Pendek, T=0,2 detik, S <sub>s</sub>				
	S <sub>s</sub> ≤ 0.25	S <sub>s</sub> = 0.5	S <sub>s</sub> = 0.75	S <sub>s</sub> = 1	S <sub>s</sub> ≥ 1.25
SA	0.8	0.8	0.8	0.8	0.8
SB	1	1	1	1	1
SC	1.2	1.2	1.1	1	1
SD	1.6	1.4	1.2	1.1	1
SE	2.5	1.7	1.2	0.9	0.9
SF	SS <sup>b</sup>				

#### **Koefisien Situs, Fv**

Kelas Situs	Parameter Respons Spektral Percepatan Gempa MCE <sub>R</sub> Terpetakan pada Periode Pendek, T=1 detik, S <sub>1</sub>				
	S <sub>1</sub> ≤ 0.1	S <sub>1</sub> = 0.2	S <sub>1</sub> = 0.3	S <sub>1</sub> = 0.4	S <sub>1</sub> ≥ 0.5
SA	0.8	0.8	0.8	0.8	0.8
SB	1	1	1	1	1
SC	1.7	1.6	1.5	1.4	1.3
SD	2.4	2	1.8	1.6	1.5
SE	3.5	3.2	2.8	2.4	2.4
SF	SS <sup>b</sup>				

#### Faktor Amplifikasi untuk PGA, F<sub>PGA</sub>

Kelas Situs	Parameter Respons Spektral Percepatan Gempa MCE <sub>R</sub> Terpetakan pada Periode Pendek, T=0,2 detik, S <sub>s</sub>				
	PGA ≤ 0.1	PGA = 0.2	PGA = 0.3	PGA = 0.4	PGA ≥ 0.5
SA	0.8	0.8	0.8	0.8	0.8
SB	1	1	1	1	1
SC	1.2	1.2	1.1	1	1
SD	1.6	1.4	1.2	1.1	1
SE	2.5	1.7	1.2	0.9	0.9
SF	S <sub>s</sub> <sup>b</sup>				

Dari tabel tersebut didapatkan nilai : F<sub>a</sub> = 2.020

F<sub>v</sub> = 3.200

F<sub>PGA</sub> = 1.450

Nilai F<sub>a</sub> dan F<sub>v</sub> selanjutnya digunakan untuk menghitung parameter percepatan spektral desain untuk periode pendek, S<sub>DS</sub>, dan untuk periode 1 detik, S<sub>D1</sub>, dapat dihitung sebagai berikut:

$$\begin{aligned} S_{DS} &= F_a \cdot S_s \\ &= 2.02 \times 0.4 \\ &= 0.808 \end{aligned}$$

$$\begin{aligned} S_{D1} &= F_v \cdot S_1 \\ &= 3.2 \times 0.2 \\ &= 0.640 \end{aligned}$$

$$\begin{aligned} A_s &= F_{PGA} \cdot PGA \\ &= 1.45 \times 0.25 \\ &= 0.363 \end{aligned}$$

Menentukan periode getar T<sub>o</sub> dan T<sub>s</sub>

$$\begin{aligned} T_o &= 0,20 \times S_{D1} / S_{DS} \\ &= 0.158 \end{aligned}$$

$$\begin{aligned} T_s &= S_{D1} / S_{DS} \\ &= 0.792 \end{aligned}$$

Membuat grafik respon spektra

Berdasarkan **RSNI 2833:201X Pasal 5.4.2** Koefisien repon gempa elastik adalah sebagai berikut:

$$\text{Untuk } T < T_o \rightarrow C_{sm} = (S_{DS} - A_s) \frac{T}{T_o} + A_s$$

$$\text{Untuk } T \geq T_o \rightarrow C_{sm} = S_{DS}$$

$$\text{Untuk } T \geq T_s \rightarrow C_{sm} = \frac{S_{D1}}{T}$$

T	0	0.158	0.792	0.80	1.00	1.20	1.40	1.60	1.80	2.00	2.20	2.40	2.60
C <sub>sm</sub>	0.363	0.808	0.808	0.800	0.640	0.533	0.457	0.400	0.356	0.320	0.291	0.267	0.246

Tabel 5 - Zona Gempa (**RSNI 2833:201X**)

Koefisien percepatan (S <sub>D1</sub> )	Zona gempa
SD1 ≤ 0.15	1
0.15 < SD1 ≤ 0.30	2
0.30 < SD1 ≤ 0.50	3
SD1 > 0.50	4

Tabel 6 - Faktor modifikasi respon (R) untuk bangunan bawah (**RSNI 2833:201X**)

Bangunan bawah	Kategori kepentingan		
	Sangat penting	Penting	Lainnya
Pilar tipe dinding	1.5	1.5	2.0
Tiang/kolom beton bertulang			
Tiang vertikal	1.5	2.0	3.0
Tiang miring	1.5	1.5	2.0
Kolom tunggal	1.5	2.0	3.0
Tiang baja dan komposit			
Tiang vertikal	1.5	3.5	5.0
Tiang miring	1.5	2.0	3.0
Kolom majemuk	1.5	3.5	5.0

→ Arah memanjang  
(antar pier head / tiang tunggal)

→ Arah melintang  
(kelompok tiang dalam satu pierhead)

**Faktor Pengali Gempa (Scale Factor)** (untuk SAP2000 / load case)

Arah memanjang → 5.886 → U1

Arah melintang → 3.363 → U2

**Persyaratan P-Δ Arah X**Syarat :  $\Delta P_u < 0.25 \phi M_n$  >>> 107.014 < 14675.7 OKE

Dimana:

$$\Delta = R_d \Delta_e = 0.00146$$

Bila  $T < 1.25T$  Maka  $R_d = (1-1/R) 1.25 T_s/T + 1/R$ Bila  $T \geq 1.25T$  Maka  $R_d = 1$ 

$$T = 0.8 < 0.99 \text{ Maka } R_d = (1-1/R) 1.25 T_s/T + 1/R$$

$$R_d = (1-1/R) 1.25 T_s/T + 1/R = 1.11881$$

**Persyaratan P-Δ Arah Y**Syarat :  $\Delta P_u < 0.25 \phi M_n$  >>> 59.2029 < 14675.7 OKE

Dimana:

$$\Delta = R_d \Delta_e = 0.00081$$

Bila  $T < 1.25T$  Maka  $R_d = (1-1/R) 1.25 T_s/T + 1/R$ Bila  $T \geq 1.25T$  Maka  $R_d = 1$ 

$$T = 0.8 < 1 \text{ Maka } R_d = (1-1/R) 1.25 T_s/T + 1/R$$

$$R_d = (1-1/R) 1.25 T_s/T + 1/R = 1.16973$$

**HASIL SELISIH MOMEN ANTARA TAMPA DAN DENGAN P-Δ**

Momen tanpa P-Δ = 58702.98 kgm

Persen selisih = 1.30516

Momen dengan P-Δ = 59479.28 kgm

**PENGECERAN PERGOYANGAN LATERAL**

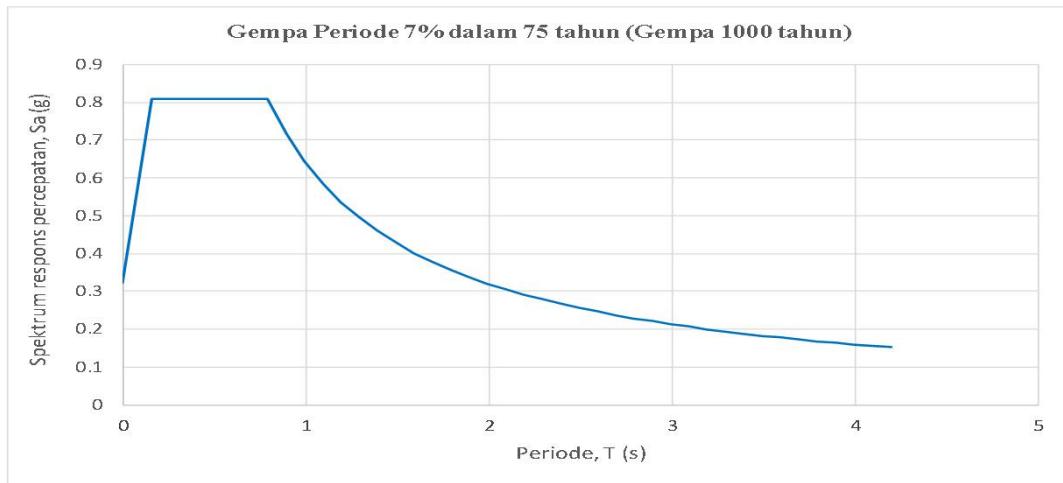
Syarat Pergoyangan Lateral Ijin diambil dari SNI 1726:2012

0.02 hx &gt; Pergoyangan Arah X dan Arah Y yang terjadi

dimana hx adalah Tinggi tiang diukur dari jepit virtual

**Arah X** 0.02 hx > U1 max  
0.091 > 0.0013 OKE**Arah Y** 0.02 hx > U2 max  
0.091 > 0.00069 OKE

### RESPON SPEKTRUM



	T	$S_a$
	0	0.323
$T_0$	0.158	0.808
$T_s$	0.792	0.808
$T_s + 0.1$	0.892	0.717
$T_s + 0.2$	0.992	0.645
$T_s + 0.3$	1.092	0.586
$T_s + 0.4$	1.192	0.537
$T_s + 0.5$	1.292	0.495
$T_s + 0.6$	1.392	0.460
$T_s + 0.7$	1.492	0.429
$T_s + 0.8$	1.592	0.402
$T_s + 0.9$	1.692	0.378
$T_s + 1$	1.792	0.357
$T_s + 1.1$	1.892	0.338
$T_s + 1.2$	1.992	0.321
$T_s + 1.3$	2.092	0.306
$T_s + 1.4$	2.192	0.292
$T_s + 1.5$	2.292	0.279
$T_s + 1.6$	2.392	0.268
$T_s + 1.7$	2.492	0.257
$T_s + 1.8$	2.592	0.247
$T_s + 1.9$	2.692	0.238
$T_s + 2$	2.792	0.229
$T_s + 2.1$	2.892	0.221
$T_s + 2.2$	2.992	0.214
$T_s + 2.3$	3.092	0.207
$T_s + 2.4$	3.192	0.200
$T_s + 2.5$	3.292	0.194
$T_s + 2.6$	3.392	0.189
$T_s + 2.7$	3.492	0.183
$T_s + 2.8$	3.592	0.178
$T_s + 2.9$	3.692	0.173
$T_s + 3$	3.792	0.169
$T_s + 3.1$	3.892	0.164
$T_s + 3.2$	3.992	0.160
$T_s + 3.3$	4.092	0.156
$T_s + 3.4$	4.192	0.153
$T_s + 4$	4.000	0.160

### Penulangan PH Tengah (60 cm)

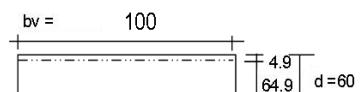
Keterangan	Nilai	Satuan
Tipe pelat	PH 2	-
f <sub>c</sub>	30	Mpa
f <sub>y</sub>	400	Mpa
B1	0.85	-
b	1000	mm
Decking	100	mm
Tebal pelat (h)	600	mm
Panjang (Ly)	1630	cm
Pendek (Lx)	140	cm

Keterangan	Rumus	Nilai	Satuan	Syarat & Kesimpulan
Rasio	Ly/Lx	11.64285714	-	<b>One way slab</b>
O tulangan lentur		19	mm	
P <sub>min</sub>	1.4/f <sub>y</sub>	0.0035	-	
P <sub>balance</sub>	$\frac{0.85 \cdot f_c' \cdot \beta}{f_y} \cdot \left[ \frac{600}{600 + f_y} \right]$	0.0325125	-	
P <sub>max</sub>	0.75 · P <sub>b</sub>	0.024384375	-	
d <sub>x</sub>	t <sub>p</sub> -decking-Otul	490.5	mm	
d <sub>y</sub>	t <sub>p</sub> -decking-Otul-0.5Otul	471.5	mm	
m	f <sub>y</sub> /0.85f <sub>c</sub>	15.68627451	-	
<b>Perhitungan Penulangan</b>				
<b>Tumpuan x</b>				
M <sub>tx</sub>		395516100	Nmm	$\rho_{min} < \rho_{perlu} < \rho_{max}$
M <sub>n</sub>	Mu/σ	439462333.3	-	0.0035 < 0.0047 < 0.0244
R <sub>n</sub>	Mn/bd <sup>2</sup>	1.826600759	-	
P perlu	$\frac{1}{m} \left[ 1 - \sqrt{1 - \frac{2mRn}{f_y}} \right]$	0.004742937	-	Maka dipakai = 0.0047
As perlu	P.b.d	2326.410498	mm <sup>2</sup>	
Pakai Tulangan		19	mm	Syarat spasi antar tulangan
S <sub>max</sub>	2. h	1200	mm	S <sub>max</sub> > S 1200 > 121.81 <b>OK</b>
S	(0.25 π Ø <sup>2</sup> b)/As	121.8121222	mm	
S pakai		125	mm	
As pakai	0.25 π Ø <sup>2</sup> b)/(S pakai)	2267.08	mm <sup>2</sup>	As pakai > As perlu 2267.1 < 2326.4 <b>NO OK</b>
Maka menggunakan :				
Tulangan Ø		19	mm	
Jarak		125	mm	
<b>Lapangan x</b>				
M <sub>lx</sub>		290660700	Nmm	$\rho_{min} < \rho_{perlu} < \rho_{max}$
M <sub>n</sub>	Mu/σ	322956333.3	Nmm	0.0035 > 0.0034 < 0.0244
R <sub>n</sub>	Mn/bd <sup>2</sup>	1.342350047	-	
P perlu	$\frac{1}{m} \left[ 1 - \sqrt{1 - \frac{2mRn}{f_y}} \right]$	0.003449184	-	Maka diperbesar 30% = 0.0045
As perlu	P.b.d	2199.372112	mm <sup>2</sup>	
Pakai Tulangan		19	mm	Syarat spasi antar tulangan
S <sub>max</sub>	2. h	1200	mm	S <sub>max</sub> > S 1200 > 128.85 <b>OK</b>
S	(0.25 π Ø <sup>2</sup> b)/As	128.8481373	mm	
S pakai		125	mm	
As pakai	0.25 π Ø <sup>2</sup> b)/(S pakai)	2267.08	mm <sup>2</sup>	As pakai > As perlu 2267.1 > 2199.4 <b>OK</b>
Maka menggunakan :				
Tulangan Ø		19	mm	
Jarak		125	mm	
<b>Tumpuan y</b>				
M <sub>ly</sub>		290325400	Nmm	

Mn	Mu/ø	322583777.8	Nmm	$\rho_{min} < \rho_{perlu} < \rho_{max}$
Rn	Mn/bd <sup>2</sup>	1.451039148	-	0.0035 < 0.0037 < 0.0244
P perlu	$\frac{1}{m} \left[ 1 - \sqrt{1 - \frac{2mRn}{fy}} \right]$	0.003737137	-	Maka dipakai = 0.00374
As perlu	P.b.d	1762.059914	mm <sup>2</sup>	Syarat spasi antar tulangan
Pakai Tulangan		16	mm	Smax > S 1200 > 114.05 <b>OK</b>
Smax	2. h	1200	mm	
S	(0.25 π Ø <sup>2</sup> b)/As	114.0483354	mm	
S pakai		100	mm	As pakai > As perlu 2009.6 > 1762.1 <b>OK</b>
As pakai	0.25 π Ø <sup>2</sup> b)/(S pakai)	2009.6	mm <sup>2</sup>	
Maka menggunakan :				
Tulangan Ø	16	mm		
Jarak	100	mm		
Lapangan y				
M <sub>lx</sub>		231945000	Nmm	$\rho_{min} < \rho_{perlu} < \rho_{max}$
Mn	Mu/ø	257716666.7	Nmm	0.0035 > 0.0030 < 0.0244
Rn	Mn/bd <sup>2</sup>	1.159255357	-	Maka diperbesar 30% = 0.003857348
P perlu	$\frac{1}{m} \left[ 1 - \sqrt{1 - \frac{2mRn}{fy}} \right]$	0.002967191	-	Syarat spasi antar tulangan
As perlu	P.b.d	1818.739798	mm <sup>2</sup>	Smax > S 1200 > 110.49 <b>OK</b>
Pakai Tulangan		16	mm	
Smax	2. h	1200	mm	
S	(0.25 π Ø <sup>2</sup> b)/As	110.4940906	mm	
S pakai		100	mm	As pakai > As perlu 2009.6 > 1818.7 <b>OK</b>
As pakai	0.25 π Ø <sup>2</sup> b)/(S pakai)	2009.6	mm <sup>2</sup>	
Maka menggunakan :				
Tulangan Ø	16	mm		
Jarak	100	mm		

#### PERHITUNGAN SHEAR CONECTOR PADA PIERHEAD & SLAB

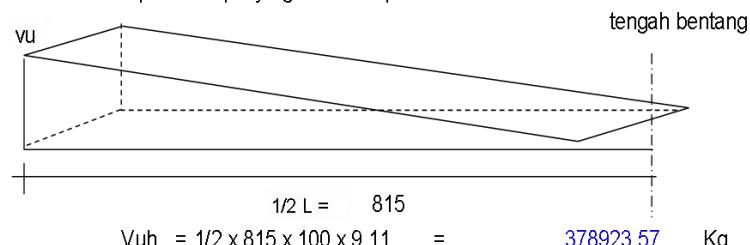
$$V_{uh} < \Phi V_n = \Phi A_v f_y \kappa$$



Tegangan geser Horisontal :

$$\begin{aligned} v_{uh} &= \frac{V_u}{b v \cdot d} = \frac{45610.31}{100.00 \times 49.1} \\ &= 9.30 \text{ Kg/cm}^2 \end{aligned}$$

Total geser Horisontal pada 1/2 panjang PH antar perletakan total :



$$V_{uh} = 1/2 \times 815 \times 100 \times 9.11 = 378923.57 \text{ Kg}$$

Tulangan yang dibutuhkan :

$$\begin{aligned} A_v & \frac{V_{uh}}{\Phi f_y \kappa} = \frac{378923.57}{0.85 \times 4000 \times 0.9} = 121.44 \text{ cm}^2 \end{aligned}$$

Ukuran PH adalah : 1,00 x 0,6

Jadi dalam satu PH dibutuhkan Luas shear conector = 121.44 cm<sup>2</sup> = 12144 mm<sup>2</sup>

Dipakai shearconector dalam satu PH D19 - 250 = 18486.07365 mm<sup>2</sup>

### Penulangan PH Tepi ( $t = 70 \text{ cm}$ )

Keterangan	Nilai	Satuan
Tipe pelat	PH 1	-
$f_c$	30	Mpa
$f_y$	400	Mpa
B1	0.85	-
b	1000	mm
Decking	100	mm
Tebal pelat (h)	700	mm
Panjang (Ly)	1630	cm
Pendek (Lx)	100	cm

Keterangan	Rumus	Nilai	Satuan & Kesimpulan	
Rasio	$Ly/Lx$	16.3	-	<b>One way slab</b>
$\emptyset$ tulangan lentur		19	mm	
$P_{min}$	$1.4/f_y$	0.0035	-	
$P_{balance}$	$\frac{0.85 \cdot f_c' \cdot \beta}{f_y} \cdot \left[ \frac{600}{600 + f_y} \right]$	0.0325125	-	
$P_{max}$	$0.75 \cdot P_b$	0.024384375	-	
$d_x$	tp-decking- $\emptyset$ tul	590.5	mm	
$d_y$	tp-decking- $\emptyset$ tul-0.5 $\emptyset$ tul	571.5	mm	
m	$f_y/0.85f_c$	15.68627451	-	

#### Perhitungan Penulangan

Tumpuan x		$\rho_{min} < \rho_{perlu} < \rho_{max}$		
M <sub>tx</sub>		293395600	Nmm	
M <sub>n</sub>	M <sub>u</sub> / $\emptyset$	325995111.1	Nmm	
R <sub>n</sub>	M <sub>n</sub> /bd <sup>2</sup>	0.934913182	-	
P perlu	$\frac{1}{m} \left[ 1 - \sqrt{1 - \frac{2mR_n}{f_y}} \right]$	0.002381776	-	
As perlu	P.b.d	2066.75	mm <sup>2</sup>	
Pakai Tulangan		19	mm	
S <sub>max</sub>	2, h	1400	mm	Syarat spasi antar tulangan
S	(0.25 $\pi \emptyset^2 b$ )/As	137.1162453	mm	1400 > 137.12 <b>OK</b>
S pakai		125	mm	
As pakai	0.25 $\pi \emptyset^2 b$ )/(S pakai)	3173.912	mm <sup>2</sup>	As pakai > As perlu 3173.9 > 2066.8 <b>OK</b>
Maka menggunakan :				
Tulangan $\emptyset$	19	mm		
Jarak	125	mm		

Lapangan x		$\rho_{min} < \rho_{perlu} < \rho_{max}$		
M <sub>tx</sub>		281955700	Nmm	
M <sub>n</sub>	M <sub>u</sub> / $\emptyset$	313284111.1	Nmm	
R <sub>n</sub>	M <sub>n</sub> /bd <sup>2</sup>	0.898459625	-	
P perlu	$\frac{1}{m} \left[ 1 - \sqrt{1 - \frac{2mR_n}{f_y}} \right]$	0.002287178	-	
As perlu	P.b.d	2066.75	mm <sup>2</sup>	
Pakai Tulangan		19	mm	
S <sub>max</sub>	2, h	1400	mm	Syarat spasi antar tulangan
S	(0.25 $\pi \emptyset^2 b$ )/As	137.1162453	mm	1400 > 137.12 <b>OK</b>
S pakai	(0.25 $\pi \emptyset^2 b$ )/As	125	mm	
As pakai	0.25 $\pi \emptyset^2 b$ )/(S pakai)	3173.912	mm <sup>2</sup>	As pakai < As perlu 3173.9 > 2066.8 <b>OK</b>
Maka menggunakan 0.25 $\pi \emptyset^2 b$ )/(S pakai)				
Tulangan $\emptyset$	19	mm		
Jarak	125	mm		
Tumpuan y				
M <sub>tx</sub>		213985300	Nmm	

Mn	Mu/o	237761444.4	-	$\rho_{min} < \rho_{perlu} < \rho_{max}$
Rn	Mn/bd <sup>2</sup>	0.727962422	-	0.0035 > 0.0018 < 0.0244
P perlu	$\frac{1}{m} \left[ 1 - \sqrt{1 - \frac{2mRn}{fy}} \right]$	0.001846652	-	OK
As perlu	b.b.d	2000.25	mm <sup>2</sup>	Maka diperbesar 30% = 0.0024
Pakai Tulangan		16	mm	Syarat spasi antar tulangan
Smax	2. h	1400	mm	Smax > S 1400 > 100.47
S	(0.25 π Ø^2 b)/As	100.4674416	mm	OK
S pakai	(0.25 π Ø^2 b)/As	100	mm	
As pakai	0.25 π Ø^2 b)/(S pakai)	2009.6	mm <sup>2</sup>	As pakai > As perlu 2009.6 > 2000.3
Maka menggunakan 0.25 π Ø^2 b)/(S pakai)				OK
Tulangan Ø		16	mm	
Jarak		100	mm	
<b>Lapangan y</b>				
Mlx		161756600	Nmm	$\rho_{min} < \rho_{perlu} < \rho_{max}$
Mn	Mu/o	179729555.6	Nmm	0.0035 > 0.0014 < 0.0244
Rn	Mn/bd <sup>2</sup>	0.550284184	-	
P perlu	$\frac{1}{m} \left[ 1 - \sqrt{1 - \frac{2mRn}{fy}} \right]$	0.001390883	-	Maka diperbesar 30% = 0.001808148
As perlu	b.b.d	2000.25	mm <sup>2</sup>	
Pakai Tulangan		16	mm	Syarat spasi antar tulangan
Smax	2. h	1400	mm	Smax > S 1400 > 100.47
S	(0.25 π Ø^2 b)/As	100.4674416	mm	OK
S pakai	(0.25 π Ø^2 b)/As	100	mm	
As pakai	0.25 π Ø^2 b)/(S pakai)	2009.6	mm <sup>2</sup>	As pakai > As perlu 2009.6 > 2000.3
Maka menggunakan 0.25 π Ø^2 b)/(S pakai)				OK
Tulangan Ø		16	mm	
Jarak		100	mm	

**TECHNICAL CALCULATION  
PC FULL DEPTH SLAB FOR HIGHWAY BRIDGES**

Project : Full Slab Toll Parong-Gempol  
Product : PC FULL DEPTH SLAB 350x2710x7500 mm  
Job. No. : 17017 A  
Rev. No. : 06

Design Ref. : - SNI T-12-2004  
Perencanaan Struktur Beton Untuk Jembatan  
- SNI T-1725-2016  
Standar Pembebanan Untuk Jembatan  
- PCI : Bridge Design Manual

## TECHNICAL CALCULATION FOR PRETENSION FULL DEPTH SLAB

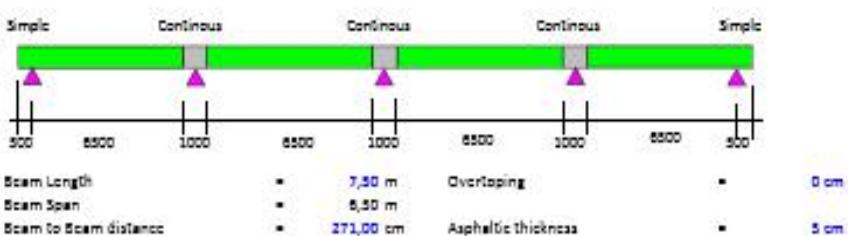
### A PENDAHULUAN

Metode pelaksanaan yang dilakukan pada pemasangan Full Slab Projek Toll Porong Gempol ini sama dengan metode pelaksanaan full slab jembatan pada umumnya, yaitu :

1. Plat precast diletakan dan bertemu pada rada ntar dalam kondisi simple span.
2. Komoditi dilakukan pregecoran dengan material non strungkage pada join antar slab.
3. Dan setelah itu dilakukan proses stresing lateral.

### B ANALISIS

#### I. BRIDGE DATA :



#### II. MATERIAL SPECIFICATION

##### 2.1 Concrete Material

###### 2.1.1. Preload Beam:

Concrete Strength at service :	$f_c' = 30,0 \text{ Mpa}$	$f_c' = 7,252 \text{ psi}$	$\sigma_{c, \text{allow}} = 35 \cdot (w^{1,5}) \cdot \sqrt{f_c' / f_{ck}}$
	= 3,25+05 kai	= 3,25+05 kg/cm <sup>2</sup>	= 3,25+04 MN/m <sup>2</sup>
Concrete Strength at release :	$f_{ci}' = 25,0 \text{ Mpa}$		

$$\text{Unit Weight } (w) = 2500 \text{ kg/m}^3 \\ = 156,05 \text{ pcf}$$

Allowable concrete stress at service ..... SNi T-12-2004

Compressive	$0,45 \cdot f_c'$	=	22,5 MN/m <sup>2</sup>
Tensile	$0,5 \cdot \text{Safe}(f_c')$	=	3,34 MN/m <sup>2</sup>

Allowable concrete stress at initial ..... SNi T-12-2004

Compressive	$0,67 \cdot f_{ci}'$	=	16,5 MN/m <sup>2</sup>
Edge Tensile	$0,5 \cdot \text{Safe}(f_{ci}')$	=	1,65 MN/m <sup>2</sup> (simple Span restrain)
Middle Tensile	$0,25 \cdot \text{Safe}(f_{ci}')$	=	1,32 MN/m <sup>2</sup> (simple Span restrain)

##### 2.2 Prestress Material

###### 2.2.1. Properties PC Strand

Prestress Steel	=	22 PC Strand Dia 12,7 mm
Steel Area (A <sub>s</sub> )	=	98,7 mm <sup>2</sup>
Ultimate Strength (f <sub>s</sub> )	=	1362 MN/m <sup>2</sup>
Modulus Elasticity (E <sub>s</sub> )	=	195055 MN/m <sup>2</sup>
Jacking Force	=	75% UTS

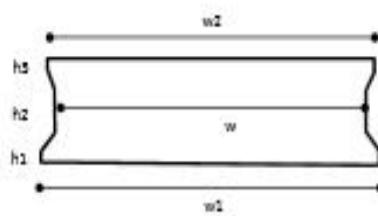
## 3.2.2. PC Strand configuration:

Line	N Strand	Ystrand (mm)	DEBONDED					JF/Strand (kN)	JACKING force (kN)
			1	1,5	2	2,5	3		
1	17	50	0	0	0	0	0	157,5	2345,2
2	5	150	0	0	0	0	0	157,5	659,2
3	0	250	0	0	0	0	0	157,5	0,0
TOTAL	22	91.364	0	0	0	0	0	TOTAL JF	3032,4

## III. SECTION ANALYSIS

## Cross Section

H = 35,0 cm  
 h1 = 9,0 cm  
 h2 = 4,0 cm  
 h3 = 11,0 cm  
 h4 = 8,5 cm  
 h5 = 2,5 cm  
 w = 285,0 cm  
 w2 = 285,0 cm  
 w1 = 271,0 cm



## 3.1 Precast Beam

[in cm]

Zone	Section Height	Width		Area cm <sup>2</sup>	Level cm	Yb cm	Area*Yb cm <sup>3</sup>	Iz cm <sup>4</sup>	Area*d <sup>2</sup> cm <sup>4</sup>	Iz cm <sup>4</sup>
		Bottom	Upper							
5	0,0	0,0	0,0	0	35	15,0	0	0	0	0
5	1,5	168,0	168,0	870	33	23,5	22613	349	173300	173349
4	3,5	168,0	168,0	1237	14	28,5	65783	13887	264330	173107
3	11,0	163,0	163,0	1293	13	18,5	53521	29171	3270	32441
2	4,0	271,0	263,0	1068	9	11,0	11737	1424	44333	45512
1	9,0	271,0	271,0	1439	0	4,5	10976	16463	402196	414657
Total	35,0			9327		17,4	161619	50994	595674	595683

## 3.2 Composite Beam

[in cm]

Zone	Height Section	Width		Area cm <sup>2</sup>	Level cm	Yb cm	Area*Yb cm <sup>3</sup>	Iz cm <sup>4</sup>	Area*d <sup>2</sup> cm <sup>4</sup>	Iz cm <sup>4</sup>
		Bottom	Upper							
2	0,0	0,0	0,0	0	35	15,0	0	0	0	0
1	35,0	271,0	0,0	9327	0	17,4	161619	50994	0	595683
Total	35,0			9327		17,4	161619	50994	0	595683

## IV. LOADING

## 4.1 Dead Load (DL)

Precast Slab      \* Ac full depth slab x Conc Weight  
                       \* 0,93265 m<sup>2</sup> x 2,5 t/m<sup>3</sup> = 2,33189 t/m<sup>3</sup>      22,33588 KN/m<sup>3</sup>

## 4.2 Non Composite Dead Load

Cf Joint      \* Ac joint x Conc Weight  
                       \* 0,00553 m<sup>2</sup> x 2,5 t/m<sup>3</sup> = 0,01383 t/m<sup>3</sup>      0,130556 KN/m<sup>3</sup>

## 4.3 Composite Dead Load

Ashphaltic      \* Ac Asphaltic x Asphaltic Weight  
                       \* 0,14 cm<sup>2</sup> m<sup>2</sup> x 2,2 t/m<sup>3</sup> = 0,2981 t/m<sup>3</sup>      1,323467 KN/m<sup>3</sup>

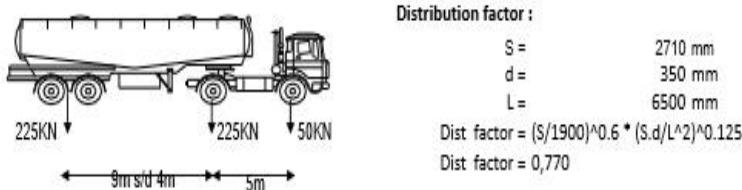
#### 4.4 Live Load (LL)

##### 4.4.1. "T" Loading (Beban Truk)

Taken from "SNI T-1725-2016"

$$DLA = 0,3$$

$$\text{Impact} = 1 + DLA = 1.3$$



##### 4.4.2. "D" Loading (Beban Lajur)

Taken from "SNI T-1725-2016"

##### D loading

a. Faktor Beban Dinamis (FBD)	$FBD = 1 + 0,4$	= 1,40	Span $\leq 50$ m
	$FBD = 1 + 0,525$	= 1,53	$50 < \text{Span} \leq 90$ m
	$FBD = 1 + 0,3$	= 1,30	$\text{Span} \geq 90$ m

b. Beban Garis Terpusat (BGT) = 49,00 KN/m

c. Distribution Factor (DF) = 1,00

##### d. Distribution Load

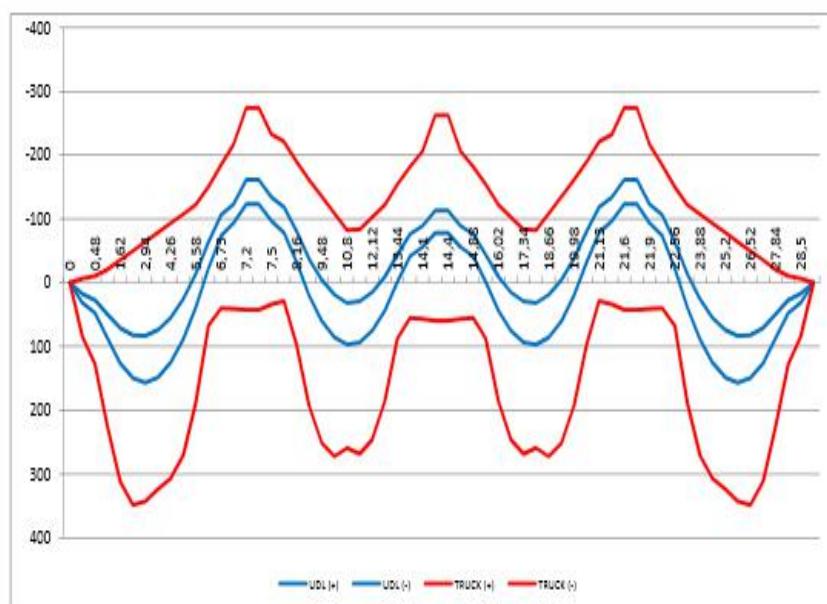
$$q = 9,00 \text{ kN/m}^2 \text{ which : } q = 9 \text{ kN/m}^2 \quad \text{for Span} \leq 30 \text{ m}$$

$$q = 9 \times (0,5 + 15/\text{span}) \text{ kN/m}^2 \quad \text{for Span} > 30 \text{ m}$$

##### e. Live load

$$\text{- Distribution load } q' = DF \times q \times s \\ = 1,00 \times 9,00 \text{ KN/m}^2 \times 2,7 \text{ m} = 24,39 \text{ KN/m}$$

$$\text{- Line load } p' = DF \times DLA \times KEL \times s \\ 1,00 \times 1,40 \times 49 \text{ KN/m} \times 2,71 \text{ m} = 185,906 \text{ KN}$$



RESUME : Moment force cause by Truck Loading is bigger than D Loading

## V. SERVICE DESIGN

### 5.1 at initial (effective prestress after short term losses apply to concrete slab)

\*\*\*\*\* Beam Stresses - Prestress & External Loads \*\*\*\*\*

Loc.	Initial Prst.		Final Prst.		Final Prst.		LL+I + 0.5 x	
	+ Beam	Top Bot	+ Total DL	Top Bot	+ Total Load	Top Bot	(Final Prst.	+ Total DL)
	(MN/m <sup>2</sup> )	(MN/m <sup>2</sup> )		(MN/m <sup>2</sup> )				
Brg	-1.319	7.600	-1.165	6.934	1.137	4.645	1.719	1.178
H/2	-1.087	7.378	-.911	6.682	2.389	3.399	2.844	.059
.10L	-.524	6.810	-.295	6.069	5.224	.579	5.372	-2.456
.20L	.094	6.195	.375	5.402	7.838	-2.821	7.651	-4.722
.30L	.536	5.755	.847	4.933	9.121	-3.297	8.697	-5.763
.40L	.801	5.492	1.120	4.661	9.247	-3.423	8.687	-5.753
.50L	.889	5.404	1.194	4.587	8.874	-3.051	8.276	-5.345

Result :

- Max tension -1,319 MPa less than allowable tension at service 0.25 sqrt (fc) = -1.37MPa
- Max compression 5,404 MPa less than allowable compression at initial 0.5 fci = 16.8MPa

### 5.2 at construction (effective prestress after long term losses)

\*\*\*\*\* Beam Stresses - Prestress & External Loads \*\*\*\*\*

Loc.	Initial Prst.		Final Prst.		Final Prst.		LL+I + 0.5 x	
	+ Beam	Top Bot	+ Total DL	Top Bot	+ Total Load	Top Bot	(Final Prst.	+ Total DL)
	(MN/m <sup>2</sup> )	(MN/m <sup>2</sup> )		(MN/m <sup>2</sup> )				
Brg	-1.319	7.600	-1.165	6.934	1.137	4.645	1.719	1.178
H/2	-1.087	7.378	-.911	6.682	2.389	3.399	2.844	.059
.10L	-.524	6.810	-.295	6.069	5.224	.579	5.372	-2.456
.20L	.094	6.195	.375	5.402	7.838	-2.821	7.651	-4.722
.30L	.536	5.755	.847	4.933	9.121	-3.297	8.697	-5.763
.40L	.801	5.492	1.120	4.661	9.247	-3.423	8.687	-5.753
.50L	.889	5.404	1.194	4.587	8.874	-3.051	8.276	-5.345

Result :

- Max tension -1,165 MPa less than allowable tension at service 0.25 sqrt (fc) = -1.37MPa
- Max compression 4,587 MPa less than allowable compression at initial 0.45 fc = 22.5MPa

### 5.3 at service (effective prestress after long term losses apply to concrete slab with SDL + LL + Impact)

\*\*\*\*\* Beam Stresses - Prestress & External Loads \*\*\*\*\*

Loc.	Initial Prst.		Final Prst.		Final Prst.		LL+I + 0.5 x	
	+ Beam	Top Bot	+ Total DL	Top Bot	+ Total Load	Top Bot	(Final Prst.	+ Total DL)
	(MN/m <sup>2</sup> )	(MN/m <sup>2</sup> )		(MN/m <sup>2</sup> )				
Brg	-1.319	7.600	-1.165	6.934	1.137	4.645	1.719	1.178
H/2	-1.087	7.378	-.911	6.682	2.389	3.399	2.844	.059
.10L	-.524	6.810	-.295	6.069	5.224	.579	5.372	-2.456
.20L	.094	6.195	.375	5.402	7.838	-2.821	7.651	-4.722
.30L	.536	5.755	.847	4.933	9.121	-3.297	8.697	-5.763
.40L	.801	5.492	1.120	4.661	9.247	-3.423	8.687	-5.753
.50L	.889	5.404	1.194	4.587	8.874	-3.051	8.276	-5.345

Result :

- Max tension 0,579 MPa less than allowable tension at service 0.5 sqrt (fc) = -3.54MPa
- Max compression 8,874 MPa less than allowable compression at service 0.45 fc = 22.5MPa

#### 5.4 Deflection

\*\*\*\*\* Camber at Centerline (m) \*\*\*\*\*

	Prestress Camber	Beam DL Deflection	Net Camber
Initial	.00476	-.00196	.00280 (- 3. mm)
Erection	.00857	-.00362	.00494 (- 5. mm)
Final	.01166	-.00529	.00637 (- 6. mm)

(+ Upward Deflection; - Downward Deflection)

\*\*\*\*\* Dead Load Deflections (m) \*\*\*\*\*

Erection:	Quarter Point	Centerline
Slab	.00000	-.00000 (- 0. mm)
Other Non-composite	-.00001	-.00001 (- 0. mm)
Composite	-.00011	-.00013 (- 0. mm)
Total	-.00012	-.00014 (- 0. mm)

Final:	Quarter Point	Centerline
Slab	.00000	-.00000 (- 0. mm)
Other Non-composite	-.00002	-.00002 (- 0. mm)
Composite	-.00034	-.00038 (- 0. mm)
Total	-.00036	-.00041 (- 0. mm)

(Maximum Live Load Deflection = -.00509 m)

#### Result:

- Chamber at service load DL = 6 mm - 1 mm = 5 mm
- Deflection due to LL+I = -4,13 mm (edge span) less than allowable deflection L/800 = -8,25mm
- Deflection at service load = DL + LL\*I = 0,87mm

#### VI. STRENGTH DESIGN

##### 6.1 Positive flexural moment

###### Reduction coefficient :

$$\begin{aligned} \text{Flexural Moment Koef.} &= 0,9 \\ \text{Shear koef.} &= 0,7 \end{aligned}$$

###### Load Factor :

$$\text{Moment Ultimate} = 1.2 \text{ DL} + 1.3 \text{ Joint CIP} + 2 \text{ Asphalt} + 1.8 \text{ LL*I}$$

###### Beam Self Weight Moment

$$\begin{aligned} M &= 1/8 \times q \times L^2 \\ &= 1/8 \times 21,4 \times 6,5 \times 6,5 \\ &= 120,77 \text{ Kn-m} \end{aligned}$$

###### Non Composite Dead Load Moment

$$\begin{aligned} M &= 1/8 \times q \times L^2 \\ &= 1/8 \times 0,13 \times 6,5 \times 6,5 \\ &= 0,69 \text{ Kn-m} \end{aligned}$$

###### Composite Dead Load Moment

$$\begin{aligned} M &= 1/8 \times q \times L^2 \\ &= 1/8 \times 2,7 \times 6,5 \times 6,5 \\ &= 15,44 \text{ Kn-m} \end{aligned}$$

###### Manual Calculation - middle span :

$$\text{Loading Factor Mu} = 1.2 \text{ DL} + 1.3 \text{ Joint CIP} + 2 \text{ Asphalt} + 1.8 \text{ LL*I}$$

Beam Self Weight Moment	=	120,77 Kn-m x	1,2 =	144,9187 Kn-m
Non Composite Dead Load Moment	=	0,69 Kn-m x	1,3 =	0,896346 Kn-m
Composite Dead Load Moment	=	15,44 Kn-m x	2 =	30,879 Kn-m
Live Load Moment	=	365,62 Kn-m x	1,8 =	658,116 Kn-m
<b>Ultimate Moment</b>	<b>=</b>			<b>834,8 Kn-m</b>

###### Result :

$$\begin{aligned} \text{Momen Capacity} (\varphi M_n) &= 841 \text{ Kn-m} \\ \text{Momen Ultimate} &= 834,8 \text{ Kn-m} \\ 1.2 \text{ Momen Crack} &= 758 \text{ Kn-m} \end{aligned}$$

\*\*\*\*\* Ultimate Moment (kN-m) \*\*\*\*\*

Provided	Required (Load)	Required (Cracking)
$\Phi \times M_n = 841.$	$M_u = 835.$	$1.2 \times M_{cr} = 758.$

$$\varphi M_n / M_u = 1,007 > 1$$

$$\varphi M_n / 1,2 M_{cr} = 1,1095 > 1$$

Resume : Ultimate Capacity Requirement Achieve

Resume : Cracking Capacity Requirement Achieve

## 6.2 Negative flexural moment

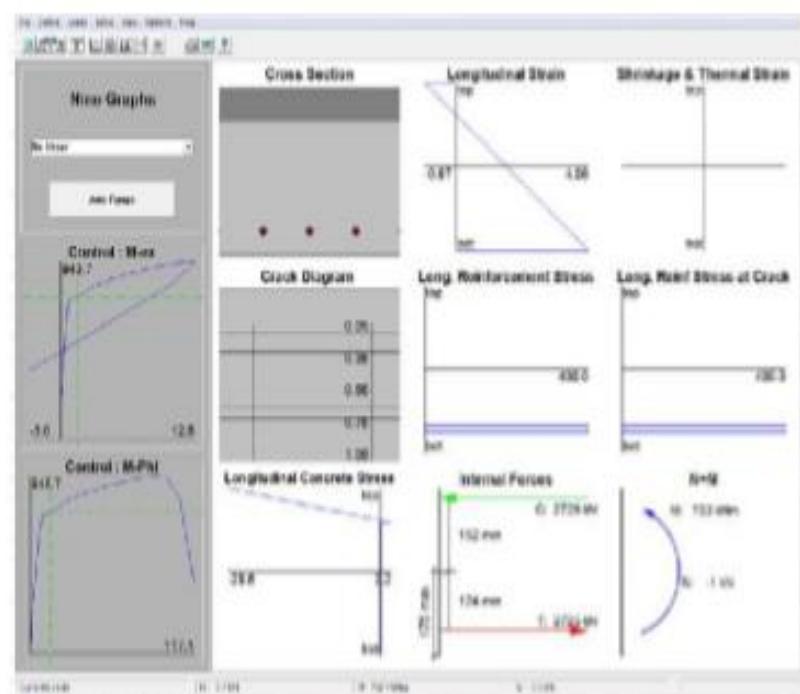
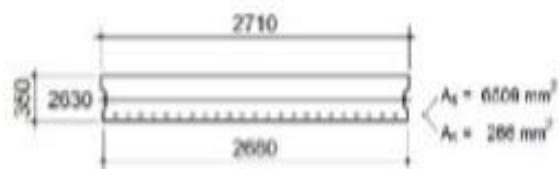
### a. Moment Capacity at pier

Rebar: 11 D19+12 D19+2D13

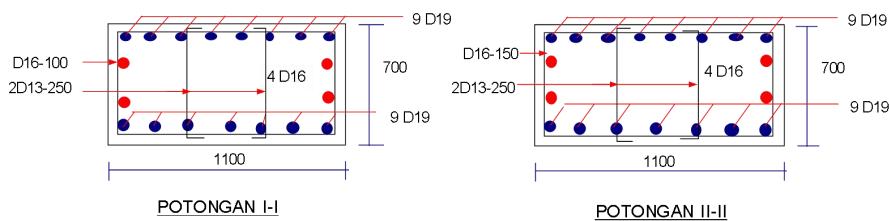
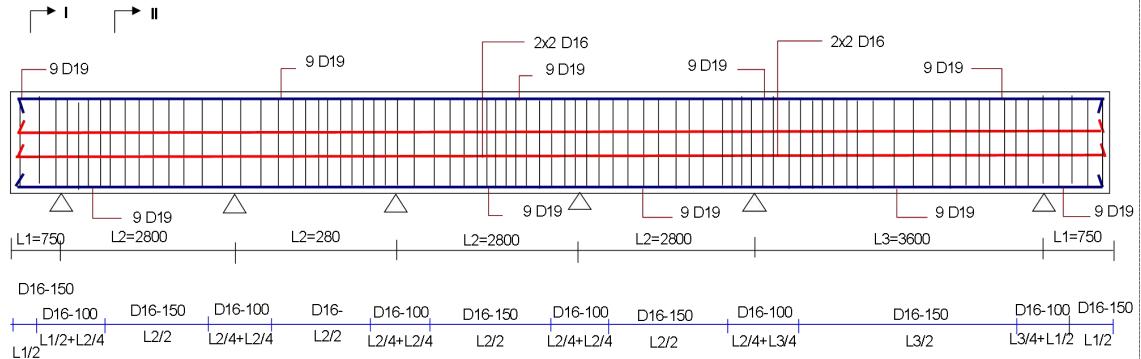
$$M_n = 753$$

$$\varphi = 0,9$$

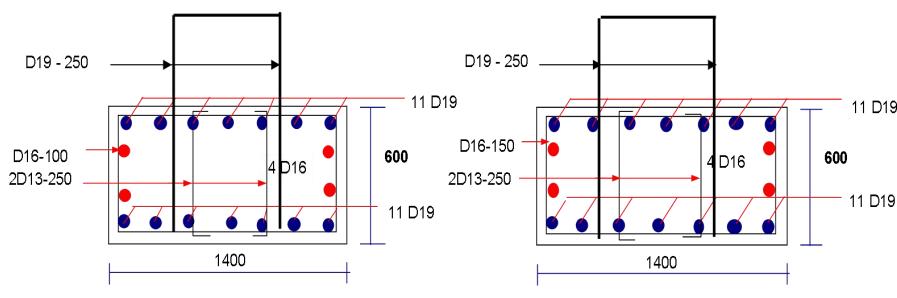
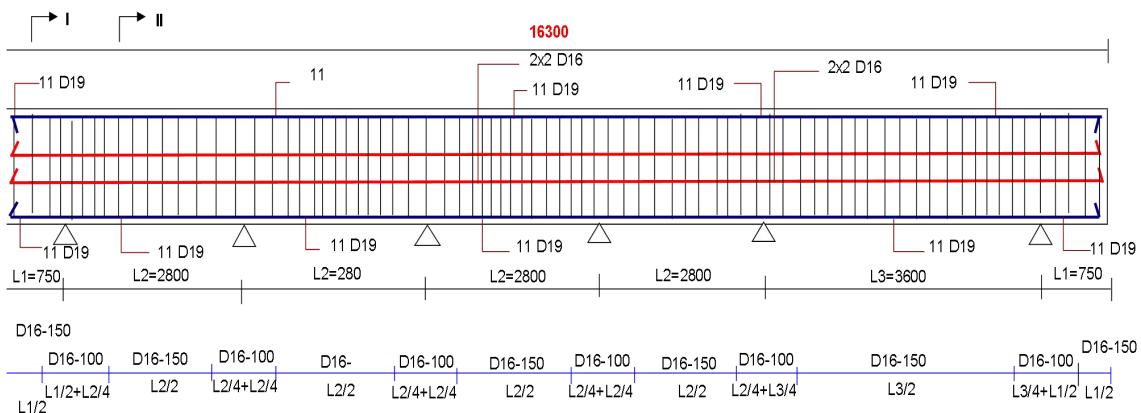
$$\varphi M_n = 677,7 \text{ KN-m}$$



## **1. PIERHEAD TEPI**



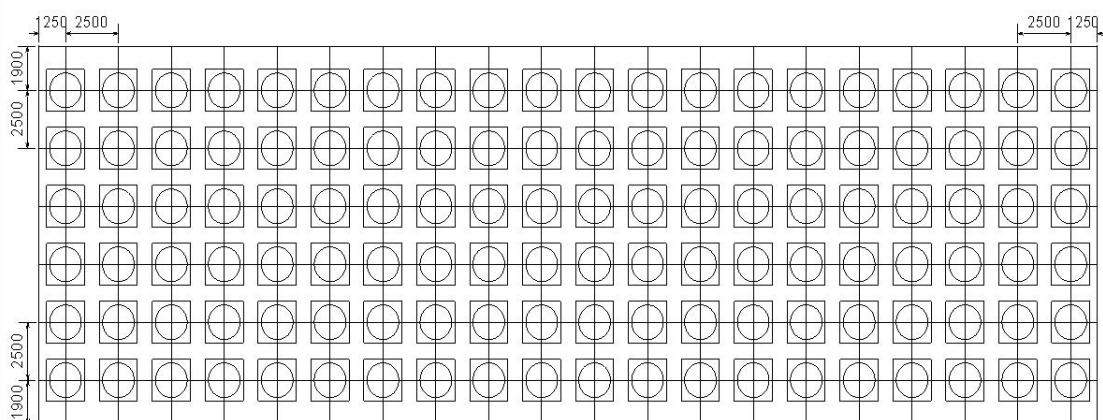
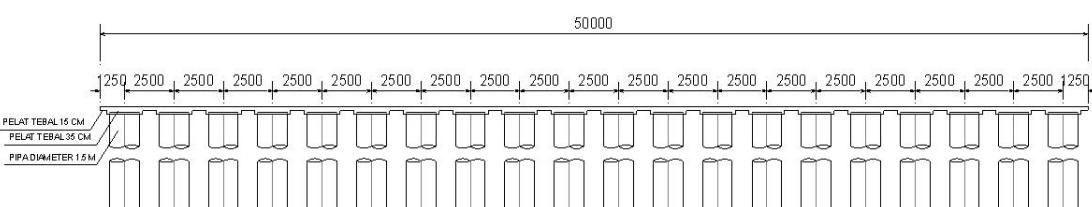
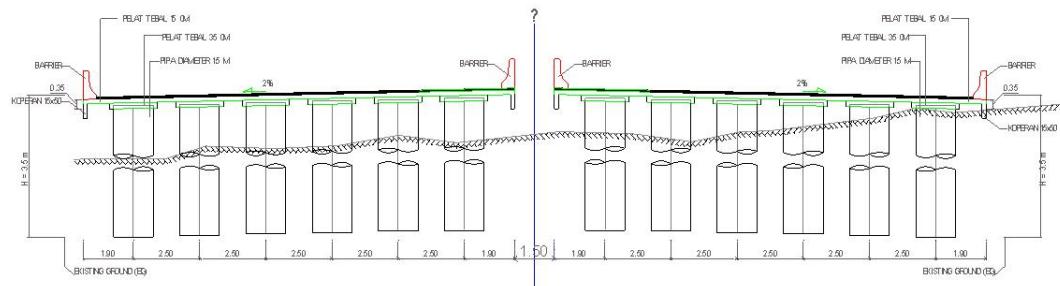
## 2. PIERHEAD TENGAH



**PERHITUNGAN PEMBEBANAN PELAT CAKAR AYAM**

**Cakar Ayam Bentang 50 x 16,3 m<sup>2</sup>**

**Jarak antar Pipa 2.5 meter**



**1.1. PEMBEBANAN PELAT CAKAR AYAM :**

**PERHITUNGAN BERAT STRUKTUR ATAS**

- Berat beton	: Wc	=	2.50	t/m'
- Berat tanah	: Wt	=	1.80	t/m'
- Berat aspal	: Was	=	2.20	t/m'
- Berat air hujan	: Ww	=	1.00	t/m'
- Tebal aspal	: taki	=	0.06	m
- Tebal air hujan	: twki	=	0.03	m
- Tebal air hujan	: twki	=	0.03	m
- Tebal timbunan	:	=	3	m

### A. Beban Mati

1. Berat Sendiri Slab : By program

2. Berat Aspal + Air

- Berat Aspal	: taki x Was =	0.13200	t/m <sup>2</sup>
- Berat Timbunan	: taki x Was =	5.40000	t/m <sup>2</sup>
- Berat Air	: twki x Ww =	0.03000	t/m <sup>2</sup>
Berat total	:	5.56200	t/m <sup>2</sup>

3. Berat tambahan

- Berat barrier	=	0.776	t/m
		0.776	t/m

### B. Beban Hidup

Beban merata "D" : Q1 = 0.80 t/m

Beban Garis "D" : Pl = 4.40 ton

Beban terpusat "T" = 12.5 ton

Faktor Dinamis = 0.40

Koefisien Kejut = 1.40

Beban Merata "D" = Q1 . 1 = 0.8 Ton

Beban "KEL" "P" = P x K = 6.2 Ton

Beban hidup (LL) = 7.0 Ton

#### Beban Hidup Slab bentang 1 (1.9 X 1.25):

Luas Slab = 2.375 m<sup>2</sup>

Qs1 = 2.931 Ton/m<sup>2</sup> 2930.52632 Kg/m<sup>2</sup>

#### Beban Hidup Slab bentang 3 (2.5 X 2.5):

Luas Slab = 6.25 m<sup>2</sup>

Qs3 = 1.114 Ton/m<sup>2</sup> 1113.6 Kg/m<sup>2</sup>

#### Beban Hidup Slab bentang 2 (1.9 X 2.5):

Luas Slab = 4.75 m<sup>2</sup>

Qs2 = 1.465 Ton/m<sup>2</sup> 1465.26316 Kg/m<sup>2</sup>

#### Beban Hidup Slab bentang 4 (1.25 X 2.5):

Luas Slab = 3.13 m<sup>2</sup>

Qs4 = 2.227 Ton/m<sup>2</sup> 2227.2 Kg/m<sup>2</sup>

### C. Beban Spring pada Tiang Pancang

Berdasarkan Data BH VI : dimana : Cu adalah Shear Undrained

Kh = 67 . Cu/d nh adalah Typical Values

z adalah kedalaman komulatif per 1 m

d adalah diameter tiang pancang

Kh = nh . z/d Softer soil

nh = 1 lb/in<sup>3</sup> = 27679.9 kg/m<sup>3</sup>

0.0002768 N/mm<sup>3</sup>

d = 0.6 m

Kedalaman (z)	Cu Ton/m <sup>2</sup>	Kh Kg/m <sup>3</sup>	Spring kg/m
0 m	*	0.00	0.00
1 m	*	46133.17	27679.90
2 m	*	92266.33	55359.80
3 m	*	138399.50	83039.70
4 m	*	184532.67	110719.60

### Penulangan Pelat (15 cm)

Keterangan	Nilai	Satuan
Tipe pelat	Plat 15 cm	-
f'c	30	Mpa
f <sub>y</sub>	400	Mpa
B1	0.85	-
b	1000	mm
Decking	4	mm
Tebal pelat (h)	150	mm
Panjang (Ly)	1630	cm
Pendek (Lx)	500	cm

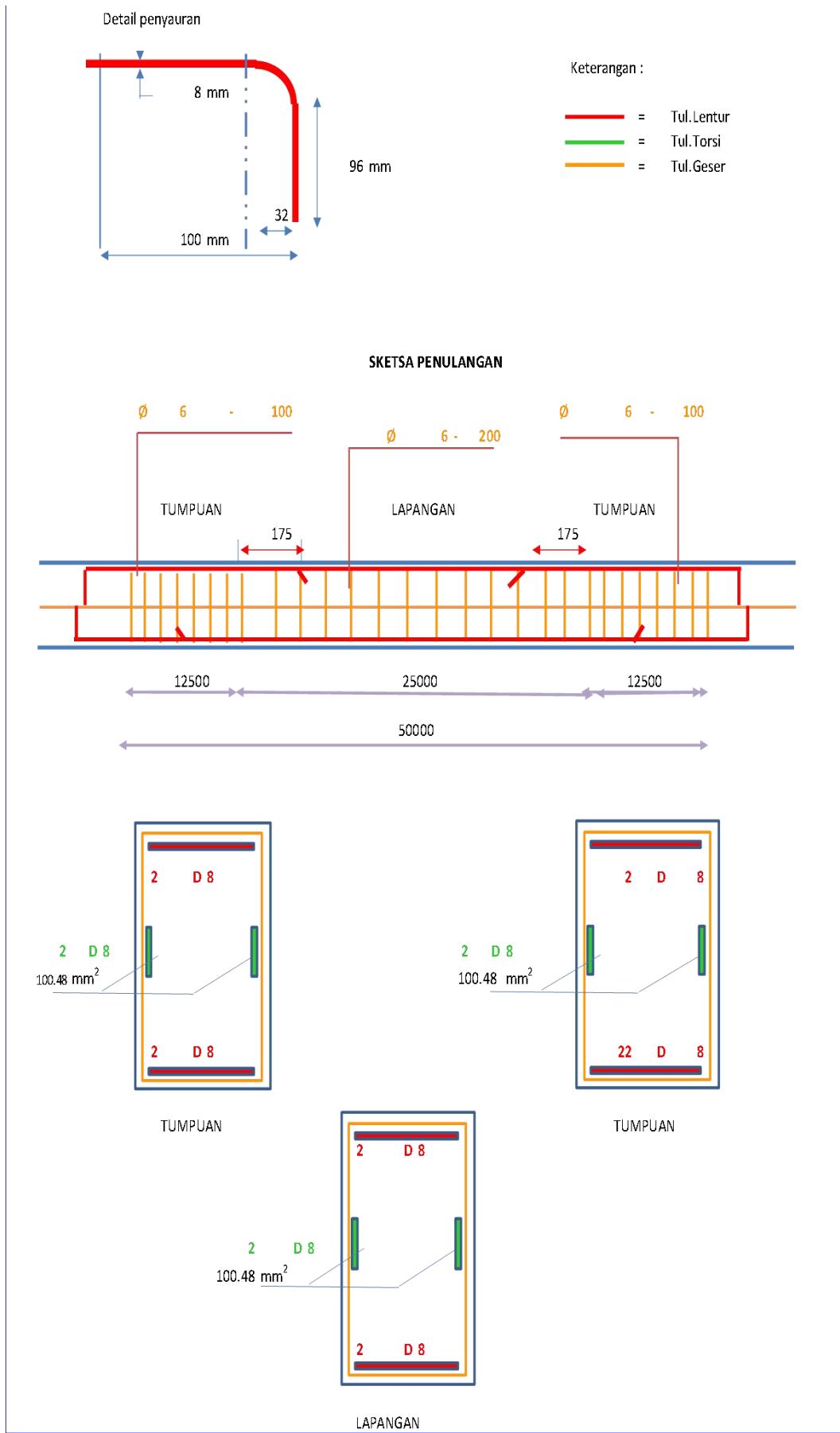
Keterangan	Rumus	Nilai	Satuan	Syarat & Kesimpulan
Rasio	Ly/Lx	3.26	-	<b>One way slab</b>
Ø tulangan lentur		13	mm	
P <sub>min</sub>	1.4/f <sub>y</sub>	0.0035	-	
P <sub>balance</sub>	$\frac{0.85 \cdot f'c' \cdot \beta}{f_y} \cdot \left[ \frac{600}{600 + f_y} \right]$	0.0325125	-	SNI 03-2847-2013 Pasal B.8.4.2
P <sub>max</sub>	0.75 · P <sub>b</sub>	0.024384375	-	
d <sub>x</sub>	tp-decking-Øtul	139.5	mm	
d <sub>y</sub>	tp-decking-Øtul-0.5Øtul	126.5	mm	
m	f <sub>y</sub> /0.85f'c	15.68627451	-	

#### Perhitungan Penulangan

<b>Tumpuan x</b>			
M <sub>tx</sub>		55276500	Nmm
M <sub>n</sub>	M <sub>u</sub> / $\phi$	61418333.33	-
R <sub>n</sub>	M <sub>n</sub> /bd <sup>2</sup>	3.156091691	-
P perlu	$\frac{1}{m} \left[ 1 - \sqrt{1 - \frac{2mRn}{fy}} \right]$	0.008450287	-
As perlu	P.b.d	1178.815017	mm <sup>2</sup>
Pakai Tulangan		13	mm
S <sub>max</sub>	2. h	300	mm
S	(0.25 $\pi$ Ø <sup>2</sup> b)/As	112.5409823	mm
S pakai		100	mm
As pakai	0.25 $\pi$ Ø <sup>2</sup> b)/(S pakai)	1326.65	mm <sup>2</sup>
Maka menggunakan :			
Tulangan Ø		13	mm
Jarak		100	mm
<b>Lapangan x</b>			
M <sub>lx</sub>		50901900	Nmm
M <sub>n</sub>	M <sub>u</sub> / $\phi$	56557666.67	Nmm
R <sub>n</sub>	M <sub>n</sub> /bd <sup>2</sup>	2.906317579	-
P perlu	$\frac{1}{m} \left[ 1 - \sqrt{1 - \frac{2mRn}{fy}} \right]$	0.007735058	-
As perlu	P.b.d	1079.040536	mm <sup>2</sup>
Pakai Tulangan		13	mm
S <sub>max</sub>	2. h	300	mm
S	(0.25 $\pi$ Ø <sup>2</sup> b)/As	122.9471884	mm
S pakai		100	mm
As pakai	0.25 $\pi$ Ø <sup>2</sup> b)/(S pakai)	1326.65	mm <sup>2</sup>
Maka menggunakan :			
Tulangan Ø		13	mm
Jarak		100	mm
<b>Susut Atas</b>			
M <sub>lx</sub>		44804600	Nmm

Mn	Mu/o	49782888.89	Nmm	$\rho_{min} < \rho_{perlu} < \rho_{max}$ $0.0035 < 0.0083 < 0.0244$  Maka dipakai p min = 0.00180
Rn	Mn/bd <sup>2</sup>	3.110993072	-	
P perlu	$\frac{1}{m} \left[ 1 - \sqrt{1 - \frac{2mRn}{fy}} \right]$	0.008320464	-	
As perlu	P.b.d	227.7	mm <sup>2</sup>	
Pakai Tulangan		8	mm	
Smax	2. h	300	mm	
S	(0.25 π Ø^2 b)/As	220.6411946	mm	
S pakai		200	mm	
As pakai	0.25 π Ø^2 b)/(S pakai)	251.2	mm <sup>2</sup>	
Maka menggunakan :				
Tulangan Ø		8	mm	Syarat spasi antar tulangan Smax > S 300 > 220.64 <b>OK</b>
Jarak		200	mm	
<b>Susut Bawah</b>				
M <sub>I</sub> x		36194600	Nmm	$\rho_{min} < \rho_{perlu} < \rho_{max}$ $0.0035 < 0.0066 < 0.0244$  Maka dipakai p min = 0.0018
Mn	Mu/o	40216222.22	Nmm	
Rn	Mn/bd <sup>2</sup>	2.513160476	-	
P perlu	$\frac{1}{m} \left[ 1 - \sqrt{1 - \frac{2mRn}{fy}} \right]$	0.00662739	-	
As perlu	P.b.d	227.7	mm <sup>2</sup>	
Pakai Tulangan		8	mm	
Smax	2. h	300	mm	
S	(0.25 π Ø^2 b)/As	220.6411946	mm	
S pakai		200	mm	
As pakai	0.25 π Ø^2 b)/(S pakai)	251.2	mm <sup>2</sup>	
Maka menggunakan :				Syarat spasi antar tulangan Smax > S 300 > 220.64 <b>OK</b>
Tulangan Ø		8	mm	
Jarak		200	mm	





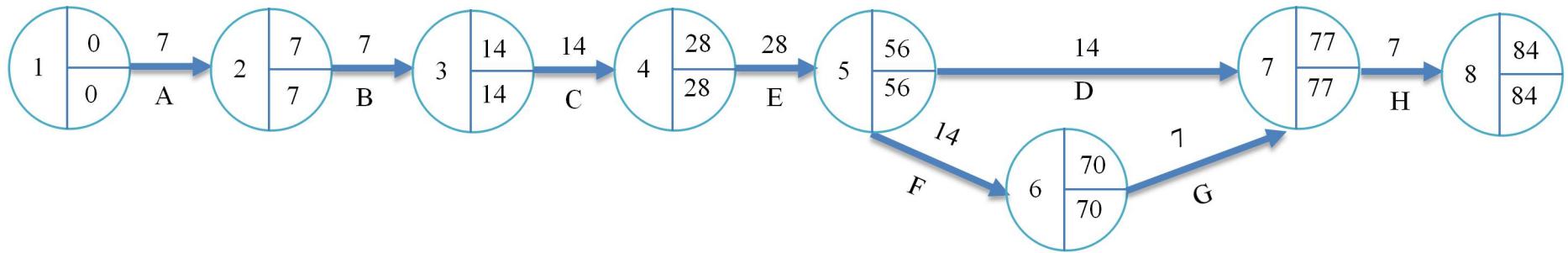
**DAFTAR AKTIFITAS PEKERJAAN PILED SLAB:**

<b>JENIS PEKERJAAN</b>	<b>NOMOR AKTIFITAS</b>	<b>DURASI (HARI)</b>
UMUM (A)	1	7
PEBERSIHAN TEMPAT KERJA (B)	2	7
PEKERJAAN TANAH ( C )	3	14
DRAINASE (D)	4	14
STRUKTUR ( E )	5	28
PEKERJAAN ASPAL (F)	6	14
PENCAHAYAAN DAN LAMPU LALU LINTAS (G)	7	7
PEKERJAAN LAIN-LAIN (H)	8	7

**DAFTAR AKTIFITAS PEKERJAAN CAKAR AYAM:**

<b>JENIS PEKERJAAN</b>	<b>NOMOR AKTIFITAS</b>	<b>DURASI (HARI)</b>
UMUM (A)	1	7
PEBERSIHAN TEMPAT KERJA (B)	2	7
PEKERJAAN TANAH ( C )	3	30
DRAINASE (D)	4	14
STRUKTUR ( E )	5	40
PEKERJAAN ASPAL (F)	6	14
PENCAHAYAAN DAN LAMPU LALU LINTAS (G)	7	7
PEKERJAAN LAIN-LAIN (H)	8	7

## CPM PEKERJAAN PILED SLAB



## CPM PEKERJAAN CAKAR AYAM

