BAB IV
HASIL DAN PEMBAHASAN

4.1 Pondasi Tiang Pancang

4.1.1 Daya Dukung Tanah

Daya dukung tanah ditentukan berdasarkan uji laboratorium dan uji in situ. Data uji laboratorium didapatkan melalui uji triaxial, direct shear, dan konsolidasi. Sedangkan, data uji lapangan yang tersaji adalah data uji N-SPT. Nilai daya dukung tersebut dipakai untuk menentukan jumlah tiang yang dibutuhkan pada pembangunan gedung rumah sakit gigi dan mulut Universitas Brawijaya.

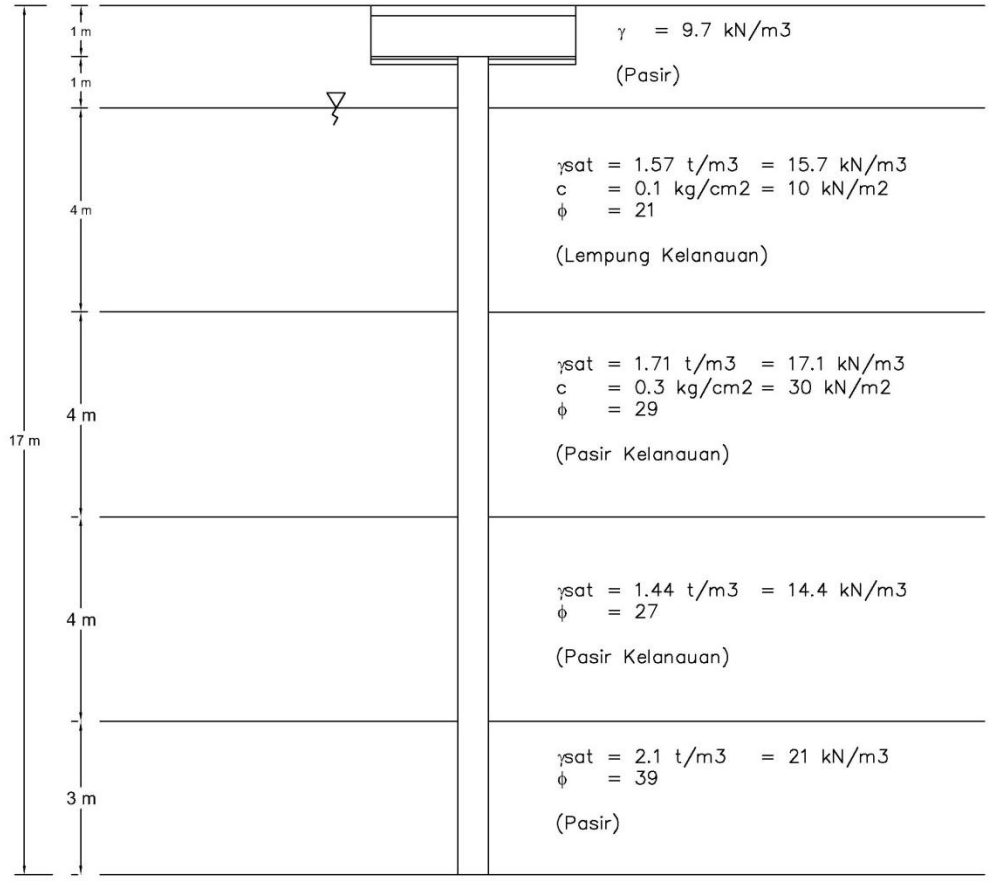
4.1.1.1 Daya Dukung Tanah Menggunakan Data Laboratorium

Data laboratorium dapat digunakan untuk mendapatkan nilai daya dukung ujung dan daya dukung gesek. Pada perhitungan daya dukung menggunakan data laboratorium digunakan beberapa metode.

Data-data yang ditentukan :

H tiang : 16 m

Diameter tiang: 0,4 m



*Gambar 4.*1 Parameter data tanah uji laboratorium

a. Perhitungan Daya Dukung Ujung pada tanah pasir

1. Metode Mayerhof

15D = 15 x 0.4

= 6 m

Tegangan maksimum yang berada pada 9 m

q pada 9 m = (9.7) x 1 + (15.7-9.81) x 4 + (17.1-9.81) x (6-(1+4))

= 40.547 kN/m^2

Pada tanah pasir c = 0

Nq^* = 276 (didapatkan nilai dari tabel 9.5)

q_p = q' x N_q^*

= 40.547 x 276

$= 11191.039 \text{ kN/m}^2$

Syarat $q_p \leq q_t$

Pa $= 100 \text{ kN/m}^2$

$q_t = 0.5 \times Pa \times N_q^* \times \tan\phi'$

$= 0.5 \times 100 \times 276 \times \tan(39)^o$

$= 11175.02 \text{ kN/m}^2$

Karena $q_p \geq q_t$ tidak memenuhi syarat maka yang dipakai adalah q_p terkecil

$q_p = 11175.02 \text{ kN/m}^2$

Menghitung daya dukung ujung

$Q_p = A_p \times q_p$

$= ¼ \times \pi \times D^2 \times q_p$

$= ¼ \times \pi \times 0.4^2 \times 11175.02$

$= 1404.294 \text{ kN}$

2. Metode Vesic

$q' = (9.7) \times 1 + (15.7-9.81) \times 4 + (17.1-9.81) \times 4 + (14.4-9.81) \times 4 + (21-9.81) \times 3$

$= 114.347 \text{ kN/m}^2$

$\overline{\sigma_o}' = (\frac{1+2(1-sin\phi')}{3})q'$

$= (\frac{1+2(1-\sin 39)}{3}) \times 114.347$

$= 66.373 \text{ kN/m}^2$

$\frac{E_s}{P_a} = m$

$E_s = 500 \times 100$

$= 50000 \text{ kN/m}^2$

μ_s $= 0{,}1 + 0{,}3\left(\frac{\phi' - 25}{20}\right)$

$= 0.1 + 0.3\left(\frac{39-25}{20}\right)$

$= 0.310$

Δ $= 0{,}005\left(1 - \frac{\phi' - 25}{20}\right)\frac{q'}{P_a}$

$= 0.005\left(1 - \frac{39-25}{20}\right)\frac{114.347}{100}$

$= 0.0017$

I_r $= \frac{E_s}{2(1+\mu_s)q' tan\phi'}$

$= \frac{50000}{2(1+0.310)114.347\ x\ \tan(39)}$

$= 206.098$

I_{rr} $= \frac{I_r}{1+I_r\Delta}$

$= \frac{206.098}{1+206.098\ x\ 0.0017}$

$= 152.27$

Dari tabel 9.7 apabila Irr 152.27 dan ϕ'= 39 didapatkan $\overline{N_\sigma}^*$= 156.553

Q_p $= A_p$ x $\overline{\sigma o}$' x N_σ^*

$= ¼$ x π x 0.4^2 x 66.373 x 156.553

$= 1305.762$ kN

3. Metode Coyle and Castello

L/D $= 16/0.4$

$= 40$

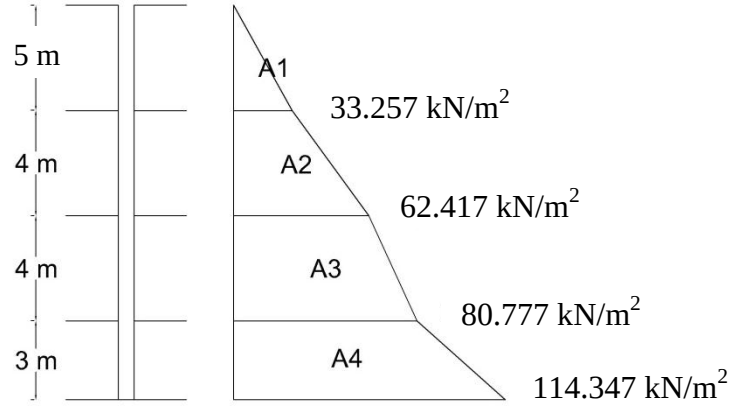
Dari grafik pada gambar 9.15 apabila ϕ' = 39 maka didapat Nq* = 80

Q_p $= A_p$ x N_q^* x q'

= ¼ x π x 0.4^2 x 80 x 114.37

= 1149.544 kN

b. Perhitungan Daya Dukung Selimut pada Tanah Pasir



Gambar 4.2 Diagram tegangan

Tabel 4.1
Perhitungan tegangan dan luas perlapisan tanah

Lapisan 1 **(2.2-6 m)**	Q	=	33.257	kN/m2
	A1	=	83.143	m2
Lapisan 2 **(6-10 m)**	Q	=	62.417	kN/m2
	A2	=	191.349	m2
Lapisan 3 **(10-14 m)**	Q	=	80.777	kN/m2
	A2	=	286.389	m2
Lapisan 4 **(14-17 m)**	Q	=	114.347	kN/m2
	A3	=	292.687	m2

$K_{(1)}$ = 1.4(1-sinϕ')

= 1.4(1-sin(21^o))

= 0.898

$Q_{s(1)}$ = K x $\overline{\sigma_o}$' x tan(0,8ϕ') x p x L

= 0.898 x 33.257 x tan(0.8 x 21)^o x 1.257 x 5

= 56.672 kN

$K_{(2)}$ = 1.4(1-sinϕ')

= 1.4(1-sin(29^o))

= 0.721

$Q_{s(2)}$ = K x $\overline{\sigma_o}$' x tan(0,8ϕ') x p x L

= 0.721 x 62.417 x tan(0.8 x 29)^o x 1.257 x 4

= 96.989 kN

$K_{(3)}$ = 1.4(1-sinϕ')

= 1.4(1-sin(27^o))

= 0.764

$Q_{s(3)}$ = K x $\overline{\sigma_o}$' x tan(0,8ϕ') x p x L

= 0.764 x 80.777 x tan(0.8 x 27)^o x 1.257 x 4

= 122.886 kN

$K_{(4)}$ = 1.4(1-sinϕ')

= 1.4(1-sin(39^o))

= 0.519

$Q_{s(4)}$ = K x $\overline{\sigma_o}$' x tan(0,8ϕ') x p x L

= 0.519 x 114.347 x tan(0.8 x 39)^o x 1.257 x 3

= 135.483 kN

$Q_{s(\text{total pada tanah pasir})}$ = 56.672 + 96.989 + 122.886 + 135.483

= 412.03 kN

- Perhitungan Daya Dukung Selimut Pada Tanah Lempung

1. Metode α

p = π x D

= π x 0.4

= 1.257

Tabel 4.2
Perhitungan daya dukung selimut perlapisan tanah menggunakan metode α

Kedalaman	ΔL (m)	Cu (kN/m2)	α	α.cu.p.L
2 - 6 m	4	10	1	50.265
6 - 10 m	4	30	0.82	123.653
		$Q_{s(1-2)}$	=	173.919 kN

Karena pada tanah lapisan 1 dan 2 terdapat unsur c dan ϕ, maka untuk Q_s menjadi :

$Q_{s(1)}$ = 56.672 + 50.265

= 106.938 kN

$Q_{s(2)}$ = 96.989 + 123.653

= 220.642 kN

$Q_{s(3)}$ = 122.886 kN

$Q_{s(4)}$ = 135.483 kN

Total daya dukung selimut pada metode α adalah

$Q_{s(total)}$ = 106.938 + 220.642 + 122.886 + 135.483

= 585.949 kN

2. Perhitungan Daya Dukung Selimut pada Tanah Lempung Metode λ

$c_u = \frac{C_{u(1)} x \Delta L_{(1)} + C_{u(2)} x \Delta L_{(2)}}{\Sigma \Delta L}$

$= \frac{10\ x\ 5\ +30\ x\ 4}{9}$

= 18.889

$\overline{\sigma_o}' = \frac{A_1 + A_2}{L}$

$= \frac{83.143\ +191.349}{9}$

= 30.5

dari tabel 9.9 interpolasi nilai l dari panjang L = 9 m, didapatkan λ = 0.263

$f_{av} = \lambda(\overline{\sigma_o}' + 2c_u)$

= 0.263 (30.5 + 2 x 18.889)

= 17.957 kN/m^2

$Q_{s(1-2)}$ = p x L x f_{av}

= 1.257 x 9 x 17.957

= 203.087 kN

$Q_{s(total)}$ = $Q_{s(1-2)}$ + $Q_{s(untuk\ tanah\ pasir)}$

= 203.087 + 412.03

= 615.117 kN

3. Perhitungan Daya Dukung Selimut pada Tanah Lempung Metode β

$f_{av(1)}$ = $(1 - \sin\phi'_R)\tan\phi'_R\sigma_o'$

= (1- sin(21)) x tan(21) x $(\frac{0 + 33.257}{2})$

= 4.096

$f_{av(2)}$ = $(1 - \sin\phi'_R)\tan\phi'_R\sigma_o'$

= (1- sin(29)) x tan(29) x $(\frac{33.257+62.417}{2})$

= 13.661

$Q_{s(1-2)}$ = $p\Sigma f\Delta L$

= 1.257 (4.096 x 5 + 13.661 x 4)

= 94.402 kN

$Q_{s(total)}$ = $Q_{s(1-2)}$ + $Q_{s(untuk\ tanah\ pasir)}$

= 94.402 + 412.03

= 506.432 kN

Tabel 4.3
Hasil perhitungan daya dukung menggunakan beberapa metode

Daya Dukung Tiang (kN)			
Metode	Daya Dukung Ujung (kN)	Metode	Daya Dukung Selimut (kN)
Mayerhof	1404.294	α	585.949
Vesic	1305.762	l	615.117
Coyle and Castello	1149.544	β	506.432
Rata-rata	1286.534		569.166

- Daya dukung ultimit tiang

$Q_u = Q_p + Q_s$

$= 1286.534 + 569.166$

$= 1855.700$ kN

- Daya dukung ijin

$Q_a = \frac{Q_u}{2,5}$

$Q_a = \frac{1855.7}{2.5}$

$= 742.28$ kN

4.1.1.2 Daya Dukung Tanah Menggunakan Data SPT

a. Koreksi Nilai N-SPT

$N_{60} = \frac{E_m C_B C_S C_R N}{0,6}$

Dari tabel 2.15 dan tabel 2.16 didapatkan nilai

Nilai E_m hasil dari nilai koreksi energy ratio jenis hammer yang dipakai yaitu : 0.78

Nilai C_B hasil dari nilai koreksi diameter lubang bor yaitu : 1.05

Nilai C_S hasil dari nilai koreksi metode pengambilan sampel yaitu : 1

Nilai C_R hasil dari nilai koreksi panjang batang bor yaitu : 1

$N_{60} = \frac{E_m C_B C_S C_R N}{0,6}$

$= \frac{0.78\ x\ 1.05\ x\ 1\ x\ 1\ x\ N}{0.6}$

$N_{60(1m)} = \frac{0.78\ x\ 1.05\ x\ 1\ x\ 1\ x\ 5}{0.6}$

$= 6.825$

- Metode Aoki-Velloso (1975)

$N_{70} = \frac{0.78\ x\ 1.05\ x\ 1\ x\ 1\ x\ 5}{0.7}$

$= 5.85$

Tabel 4.4
Parameter data tanah menggunakan data SPT Metode Aoki-Velloso (1975)

Kedalaman (m)	Jenis Tanah	N-SPT	N_{70}	N_{gb}	N_{gs}
0	Pasir	0	0		
1		5	5.85		
2					5.46
3	Lempung Kelanauan	4	4.68		
4					
5		6	7.02		
6					
7	Pasir Kelanauan	18	21.06		34.32
8					
9		33	38.61		
10					
11	Pasir Kelanauan	13	15.21		
12					
13 (8D)		12	14.04		
14					
15	Pasir	50	58.5	58.50	
16					
17		50	58.5		
18 (4D)					
19		50	58.5		

b. Perhitungan Daya Dukung Ujung

Data – data yang ditentukan

Kedalaman = 17 m

H tiang dari dasar pile cap = 16 m

Diameter tiang= 0.4 m

N_{gb} = Rata-rata 3 nilai N dibawah dasar tiang

$N_{gb} = \frac{58.5+58.5+58.5}{3}$

= 58.5

K untuk tanah pasir = 1

F_1 = 1.75

$Q_b = (k/F_1) \times N_{gb} \times A_b$

$= (1/1.75) \times 58.5 \times ¼ \times \pi \times 0.4^2$

= 4.201 kN

c. Perhitungan daya dukung selimut

- untuk kedalaman 1 – 6 m (Lempung)

N_{gs} = Rata-rata nilai N disekitar tiang

$N_{gs} = \frac{5.85+4.68+7.02}{3}$

= 5.85 kN

α = 60

k = 0.2

$F_2 = 2F_1 = 3.5$

$Q_{s1} = (\alpha K/F_2) \times N_{gs} \times p \times \Delta L$

$= (60 \times 0.2 / 3.5) \times 5.85 \times \pi \times 0.4 \times 5$

= 126.023 kN

- untuk kedalaman 6 – 17 m (Pasir)

N_{gs} = Rata-rata nilai N disekitar tiang

$N_{gs} = \frac{21.06+38.61+15.21+14.04+58.5+58.5}{6}$

= 34.32 kN

α = 14

Q_{s2} = $(\alpha K/F_2)$ x N_{gs} x p x ΔL

= (14 x 1 / 1.75) x 34.32 x π x 0.4 x 11

= 1897.622 kN

Q_{stotal} = 126.023 + 1897.622

= 2023.65 kN

D. Daya dukung ultimit tiang

Q_u = $Q_b + Q_s$

= 4.201 + 2023.65

= 2027.85 kN

➢ Daya dukung ijin

$Q_{ijin} = \frac{Q_u}{2.5}$

$= \frac{2027.85}{2.5}$

= 811.138 kN

- Metode Meyerhof (1976)

$N_{55} = \frac{0.78\ x\ 1.05\ x\ 1\ x\ 1\ x\ 5}{0.55}$

= 7.45

Tabel 4.5
Parameter data tanah menggunakan data SPT Metode Meyerhof

Kedalaman (m)	Jenis Tanah	N-SPT	N_{60}	N_{gb}	N_{gs}
0	Pasir	0	0		
1		5	7.45		
2					31.602
3	Lempung Kelanauan	4	5.96		
4					
5		6	8.93		
6					
7	Pasir Kelanauan	18	26.8		
8					
9		33	49.14		
10					
11	Pasir Kelanauan	13	19.36		
12					
13		12	17.87		
14 (8D)		36	53.61	69.243	
15	Pasir	50	74.45		
16					
17		50	74.45		
18		50			
19 (4D)		50	74.45		

Data – data yang ditentukan

Kedalaman = 17 m

H tiang dari dasar pile cap = 16 m

Diameter tiang= 0.4 m

b. Perhitungan Daya Dukung Ujung

N_{gb} = Rata-rata nilai N antara 8D diatas dasar tiang sampai 4D dibawah dasar tiang

8D = 17 – (8 x 0.4) = 13.8 m

4D = 17 + (4 x 0.4) = 18.6 m

$N_{gb} = \frac{53.61+74.45+74.45+74.45}{4}$

$= 69.243$

$m = 0.4$

syarat $m \times N_{gb} \leq L/D$

$0.4 \times 69.243 \leq 16/0.4$

$27.697 \leq 40$ (ok!)

$n_b = 0.04$

$Q_b = n_b \times N_{gb} \times A_b$

$= 0.04 \times 69.243 \times ¼ \times \pi \times 0.4^2$

$= 0.348$ kN

c. Perhitungan daya dukung selimut

N_{gs} = Rata-rata nilai N disekitar tiang

$N_{gs} = \frac{7.45+5.96+8.93+26.8+49.14+19.36+17.87+74.45+74.45}{9}$

$= 31.602$ kN

$n_s = 2$

$Q_{s1} = n_s \times N_{gs} \times p \times \Delta L$

$= 2 \times 31.602 \times \pi \times 0.4 \times 16$

$= 1270.785$ kN

d. Daya dukung ultimit tiang

$Q_u = Q_b + Q_s$

$= 0.348 + 1270.785$

$= 1271.133$ kN

➢ Daya dukung ijin

$Q_{ijin} = \frac{Q_u}{2.5}$

$= \frac{1271.133}{2.5}$

= 508.453 kN

- Bazaraa & Kurkur (1986)

Tabel 4.6
Parameter data tanah menggunakan data SPT Bazaraa & Kurkur (1986)

Kedalaman (m)	Jenis Tanah	N-SPT	N_{60}	N_{gb}	N_{gb}
0	Pasir	0	0		
1		5	6.825		
2					28.968
3	Lempung Kelanauan	4	5.46		
4					
5		6	8.19		
6					
7	Pasir Kelanauan	18	24.57		
8					
9		33	45.045		
10					
11	Pasir Kelanauan	13	17.745		
12					
13		12	16.38		
14					
15	Pasir	50	68.25		
16					
17 (1D)		50	68.25	68.25	
18					
19 (3.75D)		50	68.25		

Data – data yang ditentukan

Kedalaman = 17 m

H tiang dari dasar pile cap = 16 m

Diameter tiang= 0.4 m

b. Perhitungan Daya Dukung Ujung

N_{gb} = Rata-rata nilai N antara 1D diatas dasar tiang sampai 3.75D dibawah dasar tiang

1D = 17 – (1 x 0.4) = 16.6 m (tinggi pile cap + tanah timbunan) = 16.6

3.75D = 17 + (3.75 x 0.4) = 18.5 m

$N_{gb} = \frac{68.25+68.25+68.25}{3}$

$= 68.25$

$n_b = 0.06$

$Q_b = n_b \times N_{gb} \times A_b$

$= 0.06 \times 68.25 \times ¼ \times \pi \times 0.4^2$

$= 0.377$ kN

c. Perhitungan daya dukung selimut

N_{gs} = Rata-rata nilai N disekitar tiang

$N_{gs} = \frac{6.825+5.46+8.19+24.57+45.045+17.745+16.38+68.25+68.25}{9}$

$= 28.968$ kN

$n_s = 2$

$Q_{s1} = n_s \times N_{gs} \times p \times \Delta L$

$= 2 \times 28.968 \times \pi \times 0.4 \times 16$

$= 1164.886$ kN

d. Daya dukung ultimit tiang

$Q_u = Q_b + Q_s$

$= 0.377 + 1164.886$

= 1165.263 kN

➢ Daya dukung ijin

Q_{ijin} $= \frac{Q_u}{2.5}$

$= \frac{1165.263}{2.5}$

= 466.105 kN

- Metode Decourt (1995)

Tabel 4.7
Parameter data tanah menggunakan data SPT Metode Decourt (1995)

Kedalaman (m)	Jenis Tanah	N-SPT	N_{60}	N_{gb}	N_{gs}
0	Pasir	0	0		
1		5	6.825		6.825
2					
3	Lempung Kelanauan	4	5.46		
4					
5		6	8.19		
6					
7	Pasir Kelanauan	18	24.57		40.04
8					
9		33	45.045		
10					
11	Pasir Kelanauan	13	17.745		
12					
13		12	16.38		
14 (8D)		36	49.14	63.473	
15	Pasir	50	68.25		
16					
17		50	68.25		
18					
19 (4D)		50	68.25		

c. Perhitungan Daya Dukung Ujung

Data – data yang ditentukan

Kedalaman = 17 m

H tiang dari dasar pile cap = 16 m

Diameter tiang= 0.4 m

N_{gb} = Rata-rata nilai N disekitar dasar tiang

$N_{gb} = \frac{49.14+68.25+68.25+68.25}{4}$

= 63.473

k_b = 0.325

$Q_b = k_b \times N_{gb} \times A_b$

$= 0.325 \times 63.473 \times ¼ \times \pi \times 0.4^2$

= 2.592 kN

c. Perhitungan daya dukung selimut

- untuk kedalaman 1 – 6 m (Lempung)

N_{gs} = Rata-rata nilai N disekitar tiang

$N_{gs} = \frac{6.825 +5.46+8.19}{3}$

= 6.825 kN

α = 1

$Q_{s1} = \alpha (2.8 \times N_{gs} + 10) \times p \times \Delta L$

$= 1 (2.8 \times 6.825 + 10) \pi \times 0.4 \times 5$

= 182.904 kN

- untuk kedalaman 6 – 17 m (Pasir)

N_{gs} = Rata-rata nilai N disekitar tiang

$N_{gs} = \frac{24.57+45.045+17.745+16.38+68.25+68.25}{6}$

= 40.04 kN

α = 0.5

Q_{s1} = α (2.8 x N_{gs} + 10) x p x ΔL

= 0.5 (2.8 x 40.04 + 10) π x 0.4 x 11

= 843.978 kN

Q_{stotal} = 182.904 + 843.978

= 1026.881 kN

D. Daya dukung ultimit tiang

Q_u = Q_b + Q_s

= 2.592 + 1026.881

= 1029.473 kN

- Daya dukung ijin

Q_{ijin} = $\frac{Q_u}{2.5}$

= $\frac{1029.473}{2.5}$

= 411.789 kN

- Shariatmadari et al. (2007)

Tabel 4.8
Parameter data tanah menggunakan data SPT Shariatmadari et al. (2007)

Kedalaman (m)	Jenis Tanah	N-SPT	N_{60}	N_{gb}	N_{gs}
0	Pasir	0	0		
1		5	6.825		28.968
2					
3	Lempung Kelanauan	4	5.46		
4					
5		6	8.19		
6					
7	Pasir Kelanauan	18	24.57		
8					
9		33	45.045		

10					
11	Pasir Kelanauan	13	17.745		
12					
13		12	16.38		
14 (8D)				63.473	
15	Pasir	50	68.25		
16					
17		50	68.25		
18					
19 (4D)		50	68.25		

Data – data yang ditentukan

Kedalaman = 17 m

H tiang dari dasar pile cap = 16 m

Diameter tiang= 0.4 m

b. Perhitungan Daya Dukung Ujung

Ngb = Rata-rata nilai N antara 8D diatas dasar tiang sampai 4D dibawah dasar tiang

8D = 17 – (8 x 0.4) = 13.8 m

4D = 17 + (4 x 0.4) = 18.6 m

$N_{gb} = \frac{49.14 + 68.25 + 68.25 + 68.25}{4}$

= 63.473

$Q_b = 0.385 \times N_{gb} \times A_b$

$= 0.385 \times 63.473 \times ¼ \times \pi \times 0.4^2$

= 3.071 kN

c. Perhitungan daya dukung selimut

N_{gs} = Rata-rata nilai N disekitar tiang

$N_{gs} = \frac{6.825 + 5.46 + 8.19 + 24.57 + 45.045 + 17.745 + 16.38 + 68.25 + 68.25}{9}$

= 28.968 kN

Q_s = 3.65 x N_{gs} x p x ΔL

= 3.65 x 28.968 x π x 0.4 x 16

= 2125.917 kN

d. Daya dukung ultimit tiang

Q_u = Q_b + Q_s

= 3.071 + 2125.917

= 2128.987 kN

➢ Daya dukung ijin

Q_{ijin} = $\frac{Q_u}{2.5}$

= $\frac{2128.987}{2.5}$

= 851.595 kN

- Kesimpulan Perhitungan SPT dari berbagai metode

Panjang Tiang dari dasar pile cap = 16 m

Diameter Tiang = 40 cm = 0.4 m

Tabel 4.9
Kesimpulan perhitungan SPT dari berbagai metode

	Daya Dukung	Selimut (Q_s)	Ujung (Q_b)	Ultimit (Q_u)	Ijin (Q_a)
		(kN)			
Metode	Aoki-Velloso (1975)	2023.645	4.201	2027.846	811.138
	Meyerhof (1976)	1270.785	0.348	1271.133	508.453
	Bazaraa & Kurkur (1986)	1164.886	0.377	1165.263	466.105
	Decourt (1995)	1026.881	2.592	1029.473	411.789
	Shariatmadari et al. (2007)	2125.917	3.071	2128.987	851.595

Rata-rata daya dukung ijin dari perhitungan SPT adalah:

Daya dukung selimut Q_s = 1522.423 kN

Daya dukung ujung Q_b = 2.118 kN

Daya dukung batas Q_u = 1522.423 + 2.118 = 1524.541 kN

Didapatkan daya dukung ijin yaitu :

$Q_a = \frac{1524.541}{2.5}$

= 609.816 kN

4.1.1.3 Perhitungan Tiang Grup

Dalam penyusunan tiang dan dalam menentukan berapa jumlah tiang yang dibutuhkan pada 1 kolom digunakan daya dukung ijin dari perhitungan menggunakan data SPT yaitu Q_a = 609.816 kN. Hal ini disebabkan hasil dari perhitungan daya dukung menggunakan data laboratirium lebih besar dibandingkan dari hasil menggunakan data SPT, yang menyebabkan perhitungan daya dukung ijin menggunakan data SPT memberikan perhitungan yang lebih aman. Penyusunan tiang harus mempertimbangkan jarak ijin tiang. Berikut ini merupakan persamaan yang dipakai dalam menentukan jarak tiang :

- Menentukan jarak tiang

Jarak tiang ke tepi = ≤ 1.25 D

= 0.5 m

Jarak antar tiang (d) = 2.5D - 3D

= 1 – 1.2

Digunakan jarak antar tiang (d) = 1 m

- Efisiensi tiang

Perhitungan efisiensi tiang untuk menentukan beban maksimum kelompok tiang terdapat dua cara, untuk perhitungan jumlah tiang genap perhitungan efisiensi tiang menggunakan metode *Converse-Labarre formula* (1980). Sedangkan, untuk perhitungan

jumlah tiang ganjil perhitungan efisiensi tiang menggunakan metode *Ramlah and Chickanagappa formula* (1981).

- 2 Tiang

a. Jumlah tiang (n) $= \frac{V\ (P35L)}{Qijin}$

$= 609.816$

$= 1.7 \approx 2$ tiang

$n_1 = 2$

$n_2 = 1$

$L_g = (n_1 - 1)d + 2(\frac{D}{2})$

$= (1 - 1)\ x\ 1 + 2(\frac{0.4}{2})$

$= 1.4$

$B_g = (n_2 - 1)d + 2(\frac{D}{2})$

$= (2 - 1)\ x\ 1 + 2(\frac{0.4}{2})$

$= 0.4$

Syarat $L_g \geq B_g$ (ok!)

Efisiensi kelompok tiang

$\theta = \tan^{-1}(\frac{D}{d})$

$= \tan^{-1}\left(\frac{0.4}{1}\right)$

$= 21.801$

$\eta = 1 - \theta(\frac{(n_1-1)n_2+(n_2-1)n_1}{90.n_2.n_1})$

$= 1 - 21.801\ (\frac{(2-1)x1+(1-1)x2}{90x1x2}$

= 0.88

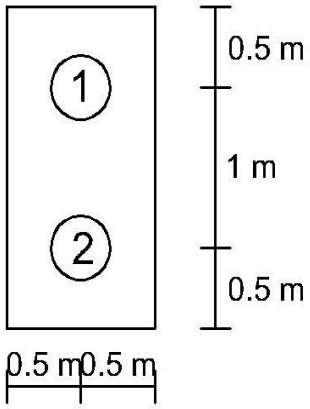
Q_{ag} = η.k.Q_u

= 0.88 x 2 x 609.816

= 1071.915 kN

Q_{ag} > V

1071.915 kN > 1059.28 kN (ok!)



Gambar 4.3 Pile cap 2 tiang

- **Menentukan P maks 1 tiang**

$$P = \frac{V}{n} \pm \frac{My.xi}{\sum x^2} \pm \frac{Mx.yi}{\sum y^2}$$

V = 1059.28 kN

Mx = -0.858 kNm

My = -20.822 kNm

Jumlah Tiang = 2

n1;n2 = 2;1

Tabel 4.10
P_{maks} 1 tiang pile cap tipe 2

Titik	X (m)	Y (m)	X^2	Y^2	Pi (kN)		Qijin (kN)
1	-0.5	0.5	0.25	0.25	549.604	<	609.816

2	0.5	0.5	0.25	0.25	507.960	<	609.816
Σ			0.5	0.5			

Syarat Pi < Qijin (ok!)

➢ 3 Tiang

a. Jumlah tiang (n) $= \frac{V\ (P05)}{Qijin}$

$= \frac{1551.68}{609.816}$

$= 2.5 \approx 3$ tiang

Tabel 4.11
Faktor reduksi tiang tiap joint pile cap tipe 3

Tipe Joint	Jumlah Tiang (n_p)	Jumlah Tiang yang Berpengaruh (n_{ad})	Faktor Reduksi Tiang	Daya Dukung Ultimit
A	3	2	$1 - \frac{2}{16} = 0.875$	2.625 Q_u

$\eta = \frac{Q_{ag(u)}}{\Sigma Q_u}$

$= \frac{2.625\ Q_u}{3\ Q_u}$

$= 0.875$

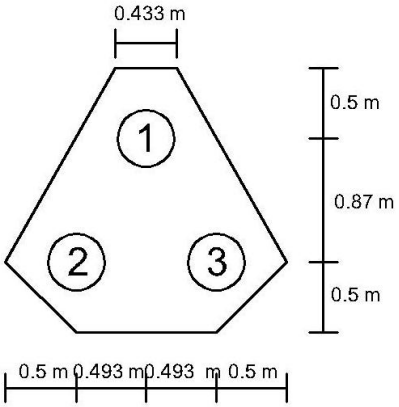
$Q_{ag} = \eta.k.Q_u$

$= 0.75 \times 3 \times 609.816$

$= 1600.767$ kN

$Q_{ag} > V$

1600.767 kN > 1551.68 kN (ok!)



Gambar 4.4 Pile cap 3 tiang

- **Menentukan P maks 1 tiang**

P = $\frac{V}{A} \pm \frac{My.xi}{\sum x^2} \pm \frac{Mx.yi}{\sum y^2}$

V = 1551.68 kN

Mx = -0.596 kNm

My = 1.036 kNm

Jumlah Tiang = 3

Tabel 4.12
P_{maks} 1 tiang pile cap tipe 3

Titik	X (m)	Y (m)	X^2	Y^2	Pi (kN)		Qijin (kN)
1	-0.006	0.132	0.000	0.017	516.823	<	609.816
2	-0.499	-0.303	0.249	0.092	517.087	<	609.816
3	0.499	-0.303	0.249	0.092	519.163	<	609.816
Σ			0.498	0.201			

Syarat Pi < Qijin (ok!)

➢ 4 Tiang

a. Jumlah tiang (n) = $\frac{V\ (P09)}{Qijin}$

= $\frac{1733.38}{609.816}$

= 2.9 ≈ 4 tiang

n_1 = 2

n_2 = 2

L_g = $(n_1 - 1)d + 2(\frac{D}{2})$

= $(2 - 1)$ x 1 + $2(\frac{0.4}{2})$

= 1.4

B_g = $(n_2 - 1)d + 2(\frac{D}{2})$

= $(2 - 1)$ x 1 + $2(\frac{0.4}{2})$

= 1.4

Syarat $L_g \geq B_g$ (ok!)

Efisiensi kelompok tiang

θ = $\tan^{-1}(\frac{D}{d})$

= $\tan^{-1}\left(\frac{0.4}{1}\right)$

= 21.801

η = $1 - \theta(\frac{(n_1-1)n_2+(n_2-1)n_1}{90.n_2.n_1})$

= $1 - 21.801\ (\frac{(2-1)x2+(2-1)x2}{90x2x2}$

= 0.76

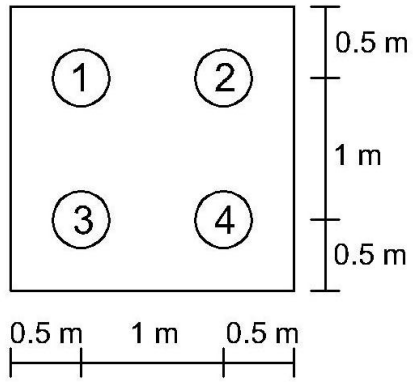
Q_{ag} = $\eta.k.Q_u$

= 0.76 x 4 x 609.816

= 1848.39 kN

$Q_{ag} > V$

1848.39 kN > 1733.38 kN (ok!)



Gambar 4.5 Pile cap 4 tiang

- **Menentukan P maks 1 tiang**

$P = \frac{V}{n} \pm \frac{My.xi}{\sum x^2} \pm \frac{Mx.yi}{\sum y^2}$

V = 1733.38 kN

Mx = 32.557 kNm

My = -5.664 kNm

Jumlah Tiang = 4

n1;n2 = 2;2

Tabel 4.13
P_{maks} 1 tiang pile cap tipe 4

Titik	X (m)	Y (m)	X^2	Y^2	Pi (kN)		Qijin (kN)
1	-0.5	0.5	0.25	0.25	452.455	<	609.816
2	0.5	0.5	0.25	0.25	446.791	<	609.816
3	-0.5	-0.5	0.25	0.25	419.899	<	609.816
4	0.5	-0.5	0.25	0.25	414.235	<	609.816
Σ			1	1			

Syarat Pi < Qijin (ok!)

➢ 5 Tiang

a. Jumlah tiang (n) $= \frac{V\ (P53)}{Qijin}$

$= \frac{2403.74}{609.816}$

$= 3.9 \approx 5$ tiang

Tabel 4.14
Faktor reduksi tiang tiap joint pile cap tipe 5

Tipe Joint	Jumlah Tiang (n_p)	Jumlah Tiang yang Berpengaruh (n_{ad})	Faktor Reduksi Tiang	Daya Dukung Ultimit
A	1	4	$1 - \frac{4}{16} = 0.75$	0.75 Q_u
B	4	3	$1 - \frac{3}{16} = 0.8125$	3.25 Q_u

$\eta = \frac{Q_{ag(u)}}{\Sigma Q_u}$

$= \frac{4\ Q_u}{5\ Q_u}$

$= 0.8$

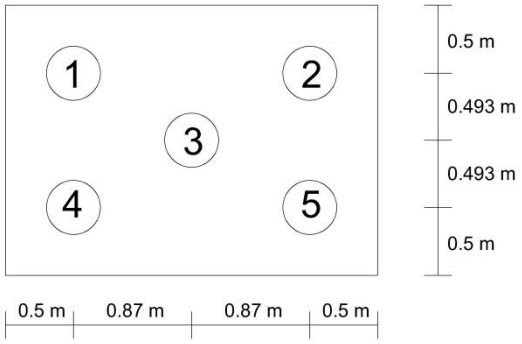
$Q_{ag} = \eta.k.Q_u$

$= 0.8 \times 5 \times 609.816$

$= 2439.265$ kN

$Q_{ag} > V$

2439.265 kN > 2403.74 kN (ok!)



Gambar 4.6 Pile cap 5 tiang

- **Menentukan P maks 1 tiang**

$P = \frac{V}{n} \pm \frac{My.xi}{\sum x^2} \pm \frac{Mx.yi}{\sum y^2}$

V = 2403.74 kN

Mx = -66.724 kNm

My = -2.682 kNm

Jumlah Tiang = 5

Tabel 4.15
P_{maks} 1 tiang pil cap tipe 5

Titik	X (m)	Y (m)	X^2	Y^2	Pi (kN)		Qijin (kN)
1	-0.493	0.87	0.243	0.757	453.083	<	609.816
2	0.493	0.87	0.243	0.757	450.363	<	609.816
3	0	0	0	0	480.748	<	609.816
4	-0.493	-0.493	0.243	0.243	498.560	<	609.816
5	0.493	-0.493	0.243	0.243	495.835	<	609.816
Σ			0.972	2.000			

Syarat Pi < Qijin (ok!)

- 6 Tiang

a. Jumlah tiang (n) $= \frac{V\ (P20)}{Qijin}$

$= \frac{2524.81}{609.816}$

$= 4.1 \approx 6$ tiang

n_1 = 3

n_2 = 2

$L_g = (n_1 - 1)d + 2(\frac{D}{2})$

$= (3 - 1) \times 1 + 2(\frac{0.4}{2})$

= 2.4

$B_g = (n_2 - 1)d + 2(\frac{D}{2})$

$= (2 - 1) \times 1 + 2(\frac{0.4}{2})$

= 1.4

Syarat $L_g \geq B_g$ (ok!)

Efisiensi kelompok tiang

$\theta = \tan^{-1}(\frac{D}{d})$

$= \tan^{-1}\left(\frac{0.4}{1}\right)$

$= 21.801$

$\eta = 1 - \theta(\frac{(n_1-1)n_2+(n_2-1)n_1}{90.n_2.n_1})$

$= 1 - 21.801\ (\frac{(3-1)x2+(2-1)x3}{90x2x3})$

$= 0.72$

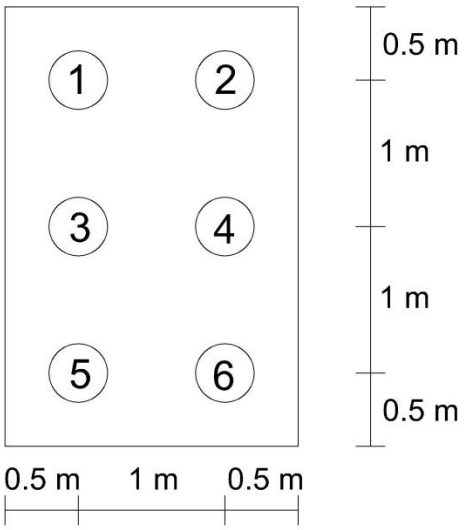
$Q_{ag} = \eta.k.Q_u$

$= 0.72 \times 6 \times 609.816$

$= 2624.87$ kN

$Q_{ag} > V$

2624.87 kN > 2524.81 kN (ok!)



Gambar 4.7 Pile cap 6 tiang

- **Menentukan P maks 1 tiang**

$P = \frac{V}{n} \pm \frac{My.xi}{\sum x^2} \pm \frac{Mx.yi}{\sum y^2}$

V = 2524.81 kN

Mx = 1.542 kNm

My = 14.329 kNm

Jumlah Tiang = 6

n1;n2 = 3;2

Tabel 4.16
P_{maks} 1 tiang pile cap tipe 6

Titik	X (m)	Y (m)	x2	y2	Pi (kN)		Qijin (kN)
1	-0.5	1	0.25	1	416.411	<	605.783
2	0.5	1	0.25	1	425.963	<	605.783
3	-0.5	0	0.25	0	416.025	<	605.783
4	0.5	0	0.25	0	425.578	<	605.783
5	-0.5	-1	0.25	1	415.640	<	605.783
6	0.5	-1	0.25	1	425.192	<	605.783
Σ			1.5	4			

Syarat Pi < Qijin (ok!)

- 7 Tiang

a. Jumlah tiang (n) $= \frac{V\ (P11)}{Qijin}$

$= \frac{3347.71}{609.816}$

$= 5.5 \approx 7$ tiang

Tabel 4.17
Faktor reduksi tiang tiap joint pile cap tipe 7

Tipe Joint	Jumlah Tiang (n_p)	Jumlah Tiang yang Berpengaruh (n_{ad})	Faktor Reduksi Tiang	Daya Dukung Ultimit
A	1	6	$1 - \frac{6}{16} = 0.75$	0.625 Q_u
B	6	3	$1 - \frac{3}{16} = 0.8125$	4.875 Q_u

$\eta = \frac{Q_{ag(u)}}{\Sigma Q_u}$

$= \frac{5.5\ Q_u}{7\ Q_u}$

$= 0.76$

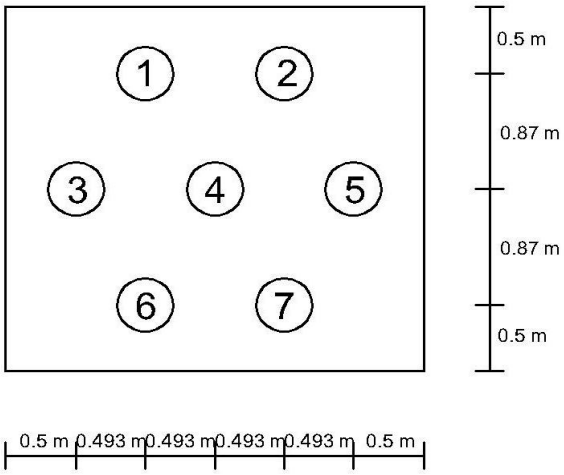
$Q_{ag} = \eta.k.Q_u$

$= 0.76 \times 7 \times 609.816$

$= 3353.989$ kN

$Q_{ag} > V$

3353.989 kN > 3347.71 kN (ok!)



Gambar 4.8 Pile cap 7 tiang

- **Menentukan P maks 1 tiang**

$P = \frac{V}{A} \pm \frac{My.xi}{\sum x^2} \pm \frac{Mx.yi}{\sum y^2}$

V = 3347.71 kN

Mx = 5.277 kNm

My = 4.226 kNm

Jumlah Tiang = 7

Tabel 4.18
P_{maks} 1 tiang pile cap tipe 7

Titik	X (m)	Y (m)	X^2	Y^2	Pi (kN)		Qijin (kN)
1	-0.493	0.87	0.243	0.757	477.617	<	609.816
2	0.493	0.87	0.243	0.757	481.904	<	609.816
3	0	0	0	0	478.244	<	609.816
4	0	0	0	0	478.244	<	609.816
5	0	0	0	0	478.244	<	609.816
6	-0.493	-0.87	0.243	0.757	474.585	<	609.816
7	0.493	-0.87	0.243	0.757	478.872	<	609.816
Σ			0.972	3.028			

Syarat Pi < Qijin (ok!)

➢ 10 Tiang

a. Jumlah tiang (n) $= \frac{V\ (P37)}{Qijin}$

$= \frac{3490.57}{609.816}$

$= 5.7 \approx 10$ tiang

Tabel 4.19
Faktor reduksi tiang tiap joint pile cap tipe 10

Tipe Joint	Jumlah Tiang (n_p)	Jumlah Tiang yang Berpengaruh (n_{ad})	Faktor Reduksi Tiang	Daya Dukung Ultimit
A	6	3	$1 - \frac{3}{16} = 0.8125$	4.875 Q_u
B	2	4	$1 - \frac{4}{16} = 0.75$	1.5 Q_u
C	2	6	$1 - \frac{6}{16} = 0.625$	1.25

$\eta = \frac{Q_{ag(u)}}{\Sigma Q_u}$

$= \frac{6.375\ Q_u}{10\ Q_u}$

$= 0.638$

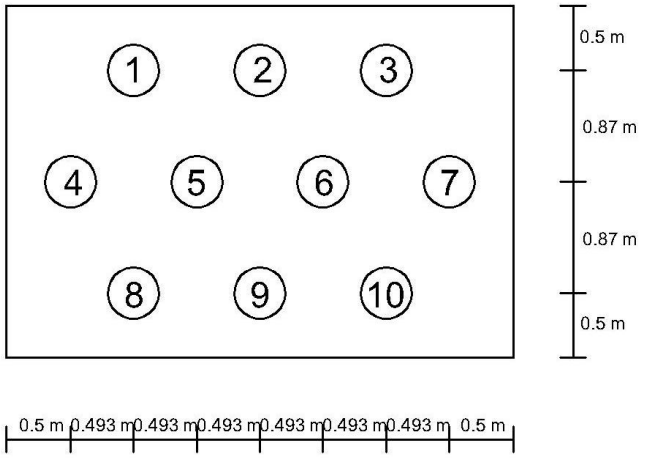
$Q_{ag} = \eta.k.Q_u$

$= 0.638 \times 10 \times 609.816$

$= 3887.578$ kN

$Q_{ag} > V$

3887.578 kN > 3490.57 kN (ok!)



Gambar 4.9 Pile cap 10 tiang

- **Menentukan P maks 1 tiang**

$P = \frac{V}{n} \pm \frac{My.xi}{\sum x^2} \pm \frac{Mx.yi}{\sum y^2}$

V = 3490.57 kN

Mx = -31.674 kNm

My = 1.325 kNm

Jumlah Tiang = 10

Tabel 4.20
P_{maks} 1 tiang pile cap tipe 10

Titik	X (m)	Y (m)	x2	y2	Pi (kN)		Qijin (kN)
1	-0.986	0.87	0.972	0.757	342.653	<	609.816
2	0	0.87	0	0.757	342.989	<	609.816
3	0.986	0.87	0.972	0.757	343.325	<	609.816
4	0	0	0	0	349.057	<	609.816
5	0	0	0	0	349.057	<	609.816
6	0	0	0	0	349.057	<	609.816
7	0	0	0	0	349.057	<	609.816
8	-0.986	-0.87	0.972	0.757	354.790	<	609.816
9	0	-0.87	0	0.757	355.125	<	609.816
10	0.986	-0.87	0.972	0.757	355.461	<	609.816
Σ			3.889	4.541			

Syarat Pi < Qijin (ok!)

Tabel 4.21
Jumlah Kebutuhan Tiang Pancang dengan menggunakan data SPT

Joint	Pu Kolom		Kebutuhan Tiang	
	ton	kN		
K01	16.189	161.89	0.3	1
K02	29.463	294.63	0.5	1
K03	29.426	294.26	0.5	1
K04	16.179	161.79	0.3	1
K05	21.691	216.91	0.4	1
K06	21.47	214.7	0.4	1
K07	12.852	128.52	0.2	1
K08	7.682	76.82	0.1	1
K09	17.884	178.84	0.3	1
K10	11.45	114.5	0.2	1
K11	0.476	4.76	0.0	1
K12	1.586	15.86	0.0	1
K13	10.285	102.85	0.2	1
K14	10.789	107.89	0.2	1
P01	89.038	890.38	1.5	2
P02	133.897	1338.97	2.2	3
P03	143.106	1431.06	2.3	3
P04	152.279	1522.79	2.5	3
P05	155.168	1551.68	2.5	3
P06	141.874	1418.74	2.3	3
P07	131.962	1319.62	2.2	3
P08	86.857	868.57	1.4	2
P09	173.338	1733.38	2.8	4
P10	229.518	2295.18	3.8	5
P11	309.736	3097.36	5.1	7
P12	299.831	2998.31	4.9	7
P13	297.694	2976.94	4.9	7
P14	312.579	3125.79	5.1	7
P15	298.904	2989.04	4.9	7
P16	309.065	3090.65	5.1	7
P17	230.009	2300.09	3.8	5
P18	161.833	1618.33	2.7	4
P19	203.757	2037.57	3.3	5
P20	252.481	2524.81	4.1	6
P21	331.87	3318.7	5.4	7
P22	296.543	2965.43	4.9	7

P23	285.981	2859.81	4.7	7
P24	293.749	2937.49	4.8	7
P25	302.954	3029.54	5.0	7
P26	331.148	3311.48	5.4	7
P27	250.878	2508.78	4.1	6
P28	189.879	1898.79	3.1	5
P29	196.035	1960.35	3.2	5
P30	268.958	2689.58	4.4	7
P31	345.435	3454.35	5.7	10
P32	311.38	3113.8	5.1	7
P33	288.334	2883.34	4.7	7
P34	207.616	2076.16	3.4	5
P36	332.245	3322.45	5.4	7
P37	349.057	3490.57	5.7	10
P38	270.469	2704.69	4.4	7
P39	196.627	1966.27	3.2	5
P44	242.572	2425.72	4.0	6
P45	316.571	3165.71	5.2	7
P46	334.771	3347.71	5.5	7
P47	322.772	3227.72	5.3	7
P48	307.167	3071.67	5.0	7
P49	248.684	2486.84	4.1	6
P50	291.33	2913.3	4.8	7
P51	317.312	3173.12	5.2	7
P52	305.528	3055.28	5.0	7
P53	240.374	2403.74	3.9	5
P54	91.382	913.82	1.5	2
P55	137.257	1372.57	2.3	3
P56	138.625	1386.25	2.3	3
P57	124.024	1240.24	2.0	3
P58	128.282	1282.82	2.1	3
P59	128.081	1280.81	2.1	3
P60	127.901	1279.01	2.1	3
P61	94.042	940.42	1.5	2
P35L	105.928	1059.28	1.7	2
P40L	83.311	833.11	1.4	2
P41L	64.272	642.72	1.1	2
P42L	88.281	882.81	1.4	2
P43L	72.191	721.91	1.2	2

4.1.2 Daya Dukung Lateral

- Metode Broms

Tabel 4.22
Perhitungan K_p

Kedalaman (m)	ϕ	Kp
Lapisan 1 (2.2 - 6 m)	21	2.117
Lapisan 2 6 - 10 m	29	2.882
Lapisan 3 10 - 14 m	27	2.663
Lapisan 4 14 - 17 m	39	4.395
Rata-rata		3.014

Diasumsikan kepala tiang tidak terjepit atau terjepit ke dalam pelat penutup kepala tiang kurang dari 60 cm, McNulty (1956) mendefinisikan tiang tersebut termasuk tiang ujung bebas (*free end pile*).

Dengan mengambil momen terhadap ujung bawah,

$H_u = \frac{(\frac{1}{2})\gamma d L^3 K_p}{e+L}$

$= \frac{0{,}5 \; x \; 9.7 \; x \; 0.4 \; x \; 16^3 \; x \; 3.014}{0+16}$

= 1496.873 m

Mencari jarak momen maksimum

$f = 0{,}82 \sqrt{\frac{H_u}{d.K_p.\gamma}}$

$= 0.82 \sqrt{\frac{1496.873}{0.4 \; x \; 3.014 \; x \; 9.7}}$

= 9.277 m

Sehingga momen maksimum dapat dinyatakan

$M_{maks} = H_u \; (e+2f/3)$

$= 1496.873 \; (0 + \frac{2 \; x \; 9.277}{3})$

= 9257.661 kNm

4.1.3 Penurunan Tiang

$S_e = S_{e(1)} + S_{e(2)} + S_{e(3)}$

$S_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p}$

Q_{wp} = 2.118 kN

ξ = 0.5

Q_{ws} = 1522.423 kN

L = 16 m

D = 0.4 m

$A_p = 1/4\pi D^2$

$= 0.126\ m^2$

E_p = 2000000 Mpa

$S_{e(1)} = \frac{(2.118 + 0.5\ x\ 1522.423)16}{0.216\ x\ 2000000}$

= 0.028 m

= 2.8 cm

$S_{e(2)} = \frac{q_w D}{E_s}\left(1 - \mu_s^2\right) I_{wp}$

E_s = 2000 kg/cm^2 = 20 kN/m^2

μ_s = 0.9

I_{wp} = 0.85

$S_{e(2)} = (\frac{2.118}{0.126})(\frac{0.4}{20})\left(1 - 0.9^2\right)0.85$

= 0.054 m

= 5.4 cm

$S_{e(3)}$ = $(\frac{Q_{ws}}{pL})\frac{D}{E_s}\left(1-\mu_s^2\right)I_{ws}$

P = π x 0.4 x 16

= 20.106 m^3

I_{ws} = $2 + 0.35\sqrt{\frac{L}{D}}$

= $2 + 0.35\sqrt{\frac{16}{0.4}}$

= 4.214

$S_{e(3)}$ = $(\frac{1522.423}{20.106\ x\ 16})(\frac{0.4}{20})\left(1-0.9^2\right)4.214$

= 0.076 m

= 7.6 cm

Penurunan total tiang S_e = 2.8 + 5.4 + 7.6 = 15.8 cm

4.1.4 Pile Cap

4.1.4.1 Penulangan Pile Cap

1) Pile Cap Tipe 1

Karena kolom tidak tertumpu pada pile, maka P yang diperhitungkan adalah P kolom

K02 = 294.63 kN

- Penulangan Pile Cap

Data teknis

- f'c = 24.9 MPa = 249 kg/cm^2
- fy = 400 MPa = 4000 kg/cm^2
- β_1 = 0,85
- Kolom = 40/40 cm

a. Kebutuhan jumlah tiang

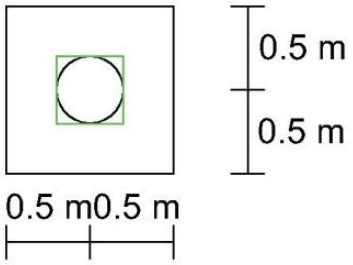
$n._{tiang} = \frac{Pu}{Qa}$

$= \frac{294.63}{565.891} = 0.5 \approx 1$ (dibutuhkan 1 buah tiang)

b. Perhitungan penulangan

Syarat jarak tiang ke tepi → $S \geq 1{,}25.d$

$S \geq 1{,}25\ (40) = 50$ cm



Gambar 4.10 Pile cap 1 tiang tampak atas

Berat mati (W_1) = 0.8 x 1 x 1 x 2400 = 1920 kg

Berat tanah timbunan (W_2) = 0.2 x 1 x 1 x 970 = 194 kg

Gaya aksial: $P_u + W_1 + W_2$ = 29463 + 1920 + 194 = 31577 kg

Gaya pada masing-masing tiang = 31577 kg

Mu = 3422.94 kgm

- Penulangan pile cap

fy	=	400	MPa	= 4000 kg/cm^2
fc'	=	24.9	Mpa	= 249 kg/cm^2
h	=	800	mm	
b	=	1000	mm	
d'	=	70	mm	
d	=	730	mm	

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

ρ max = 0,75 x ρ_b

$= 0{,}75 \left(0{,}85 \frac{f'c}{fy} \beta \frac{600}{600+fy}\right)$

$= 0{,}75 \left(0{,}85 . \frac{224.9}{400} . 0{,}85 . \frac{600}{600+400}\right) = 0{,}0202$

Rn $= \frac{Mu}{\emptyset \text{ x b x } d^2}$

$= \frac{3422.94 \text{ x } 10^4}{0{,}8 \text{ x } 1000 \text{ x } 730^2} = 0{,}08$ Mpa

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

$= \frac{400}{0{,}85 . 24.9} = 18.90$

ρ $= \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.Rn}{fy}}\right)$

$= \frac{1}{18.90}\left(1 - \sqrt{1 - \frac{2.18.90.0{,}08}{400}}\right)$

$= 0{,}0002$

Karena $\rho < \rho_{min}$ → Untuk analisa selanjutnya digunakan; ρ min

As $= \rho_{min}$ x b x d

$= 0{,}0035$ x 1000 x 730 = 2555 mm^2

As' $= 0{,}2$ x As

$= 0{,}2$ x 2555 = 511 mm^2

Digunakan Tulangan :

Tulangan utama = D19 – 100 (As = 2835 mm^2)

Tulangan bagi = D13 – 250 (As' = 531 mm^2)

2) Pile Cap Tipe 2

- Penulangan Pile Cap

Data teknis

- f'c = 24.9 MPa = 249 kg/cm^2
- fy = 400 MPa = 4000 kg/cm^2
- β_1 = 0,85
- Kolom = 40/40 cm

a. Kebutuhan jumlah tiang

$n._{tiang} = \frac{Pu}{Qa}$

$$= \frac{1059.28}{609.816}$$

= 1.7 ≈ 2 tiang

b. Perhitungan penulangan

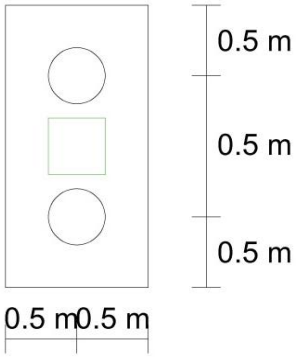
Syarat jarak tiang ke tepi → S ≥ 1,25.d

S ≥ 1,25 (40) = 50 cm

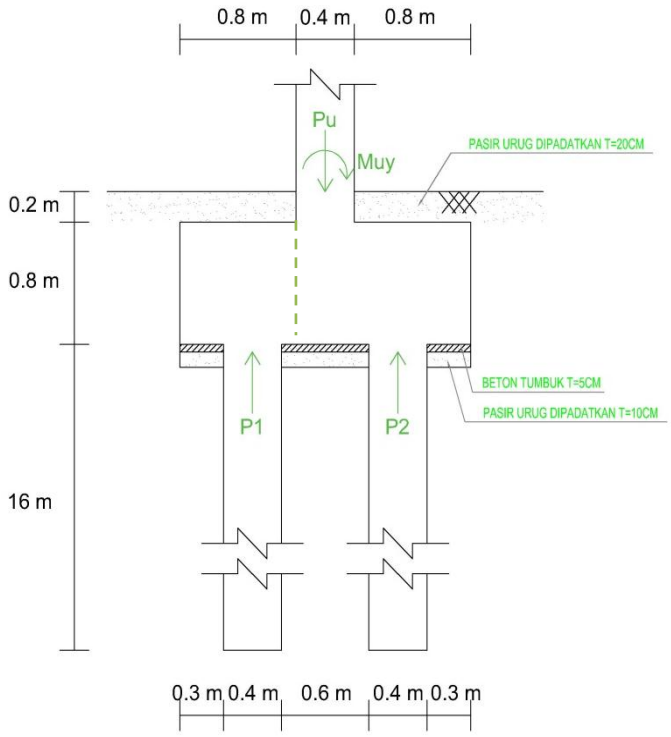
Jarak antar tiang (d) = 2.5D - 3D

= 1 – 1.2

Digunakan jarak antar tiang (d) = 1 m



Gambar 4.11 Pile cap 2 tiang tampak atas



Gambar 4.12 Pile cap 2 tiang arah memanjang

Pu = 105928 kg

Muy = -2082.22 kgm

Berat sendiri pile cap q1 = 1 x 0.8 x 2400 = 1920 kg/m

W1 = 1 x 2 x 0.8 x 2400 = 3840 kg

Berat Over Burden q2 = 1 x 0.2 x 970 = 194 kg/m

W2 = 1 x 2 x 0.2 x 970 = 388 kg/m

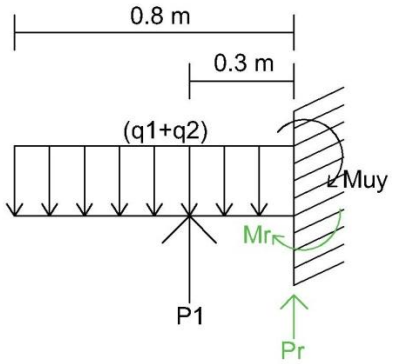
P1 $= \frac{Pu}{2} + \frac{W1}{2} + \frac{W2}{2}$

$= \frac{105928}{2} + \frac{3840}{2} + \frac{388}{2}$

= 55078 kg

q1 + q2 = 1920 + 194 = 2114 kg/m

Menghitung Momen



Gambar 4.13 Distribusi tegangan pile cap tipe 2 tiang terhadap kolom

Pr = (q1 + q2) x 0.8 – P1 = 2114 x 0.8 – 55078 = - 53386.8 kg

Mr = - (-(q1+q2) x 0.8^2 x 0.5 + P1 x 0.3 + Muy)

= - (-(2114) x 0.8^2 x 0.5 + 55078 x 0.3 - 4583.15)

= - 13764.7 kgm

Maka digunakan Momen untuk perencanaan penulangan pile cap = 13764.7 kgm

Penulangan pile cap

fy = 400 MPa = 4000 kg/cm^2

fc' = 24.9 Mpa = 249 kg/cm^2

h = 800 mm

b = 2000 mm

d' = 70 mm

d = 730 mm

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

$\rho \text{ max} = 0,75 \text{ x } \rho_b$

$= 0,75 \left(0,85 \frac{f'c}{fy} \beta \frac{600}{600+fy}\right)$

$= 0,75 \left(0,85 . \frac{224.9}{400} . 0,85 . \frac{600}{600+400}\right) = 0,0202$

$Rn = \frac{Mu}{\emptyset \text{ x b x } d^2}$

$= \frac{13764.7 \text{ x } 10^4}{0,8 \text{ x } 2000 \text{ x } 730^2} = 0,0161 \text{ Mpa}$

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

$= \frac{400}{0,85 . 24.9} = 18.90$

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

$= \frac{1}{18.90}\left(1 - \sqrt{1 - \frac{2.18.90.0,0161}{400}}\right)$

$= 0,000405$

Karena $\rho < \rho_{min} < \rho_{max}$ → Untuk analisa selanjutnya digunakan; ρ min

As = ρ_{min} x b x d

= 0,0035 x 2000 x 730 = 5110 mm^2

As' = 0,2 x As

= 0,2 x 511 = 1022 mm^2

Digunakan Tulangan :

Tulangan utama = D19 – 50 (As = 5671 mm^2)

Tulangan bagi = D13-125 (As' = 1062 mm^2)

3) Pile Cap Tipe 3

- Penulangan Pile Cap

Data teknis

- f'c = 24.9 MPa = 249 kg/cm^2
- fy = 400 MPa = 4000 kg/cm^2
- β_1 = 0,85
- Kolom = 70/70 cm

a. Kebutuhan jumlah tiang

$$n._{tiang} = \frac{Pu}{Qa}$$

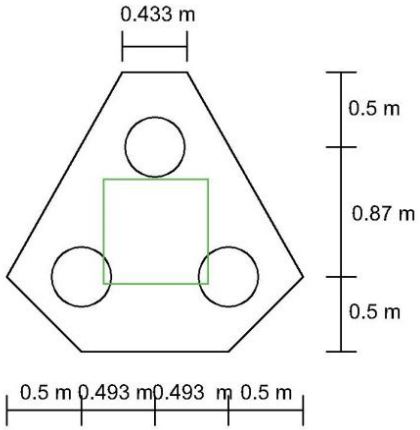
$$= \frac{1551.68}{609.816}$$

$$= 2.5 \approx 3 \text{ tiang}$$

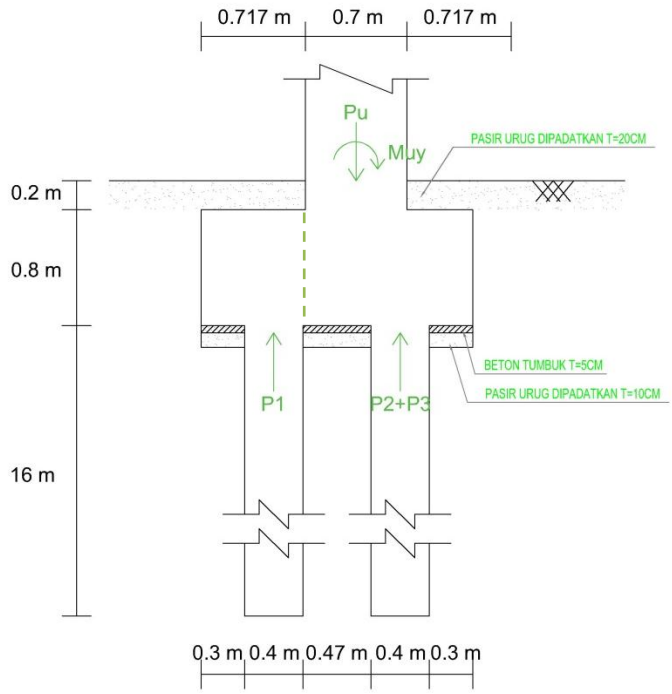
b. Perhitungan penulangan

Syarat jarak tiang ke tepi → S ≥ 1,25.d

S ≥ 1,25 (40) = 50 cm



Gambar 4.14 Pile cap 3 tiang tampak atas



Gambar 4.15 Pile cap 3 tiang arah memanjang

Gamba

Pu = 155168 kg

Muy = 103.55 kgm

Berat sendiri pile cap q1 = 0.717 x 0.8 x 2400 = 1376.64 kg/m

W1 = 1.87 x 1.986 x 0.8 x 2400 = 7130.534 kg

Berat Over Burden q2 = 0.717 x 0.2 x 970 = 139.098 kg/m

W2 = 1.87 x 1.986 x 0.2 x 970 = 720.481 kg/m

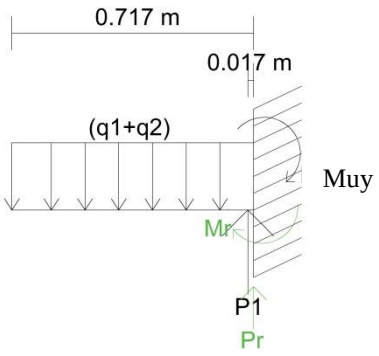
P1 $= \frac{Pu}{2} + \frac{W1}{2} + \frac{W2}{2}$

$= \frac{155168}{3} + \frac{7130.534}{3} + \frac{720.481}{3}$

= 54339.672 kg

q1 + q2 = 1376.64 + 139.098 = 1515.738 kg/m

Menghitung Momen



Gambar 4.16 Distribusi tegangan pile cap tipe 3 tiang terhadap kolom

Pr = (q1 + q2) x 0.717 – P1 = 1515.738 x 0.717 – 54339.672 = - 53252.888 kg

Mr = - (-(q1+q2) x 0.717^2 x 0.5 + P1 x 0.217 + Muy)

= - (-(1515.738) x 0.717^2 x 0.5 + 54339.672 x 0.017 + 103.55

= - 637.712 kgm

Maka digunakan Momen untuk perencanaan penulangan pile cap = 637.712kgm

Penulangan pile cap

fy	=	400	MPa	= 4000 kg/cm^2
fc'	=	24.9	Mpa	= 249 kg/cm^2
h	=	800	mm	
b	=	1986	mm	
d'	=	70	mm	
d	=	730	mm	

ρ min $= \frac{1,4}{fy} = \frac{1,4}{400} = 0,0035$

ρ max = 0,75 x ρ_b

$= 0,75 \left(0,85 \frac{f'c}{fy} \beta \frac{600}{600+fy}\right)$

$= 0,75 \left(0,85 . \frac{224.9}{400} . 0,85 . \frac{600}{600+400}\right) = 0,0202$

Rn $= \frac{Mu}{\emptyset \text{ x b x } d^2}$

$= \frac{103.55 \text{ x } 10^4}{0,8 \text{ x } 1986 \text{x } 730^2}$ = 0,0075 Mpa

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

$= \frac{400}{0{,}85\ .\ 24.9} = 18.90$

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

$= \frac{1}{18.90}\left(1-\sqrt{1-\frac{2.18.90.0{,}0075}{400}}\right)$

$= 0.000019$

Karena $\rho < \rho_{min} < \rho_{max}$ → Untuk analisa selanjutnya digunakan; ρ min

As $= \rho_{min}$ x b x d

$= 0{,}0035 \times 1986 \times 730 = 5074.23\ mm^2$

As' = 0,2 x As

$= 0{,}2 \times 5074.23 = 1014.846\ mm^2$

Digunakan Tulangan :

Tulangan utama = D19 – 50 (As = 5671 mm^2)

Tulangan bagi = D13-125 (As' = 1062 mm^2)

4) Pile Cap Tipe 4

- Penulangan Pile Cap

Data teknis

- f'c = 24.9 MPa = 249 kg/cm^2
- fy = 400 MPa = 4000 kg/cm^2
- β_1 = 0,85
- Kolom = 70/70 cm

a. Kebutuhan jumlah tiang

$n._{tiang} = \frac{Pu}{Qa}$

$= \frac{1733.38}{609.816}$

$= 2.8 \approx 4$ tiang

b. Perhitungan penulangan

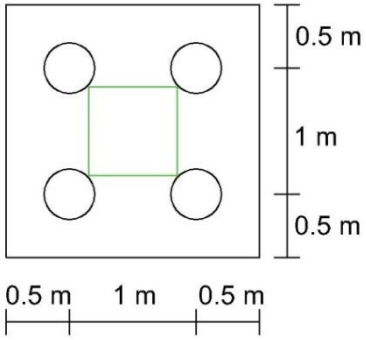
Syarat jarak tiang ke tepi → $S \geq 1{,}25.d$

$$S \geq 1{,}25\ (40) = 50\ \text{cm}$$

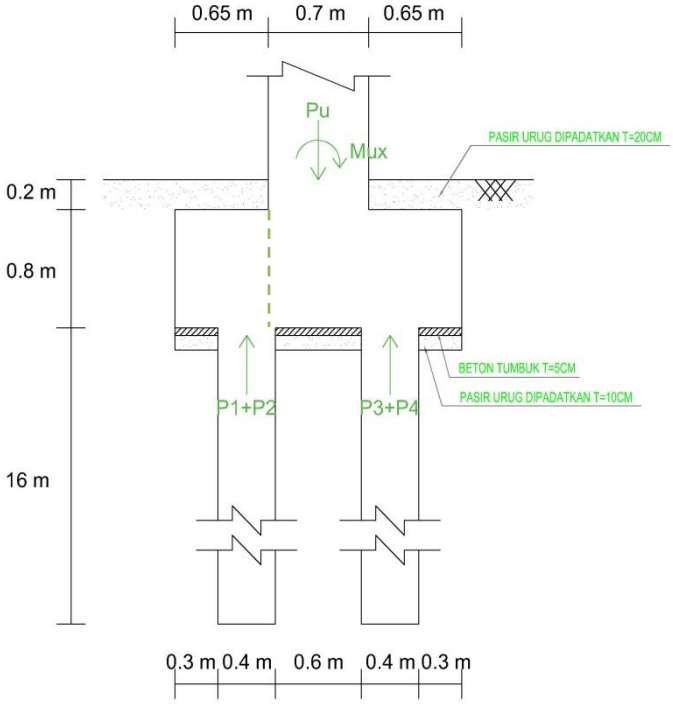
Jarak antar tiang (d) = 2.5D - 3D

= 1 – 1.2

Digunakan jarak antar tiang (d) = 1 m



Gambar 4.17 Pile cap 4 tiang tampak atas



Gambar 4.18 Pile cap 4 tiang arah melintang

Pu = 173338 kg

Mux = 3255.66 kgm

Berat sendiri pile cap q1 = 2 x 0.8 x 2400 = 3840 kg/m

W1 = 2 x 2 x 0.8 x 2400 = 7680 kg

Berat Over Burden q2 = 2 x 0.2 x 970 = 388 kg/m

W2 = 2 x 2 x 0.2 x 970 = 776 kg/m

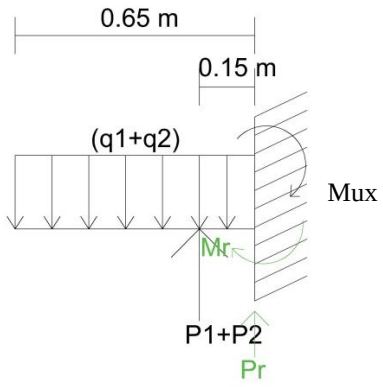
P1 = P2 = $\frac{Pu}{4} + \frac{W1}{4} + \frac{W2}{4}$

= $\frac{173338}{4} + \frac{7680}{4} + \frac{776}{4}$

= 45448.5 kg

q1 + q2 = 3840 + 388 = 4228 kg/m

Menghitung Momen



Gambar 4.19 Distribusi tegangan pile cap tipe 4 tiang terhadap kolom

Pr = (q1 + q2) x 0.65 – (P1+P2) = 4228 x 0.65 – 90897 = - 88148.8 kg

Mr = - ((-q1+q2) x 0.65^2 x 0.5 + (P1+P2) x 0.15 + Mux)

= - ((-4228) x 0.65^2 x 0.5 + 90897 x 0.15 + 3255.66)

= -15997.045 kgm

Maka digunakan Momen untuk perencanaan penulangan pile cap = 15997.045 kgm

Penulangan pile cap

fy	=	400	MPa	= 4000 kg/cm^2
fc'	=	24.9	Mpa	= 249 kg/cm^2
h	=	800	mm	
b	=	2000	mm	
d'	=	70	mm	

d = 730 mm

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

$\rho \text{ max} = 0,75 \times \rho_b$

$= 0,75 \left(0,85 \frac{f'c}{fy} \beta \frac{600}{600+fy}\right)$

$= 0,75 \left(0,85 . \frac{224.9}{400} . 0,85 . \frac{600}{600+400}\right) = 0,0202$

$Rn = \frac{Mu}{\emptyset \times b \times d^2}$

$= \frac{15997.045 \times 10^4}{0,8 \times 2000 \times 730^2} = 0,1876 \text{ Mpa}$

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

$= \frac{400}{0,85 . 24.9} = 18.90$

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

$= \frac{1}{18.90}\left(1 - \sqrt{1 - \frac{2.18.90.0,1876}{400}}\right)$

$= 0.000471$

Karena $\rho < \rho_{min} < \rho_{max}$ → Untuk analisa selanjutnya digunakan; ρ min

$As = \rho_{min} \times b \times d$

$= 0,0035 \times 2000 \times 730 = 5110 \text{ mm}^2$

$As' = 0,2 \times As$

$= 0,2 \times 5110 = 1022 \text{ mm}^2$

Digunakan Tulangan :

Tulangan utama = D19 – 50 ($As = 5671 \text{ mm}^2$)

Tulangan bagi = D13-125 ($As' = 1062 \text{ mm}^2$)

5) Pile Cap Tipe 5

- Penulangan Pile Cap

Data teknis

- f'c = 24.9 MPa = 249 kg/cm^2
- fy = 400 MPa = 4000 kg/cm^2
- β_1 = 0,85

- Kolom = 70/70 cm

a. Kebutuhan jumlah tiang

$$n._{tiang} = \frac{Pu}{Qa}$$

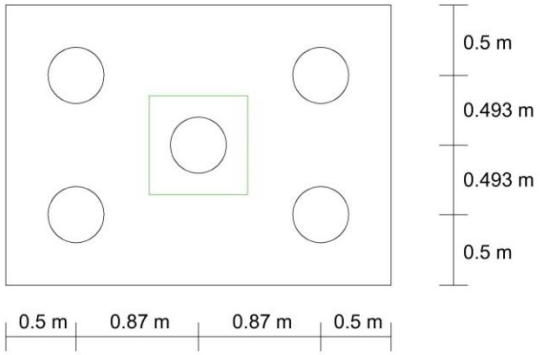
$$= \frac{2403.74}{609.816}$$

$$= 3.9 \approx 5 \text{ tiang}$$

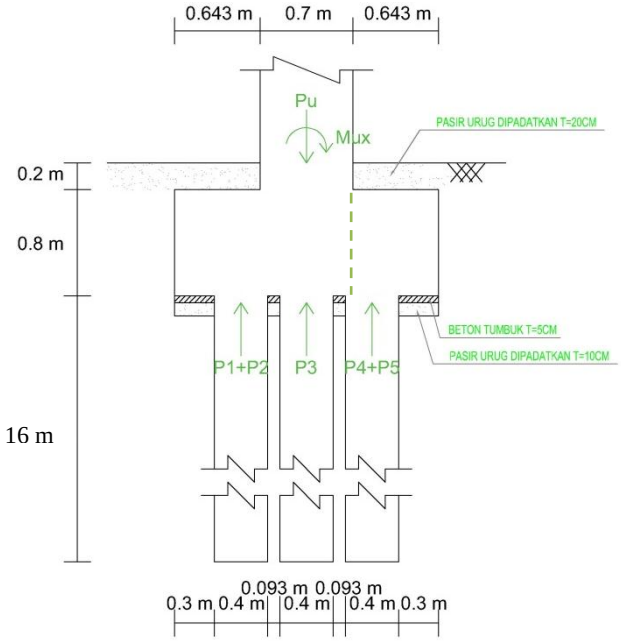
b. Perhitungan penulangan

Syarat jarak tiang ke tepi → S ≥ 1,25.d

S ≥ 1,25 (40) = 50 cm



Gambar 4.20 Pile cap 5 tiang tampak atas



Gambar 4.21 Pile cap 5 tiang arah memanjang

Pu = 240374 kg

Mux = -6672.43 kgm

Berat sendiri pile cap q1 = 2.74 x 0.8 x 2400 = 5260.8 kg/m

W1 = 2.74 x 1.986 x 0.8 x 2400 = 10447.949 kg

Berat Over Burden q2 = 2.74 x 0.2 x 970 = 531.56 kg/m

W2 = 2.74 x 1.986 x 0.2 x 970 = 1055.678 kg/m

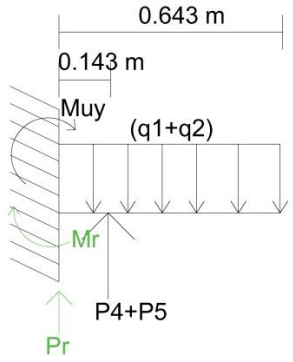
$P4 = P5 = \frac{Pu}{5} + \frac{W1}{5} + \frac{W2}{5}$

$= \frac{240374}{5} + \frac{10447.949}{5} + \frac{1055.678}{5}$

= 50375.525 kg

q1 + q2 = 5260.8 + 531.56 = 5792.36 kg/m

Menghitung Momen



Gambar 4.22 Distribusi tegangan pile cap tipe 5 tiang terhadap kolom

Pr = (q1 + q2) x 0.643 – (P4+P5) = 5792.36 x 0.65 – 100751.051 = - 97026.563 kg

Mr = - ((q1+q2) x 0.643^2 x 0.5 – (P4+P5) x 0.143 + Mux)

= - ((5792.36) x 0.643^2 x 0.5 - 97026.563 x 0.143 – 6672.43)

= - 12678.707 kgm

Maka digunakan Momen untuk perencanaan penulangan pile cap = 12678.707 kgm

Penulangan pile cap

fy = 400 MPa = 4000 kg/cm^2

fc' = 24.9 Mpa = 249 kg/cm^2

h = 800 mm

b = 2740 mm

d' = 70 mm

d = 730 mm

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

$\rho \text{ max} = 0,75 \text{ x } \rho_b$

$= 0,75 \left(0,85 \frac{f'c}{fy} \beta \frac{600}{600+fy}\right)$

$= 0,75 \left(0,85 . \frac{224.9}{400} . 0,85 . \frac{600}{600+400}\right) = 0,0202$

$Rn = \frac{Mu}{\emptyset \text{ x b x } d^2}$

$= \frac{12678.707 \text{ x } 10^4}{0,8 \text{ x } 2740 \text{ x } 730^2} = 0,1085 \text{ Mpa}$

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

$= \frac{400}{0,85 . 24.9} = 18.90$

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

$= \frac{1}{18.90}\left(1 - \sqrt{1 - \frac{2.18.90.0,1085}{400}}\right)$

$= 0.000272$

Karena $\rho < \rho_{min} < \rho_{max}$ → Untuk analisa selanjutnya digunakan; ρ min

$As = \rho_{min} \text{ x b x d}$

$= 0,0035 \text{ x } 2740 \text{ x } 730 = 7000.70 \text{ mm}^2$

As' = 0,2 x As

$= 0,2 \text{ x } 7000.70 = 1400.14 \text{ mm}^2$

Digunakan Tulangan :

Tulangan utama = D32 – 100 (As = 8042 mm^2)

Tulangan bagi = D16 – 125 (As' = 1608 mm^2)

6) Pile Cap Tipe 6

- Penulangan Pile Cap

Data teknis

- f'c = 24.9 MPa = 249 kg/cm^2
- fy = 400 MPa = 4000 kg/cm^2
- β_1 = 0,85
- Kolom = 70/70 cm

a. Kebutuhan jumlah tiang

$$n._{tiang} = \frac{Pu}{Qa}$$

$$= \frac{2524.81}{609.816}$$

$$= 4.1 \approx 6 \text{ tiang}$$

b. Perhitungan penulangan

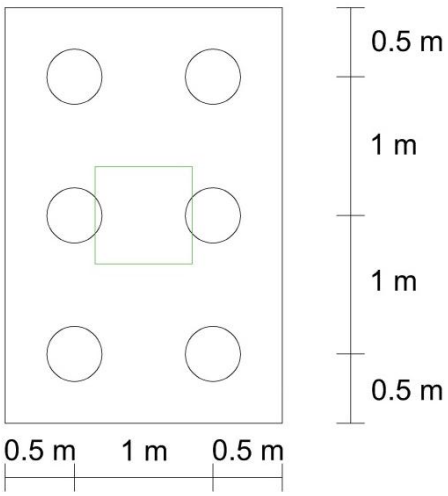
Syarat jarak tiang ke tepi → S ≥ 1,25.d

S ≥ 1,25 (40) = 50 cm

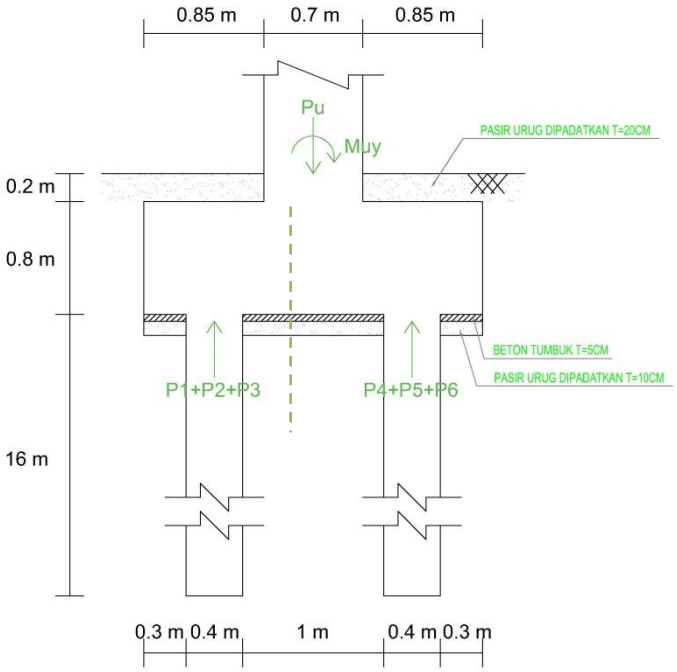
Jarak antar tiang (d) = 2.5D - 3D

= 1 – 1.2

Digunakan jarak antar tiang (d) = 1 m



Gambar 4.23 Pile cap 6 tiang tampak atas



Gambar 4.24 Pile cap 6 tiang arah melintang

Pu = 252481 kg

Muy = 1432.86 kgm

Berat sendiri pile cap q1 = 3 x 0.8 x 2400 = 5760 kg/m

W1 = 3 x 2 x 0.8 x 2400 = 11520 kg

Berat Over Burden q2 = 3 x 0.2 x 970 = 582 kg/m

W2 = 3 x 2 x 0.2 x 970 = 1164 kg/m

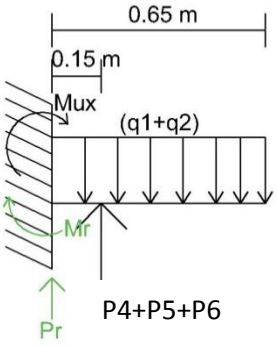
P4 = P5 = P6 $= \frac{Pu}{6} + \frac{W1}{6} + \frac{W2}{6}$

$= \frac{252481}{6} + \frac{11520}{6} + \frac{1164}{6}$

= 44194.167 kg

q1 + q2 = 5760 + 582 = 6342 kg/m

Menghitung Momen



Gambar 4.25 Distribusi teganganpile cap tipe 6 tiang terhadap kolom

Pr = (q1 + q2) x 0.65 – (P4+P5+P6) = 6342 x 0.65 – 132583 = - 128460.2 kg

Mr = - ((q1+q2) x 0.65^2 x 0.5 - (P4+P5+P6) x 0.15 + Muy)

= - ((6342) x 0.65^2 x 0.5 - 132583 x 0.15 + 1432.86)

= - 17114.843 kgm

Maka digunakan Momen untuk perencanaan penulangan pile cap = 17114.843 kgm

Penulangan pile cap

fy	=	400	MPa	= 4000 kg/cm^2
fc'	=	24.9	Mpa	= 249 kg/cm^2
h	=	800	mm	
b	=	3000	mm	
d'	=	70	mm	
d	=	730	mm	

ρ min $= \frac{1,4}{fy} = \frac{1,4}{400} = 0,0035$

ρ max $= 0,75 \times \rho_b$

$= 0,75 \left(0,85 \frac{f'c}{fy} \beta \frac{600}{600+fy}\right)$

$= 0,75 \left(0,85 . \frac{224.9}{400} . 0,85 . \frac{600}{600+400}\right) = 0,0202$

Rn $= \frac{Mu}{\emptyset \times b \times d^2}$

$= \frac{17114.843 \times 10^4}{0,8 \times 3000 \times 730^2} = 0,1338$ Mpa

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

$= \frac{400}{0{,}85\ .\ 24.9} = 18.90$

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

$= \frac{1}{18.90}\left(1-\sqrt{1-\frac{2.18.90.0{,}1338}{400}}\right)$

$= 0.000336$

Karena $\rho < \rho_{min} < \rho_{max}$ → Untuk analisa selanjutnya digunakan; ρ min

As = ρ_{min} x b x d

= 0,0035 x 3000 x 730 = 7665 mm^2

As' = 0,2 x As

= 0,2 x 7665 = 1533 mm^2

Digunakan Tulangan :

Tulangan utama = D32 – 100 (As = 8042 mm2)

Tulangan bagi = D16 – 125 (As' = 1608 mm2)

7) Pile Cap Tipe 7

- Penulangan Pile Cap

Data teknis

- f'c = 24.9 MPa = 249 kg/cm^2
- fy = 400 MPa = 4000 kg/cm^2
- β_1 = 0,85
- Kolom = 70/70 cm

a. Kebutuhan jumlah tiang

$n._{tiang} = \frac{Pu}{Qa}$

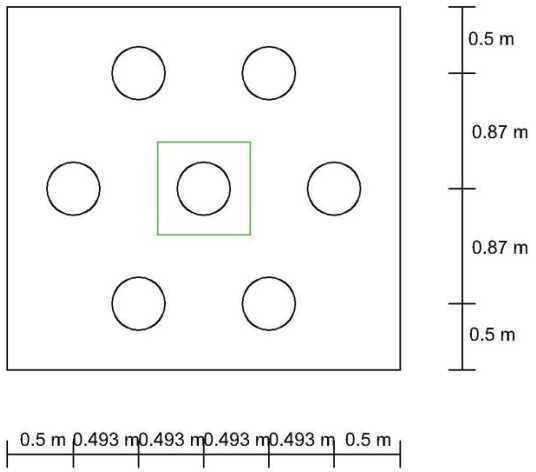
$= \frac{3347.71}{609.816}$

= 5.5 ≈ 7 tiang

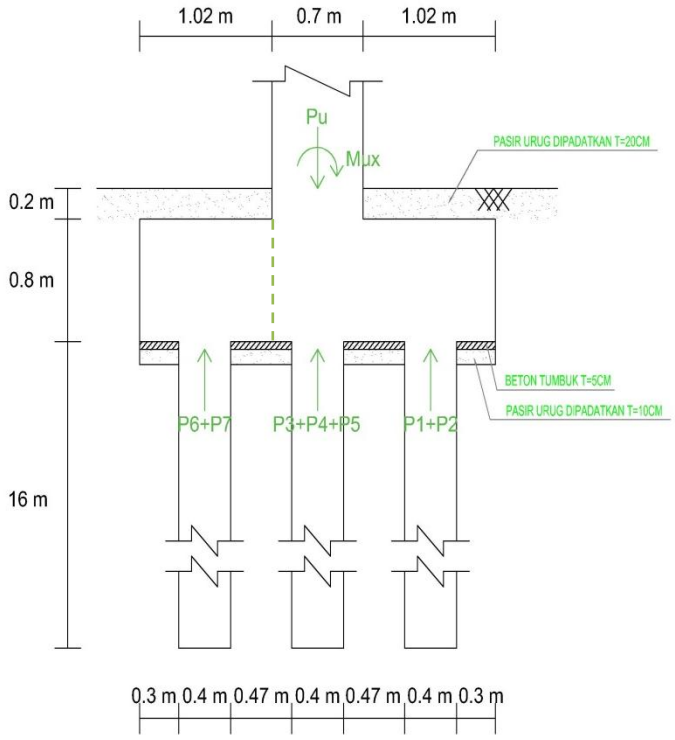
b. Perhitungan penulangan

Syarat jarak tiang ke tepi → S ≥ 1,25.d

$$S \geq 1{,}25\ (40) = 50\ \text{cm}$$



Gambar 4.26 Pile cap 7 tiang tampak atas



Gambar 4.27 Pile cap 7 tiang arah melintang

Pu = 334771 kg

Mux = -527.73 kgm

Berat sendiri pile cap q1 = 2.74 x 0.8 x 2400 = 5706.24 kg/m

W1 = 2.74 x 2.972 x 0.8 x 2400 = 15635.098 kg

Berat Over Burden q2 = 2.74 x 0.2 x 970 = 576.568 kg/m

W2 = 2.74 x 2.972 x 0.2 x 970 = 1579.796 kg/m

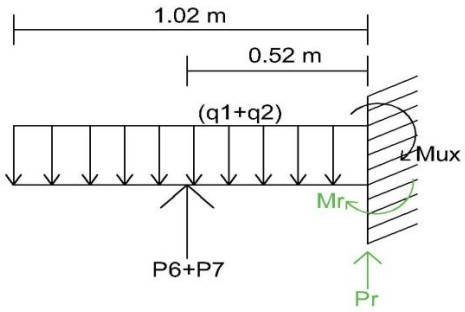
P1 = P2 $= \frac{Pu}{7} + \frac{W1}{7} + \frac{W2}{7}$

$= \frac{334771}{7} + \frac{15635.098}{7} + \frac{1579.796}{7}$

= 100567.4 kg

q1 + q2 = 5706.24 + 576.568 = 6282.808 kg/m

Menghitung Momen



Gambar 4.28 Distribusi tegangan pile cap tipe 7 tiang terhadap kolom

Pr = (q1 + q2) x 1.02 – (P1+P2) = 6282.808 x 1.02 – 100567.4 = - 94158.934 kg

Mr = - (-(q1+q2) x 1.02^2 x 0.5 + (P6+P7) x 0.52 + Mux)

= - (-(6282.808) x 1.02^2 x 0.5 + 100567.4 x 0.52 + 527.73)

= -49554.46 kgm

Maka digunakan Momen untuk perencanaan penulangan pile cap = 49554.46 kgm

Penulangan pile cap

fy	=	400	MPa	= 4000 kg/cm^2
fc'	=	24.9	Mpa	= 249 kg/cm^2
h	=	800	mm	
b	=	2972	mm	
d'	=	70	mm	
d	=	730	mm	

ρ min $= \frac{1,4}{fy} = \frac{1,4}{400} = 0,0035$

ρ max = 0,75 x ρ_b

$= 0{,}75 \left(0{,}85 \frac{f'c}{fy} \beta \frac{600}{600+fy}\right)$

$= 0{,}75 \left(0{,}85 . \frac{224.9}{400} . 0{,}85 . \frac{600}{600+400}\right) = 0{,}0202$

Rn $= \frac{Mu}{\emptyset \text{ x b x } d^2}$

$= \frac{49554.46 \text{ x } 10^4}{0{,}8 \text{ x } 2972 \text{ x } 730^2} = 0.3911$ Mpa

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

$= \frac{400}{0{,}85 . 24.9} = 18.90$

ρ $= \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.m.Rn}{fy}}\right)$

$= \frac{1}{18.90}\left(1 - \sqrt{1 - \frac{2.18.90.0{,}3911}{400}}\right)$

$= 0.000987$

Karena $\rho < \rho_{min} < \rho_{max}$ → Untuk analisa selanjutnya digunakan; ρ min

As $= \rho_{min}$ x b x d

$= 0{,}0035$ x 2972 x 730 = 7593.46 mm^2

As' $= 0{,}2$ x As

$= 0{,}2$ x 7593.46 = 1518.692 mm^2

Digunakan Tulangan :

Tulangan utama = D32 – 100 (As = 8042 mm2)

Tulangan bagi = D16 – 125 (As' = 1608 mm2)

8) Pile Cap Tipe 10

- Penulangan Pile Cap

Data teknis

- f'c = 24.9 MPa = 249 kg/cm^2
- fy = 400 MPa = 4000 kg/cm^2
- β_1 = 0,85
- Kolom = 70/70 cm

a. Kebutuhan jumlah tiang

$n._{tiang} = \frac{Pu}{Qa}$

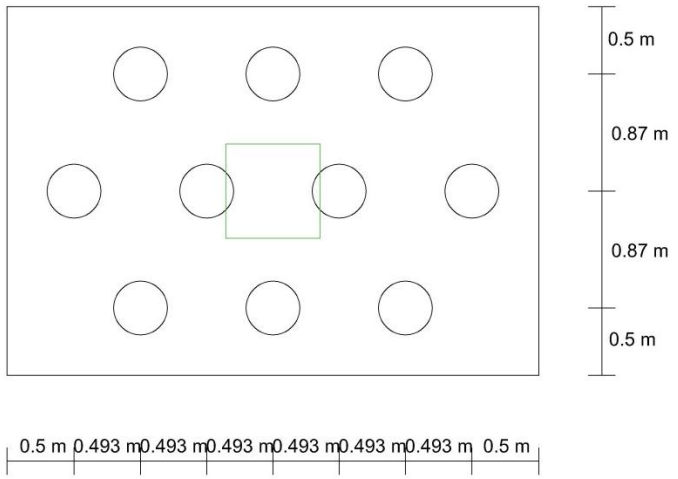
$$= \frac{3490.57}{609.816}$$

= 5.7 ≈ 10 tiang

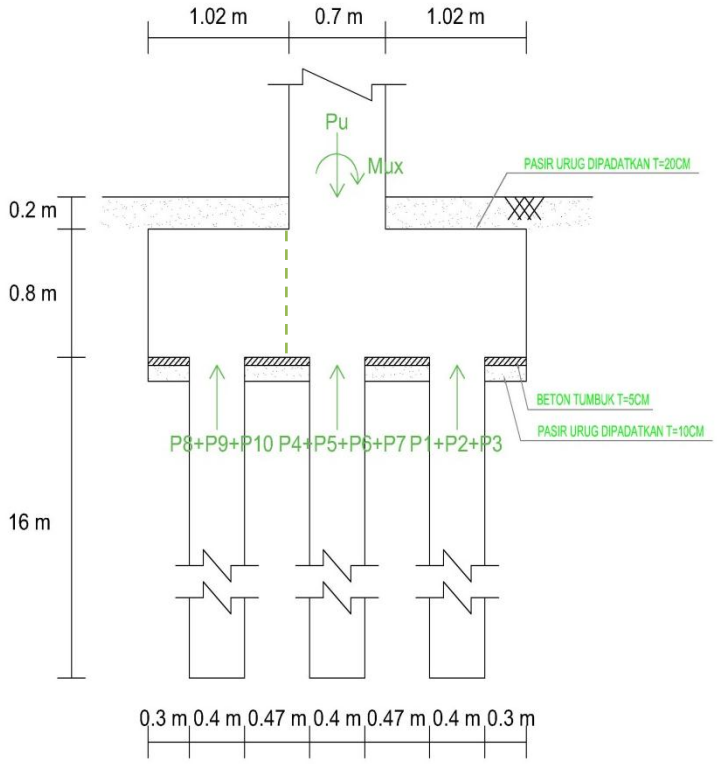
b. Perhitungan penulangan

Syarat jarak tiang ke tepi → S ≥ 1,25.d

S ≥ 1,25 (40) = 50 cm



Gambar 4.29 Pile cap 10 tiang tampak atas



Gambar 4.30 Pile cap 10 tiang arah melintang

Pu = 3490.57 kg

Mux = - 3167.4 kgm

Berat sendiri pile cap q1 = 3.958 x 0.8 x 2400 = 7599.36 kg/m

W1 = 2.74 x 3.958 x 0.8 x 2400 = 20822.246 kg

Berat Over Burden q2 = 3.958 x 0.2 x 970 = 767.852 kg/m

W2 = 2.74 x 3.958 x 0.2 x 970 = 2103.914 kg/m

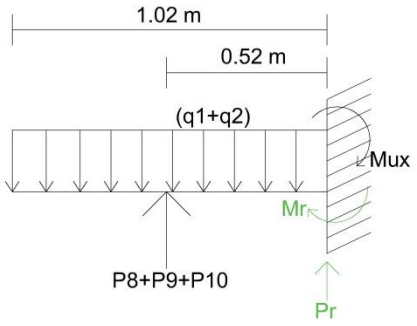
$P8 = P9 = P10 = \frac{Pu}{10} + \frac{W1}{10} + \frac{W2}{10}$

$= \frac{3490.57}{10} + \frac{20822.246}{10} + \frac{2103.914}{10}$

= 37198.316 kg

q1 + q2 = 7599.36 + 767.852 = 8367.212 kg/m

Menghitung Momen



Gambar 4.31 Distribusi tegangan pile cap tipe 10 tiang terhadap kolom

Pr = (q1 + q2) x 1.02 – (P8+P9+P10) = 8367.212 x 1.02 – 111594.9 = - 103060.392kg

Mr = - (-(q1+q2) x 1.02^2 x 0.5 + (P8+P9+P10) x 0.52 + Mux)

= - (-(8367.212) x 1.02^2 x 0.5 + 103060.392 x 0.52 – 3167.4)

= - 50509.349 kgm

Maka digunakan Momen untuk perencanaan penulangan pile cap = 50509.349 kgm

Penulangan pile cap

fy = 400 MPa = 4000 kg/cm^2

fc' = 24.9 Mpa = 249 kg/cm^2

h = 800 mm

b = 3958 mm

d' = 70 mm

d = 730 mm

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

$\rho \text{ max} = 0,75 \text{ x } \rho_b$

$= 0,75 \left(0,85 \frac{f'c}{fy} \beta \frac{600}{600+fy}\right)$

$= 0,75 \left(0,85 . \frac{224.9}{400} . 0,85 . \frac{600}{600+400}\right) = 0,0202$

$Rn = \frac{Mu}{\emptyset \text{ x b x } d^2}$

$= \frac{50509.349 \text{ x } 10^4}{0,8 \text{ x } 3958 \text{ x } 730^2} = 0.2993 \text{ Mpa}$

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

$= \frac{400}{0,85 . 24.9} = 18.90$

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

$= \frac{1}{18.90}\left(1 - \sqrt{1 - \frac{2.18.90.0,2993}{400}}\right)$

$= 0.00075$

Karena $\rho < \rho_{min} < \rho_{max}$ → Untuk analisa selanjutnya digunakan; ρ min

$As = \rho_{min} \text{ x b x d}$

$= 0,0035 \text{ x } 3958 \text{ x } 730 = 10112.69 \text{ mm}^2$

As' = 0,2 x As

$= 0,2 \text{ x } 10112.69 = 2022.538 \text{ mm}^2$

Digunakan Tulangan :

Tulangan utama = D36 – 100 (As = 10179 mm^2)

Tulangan bagi = D19 – 125 (As' = 2268 mm^2)

4.1.4.2 Penulangan Tiang Pancang

Digunakan 16 – D36 (As = 162.86 cm^2)

Luas tiang pancang (Ag) = 1256.637 cm^2

Ø.Pn = 0,85.0,7.(0,85.f'c.(Ag – As) + As.fy)

= 0,85 x 0,7 x (0,85 x 249 x (1256.637 – 162.86) + (162.86 x 4000))

= 525348.06 kg

$\frac{\varphi.Pn}{1,5} = \frac{525348.06}{1,5}$ = 350232.04 kg > 349057 kg **(Aman)**

4.1.4.3 Penulangan Geser

- Diameter tiang pancang = 400 mm
- Luas tiang pancang (Ag) = 125663.706 mm^2
- d' = 40 mm
- Dc = 400 – (2 x 40) = 320 mm
- Ac = ¼πd^2 = ¼.π.320^2 = 80424.772 mm^2
- fy = 400 MPa
- f'c = 24.9MPa
- db = 13 mm
- a_s = ¼πd^2 = ¼.π.13^2 = 132,732 mm^2

$$\rho_s = 0,45\left[\frac{Ag}{Ac} - 1\right]\frac{f'c}{fy}$$

$$= 0,45\left[\frac{125663.706}{804242.772} - 1\right]\frac{24.9}{400}$$

= 0,016

$$s = \frac{4.a_s(Dc - db)}{\rho_s.Dc^2}$$

$$= \frac{4x132,732\ x(320-13)}{0,016x320^2} = 101.018 \text{ mm}$$

➔ maka dipakai jarak sengkang spiral ø13 – 100 mm

4.2 Pondasi Tiang Bor

4.2.1 Daya Dukung Pondasi Tiang Bor

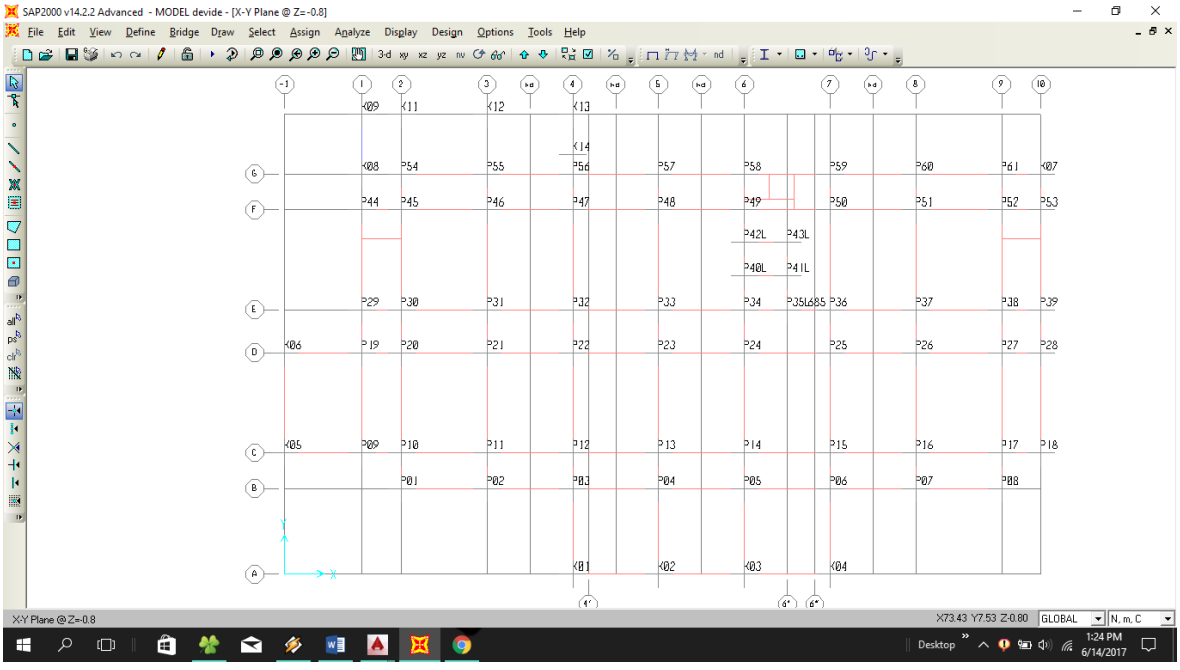
Dari data yang telah didapatkan dari konsultan perencanaan proyek gedung pembangunan Rumah Sakit Gigi dan Mulut Universitas Brawijaya. Adapun pondasi yang digunakan menggunakan tiang bor diameter 80 cm dengan data perhitungan kekuatan sebagai berikut: (hasil tes tanah terlampir)

Tabel 4.23
Daya dukung tiang bor berdasarkan data SPT

Depth m	N-SPT bpf	qp t/m2	fs t/m2	Qp ton	Qs ton	Qult ton	Qall-tention ton	Qall-comp ton
1.25	7	46.8	14.0	23.5	14.0	37.5	3.91	13.44
3.25	2	171.7	10.8	86.3	24.8	111.1	6.94	38.67
5.25	42	274.2	45.2	137.8	70.0	207.8	19.59	73.93
7.25	40	301	84.3	151.3	154.3	305.6	43.18	112.11
9.25	50	334.4	92.4	168.1	246.7	414.8	69.96	154.69
11.25	50	263.5	75.2	132.4	321.9	454.3	90.10	172.86
13.25	29	205.5	63.1	103.3	384.9	488.2	107.78	188.40
15.25	50	210.1	59.3	105.6	444.3	549.9	124.29	212.90
17.25	13	210.7	64.7	105.9	508.9	614.8	142.51	238.88
19.25	50	333.5	47.2	167.6	556.1	723.8	155.74	278.36
21.25	50	256.8	75.2	129.1	631.3	760.4	176.78	295.56
23.25	27	193.4	57.9	97.2	689.2	786.4	192.96	308.07
25.25	31	270.1	43.5	135.8	732.7	868.4	205.14	338.31
27.25	50	333.5	60.8	167.6	793.5	961.1	222.17	373.26
29.25	50	333.5	75.2	167.6	868.7	1036.3	243.21	403.32

Dipakai tiang bor diameter 80 cm kedalaman 13,25 m dengan daya dukung 1 tiang bor sebesar 188,40 ton

Kebutuhan tiang bor



Gambar 4.32 Denah perletakan kolom

Tabel 4.24
Kebutuhan tiang bor

Joint	Kombinasi	P Ton	Kebutuhan Tiang Bor	Pasang Tiang Bor	Kedalaman m	Keterangan
K01	1D+1L	16.189	0.395	1	9	Ø60 cm
K02	1D+1L	29.463	0.719	1	9	Ø60 cm
K03	1D+1L	29.426	0.719	1	9	Ø60 cm
K04	1D+1L	16.179	0.395	1	9	Ø60 cm
K05	1D+1L	21.691	0.53	1	9	Ø60 cm
K06	1D+1L	21.47	0.524	1	9	Ø60 cm
K07	1D+1L	12.852	0.314	1	9	Ø60 cm
K08	1D+1L	7.682	0.188	1	9	Ø60 cm
K09	1D+1L	17.884	0.437	1	9	Ø60 cm
K10	1D+1L	11.45	0.28	1	9	Ø60 cm
K11	1D+1L	0.476	0.012	1	9	Ø60 cm
K12	1D+1L	1.586	0.039	1	9	Ø60 cm
K13	1D+1L	10.285	0.251	1	9	Ø60 cm
K14	1D+1L	10.789	0.263	1	9	Ø60 cm
P01	1D+1L	89.038	0.473	1	13.5	Ø80 cm
P02	1D+1L	133.897	0.711	2	13.5	Ø80 cm
P03	1D+1L	143.106	0.76	2	13.5	Ø80 cm
P04	1D+1L	152.279	0.808	2	13.5	Ø80 cm
P05	1D+1L	155.168	0.824	2	13.5	Ø80 cm
P06	1D+1L	141.874	0.753	2	13.5	Ø80 cm
P07	1D+1L	131.962	0.7	2	13.5	Ø80 cm
P08	1D+1L	86.857	0.461	1	13.5	Ø80 cm
P09	1D+1L	173.338	0.92	2	13.5	Ø80 cm
P10	1D+1L	229.518	1.218	2	13.5	Ø80 cm
P11	1D+1L	309.736	1.644	3	13.5	Ø80 cm
P12	1D+1L	299.831	1.591	3	13.5	Ø80 cm
P13	1D+1L	297.694	1.58	3	13.5	Ø80 cm
P14	1D+1L	312.579	1.659	3	13.5	Ø80 cm
P15	1D+1L	298.904	1.587	3	13.5	Ø80 cm
P16	1D+1L	309.065	1.64	3	13.5	Ø80 cm
P17	1D+1L	230.009	1.221	2	13.5	Ø80 cm
P18	1D+1L	161.833	0.859	2	13.5	Ø80 cm
P19	1D+1L	203.757	1.082	2	13.5	Ø80 cm
P20	1D+1L	252.481	1.34	2	13.5	Ø80 cm
P21	1D+1L	331.87	1.762	3	13.5	Ø80 cm
P22	1D+1L	296.543	1.574	3	13.5	Ø80 cm
P23	1D+1L	285.981	1.518	3	13.5	Ø80 cm
P24	1D+1L	293.749	1.559	3	13.5	Ø80 cm
P25	1D+1L	302.954	1.608	3	13.5	Ø80 cm

P26	1D+1L	331.148	1.758	3	13.5	Ø80 cm
P27	1D+1L	250.878	1.332	2	13.5	Ø80 cm
P28	1D+1L	189.879	1.008	2	13.5	Ø80 cm
P29	1D+1L	196.035	1.041	2	13.5	Ø80 cm
P30	1D+1L	268.958	1.428	3	13.5	Ø80 cm
P31	1D+1L	345.435	1.834	3	13.5	Ø80 cm
P32	1D+1L	311.38	1.653	3	13.5	Ø80 cm
P33	1D+1L	288.334	1.53	3	13.5	Ø80 cm
P34	1D+1L	207.616	1.102	2	13.5	Ø80 cm
P36	1D+1L	332.245	1.764	3	13.5	Ø80 cm
P37	1D+1L	349.057	1.853	3	13.5	Ø80 cm
P38	1D+1L	270.469	1.436	3	13.5	Ø80 cm
P39	1D+1L	196.627	1.044	2	13.5	Ø80 cm
P44	1D+1L	242.572	1.288	2	13.5	Ø80 cm
P45	1D+1L	316.571	1.68	3	13.5	Ø80 cm
P46	1D+1L	334.771	1.777	3	13.5	Ø80 cm
P47	1D+1L	322.772	1.713	3	13.5	Ø80 cm
P48	1D+1L	307.167	1.63	3	13.5	Ø80 cm
P49	1D+1L	248.684	1.32	2	13.5	Ø80 cm
P50	1D+1L	291.33	1.546	3	13.5	Ø80 cm
P51	1D+1L	317.312	1.684	3	13.5	Ø80 cm
P52	1D+1L	305.528	1.622	3	13.5	Ø80 cm
P53	1D+1L	240.374	1.276	2	13.5	Ø80 cm
P54	1D+1L	91.382	0.485	1	13.5	Ø80 cm
P55	1D+1L	137.257	0.729	2	13.5	Ø80 cm
P56	1D+1L	138.625	0.736	2	13.5	Ø80 cm
P57	1D+1L	124.024	0.658	1	13.5	Ø80 cm
P58	1D+1L	128.282	0.681	2	13.5	Ø80 cm
P59	1D+1L	128.081	0.68	1	13.5	Ø80 cm
P60	1D+1L	127.901	0.679	1	13.5	Ø80 cm
P61	1D+1L	94.042	0.499	1	13.5	Ø80 cm
P35L	1D+1L	105.928	0.562	1	13.5	Ø80 cm
P40L	1D+1L	83.311	0.442	1	13.5	Ø80 cm
P41L	1D+1L	64.272	0.341	1	13.5	Ø80 cm
P42L	1D+1L	88.281	0.469	1	13.5	Ø80 cm
P43L	1D+1L	72.191	0.383	1	13.5	Ø80 cm

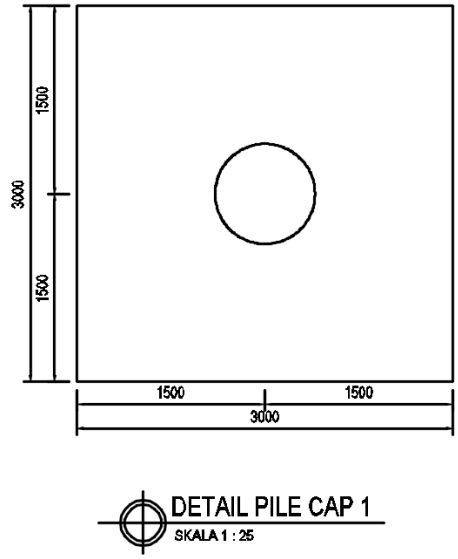
4.2.2 Pile Cap Pondasi Tiang Bor

Syarat jarak antar tiang pancang (s) **berdasarkan Dirjen Bina Marga Departemen PU :**

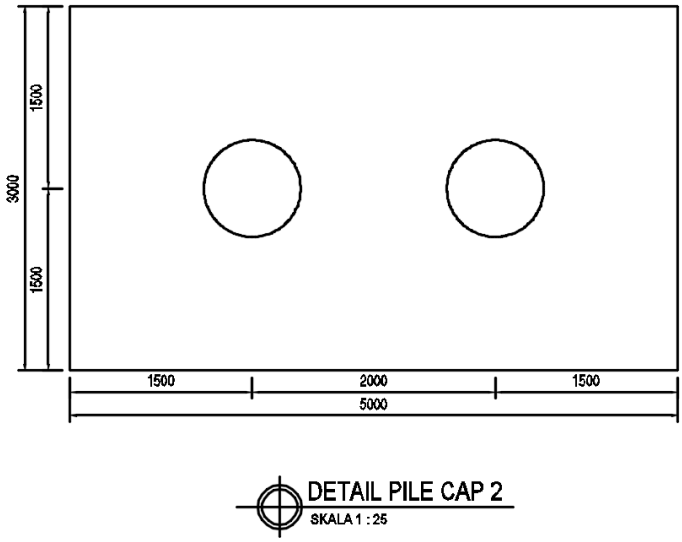
2,5 D $\leq s \leq$ 3 D

200 ≤ s ≤ 240
Maka dipakai s = 200 cm

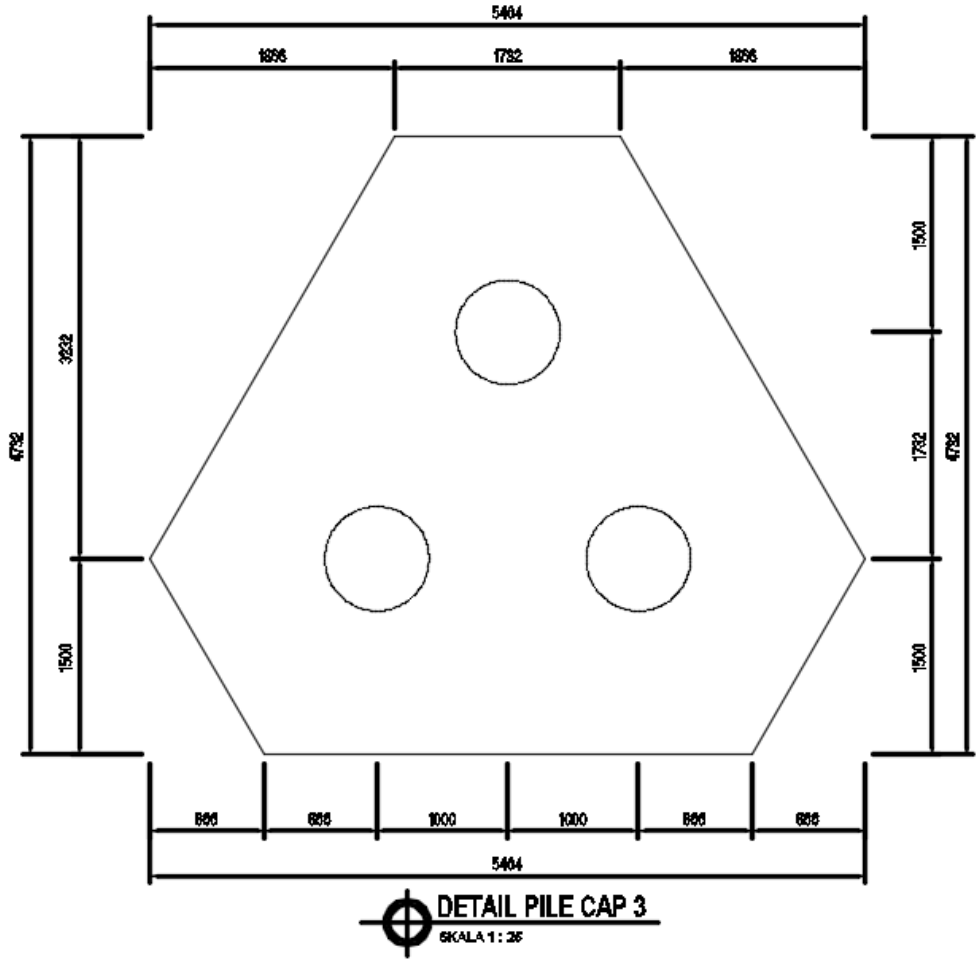
1,5 D ≤ s' ≤ 2 D
120 ≤ s' ≤ 160
Maka dipakai s' = 150 cm



Gambar 4.33 Rencana Pile Cap dengan 1 Tiang Bor Diameter 80 cm



Gambar 4.34 Rencana Pile Cape dengan 2 Bor Diameter 80 cm



Gambar 4.35 Rencana Pile Cape dengan 3 Bor Diameter 80 cm

Direncanakan Tebal Pile Cap 80 cm

4.3 Rancangan Anggaran Biaya

4.3.1 Rancangan Anggaran Biaya Pondasi Tiang Pancang

Tabel 4.25
Rancangan anggaran biaya pondasi tiang pancang

No	Uraian Pekerjaan	Satuan	Volume	Harga Satuan	Total Harga
I	**PEKERJAAN PERSIAPAN**				
1	Pembersihan Lapangan & Pembuangan sampah keluar lokasi	m2	2,133.60	13,250.00	28,270,200.00
2	Pek. Pengukuran dan Pemasangan Bouwplank	m	674.40	50,485.00	34,047,084.00
3	Pek. Air Kerja	ls	1.00	14,570,000.00	14,570,000.00
4	Pek. Listrik Kerja	ls	1.00	16,500,000.00	16,500,000.00
5	Sewa Mobile Crane + Mob Demob	bln	3.00	66,800,000.00	200,400,000.00
SUB TOTAL I.					**293,787,284.00**
II	**PEKERJAAN TANAH**				
1	Pek. Galian Tanah Pile Cap P1	m3	10.40	62,625.00	651,300.00
2	Pek. Galian Tanah Pile Cap P2	m3	14.40	62,625.00	901,800.00
3	Pek. Galian Tanah Pile Cap P3	m3	7.77	62,625.00	486,797.65
4	Pek. Galian Tanah Pile Cap P4	m3	6.40	62,625.00	400,800.00
5	Pek. Galian Tanah Pile Cap P5	m3	34.83	62,625.00	2,181,009.31
6	Pek. Galian Tanah Pile Cap P6	m3	19.20	62,625.00	1,202,400.00
7	Pek. Galian Tanah Pile Cap P7	m3	156.35	62,625.00	9,791,479.87
9	Pek. Galian Tanah Pile Cap P10	m3	17.35	62,625.00	1,086,660.98
8	Pek. Pasir Urug t = 20 cm diatas Pile Cap P1	m3	2.18	133,050.00	290,581.20
9	Pek. Pasir Urug t = 20 cm diatas Pile Cap P2	m3	3.31	133,050.00	440,661.60
10	Pek. Pasir Urug t = 20 cm diatas Pile Cap P3	m3	4.58	133,050.00	609,901.20
11	Pek. Pasir Urug t = 20 cm diatas Pile Cap P4	m3	1.40	133,050.00	186,802.20
12	Pek. Pasir Urug t = 20 cm diatas Pile Cap P5	m3	7.92	133,050.00	1,054,105.12
13	Pek. Pasir Urug t = 20 cm diatas Pile Cap P6	m3	4.41	133,050.00	586,484.40
14	Pek. Pasir Urug t = 20 cm diatas Pile Cap P7	m3	36.74	133,050.00	4,887,690.74
16	Pek. Pasir Urug t = 20 cm diatas Pile Cap P10	m3	4.14	133,050.00	551,088.84
15	Pek. Lantai Kerja t = 5 cm dibawah Pile Cap P1	m3	0.65	660,060.00	429,039.00
16	Pek. Lantai Kerja t = 5 cm dibawah Pile Cap P2	m3	0.90	660,060.00	594,054.00
17	Pek. Lantai Kerja t = 5 cm dibawah Pile Cap P3	m3	1.44	660,060.00	950,486.40
18	Pek. Lantai Kerja t = 5 cm dibawah Pile Cap P4	m3	0.40	660,060.00	264,024.00
19	Pek. Lantai Kerja t = 5 cm dibawah Pile Cap P5	m3	2.18	660,060.00	1,436,723.56
20	Pek. Lantai Kerja t = 5 cm dibawah Pile Cap P6	m3	1.20	660,060.00	792,072.00
21	Pek. Lantai Kerja t = 5 cm dibawah Pile Cap P7	m3	9.77	660,060.00	6,450,064.08
23	Pek. Lantai Kerja t = 5 cm dibawah Pile Cap P10	m3	1.08	660,060.00	715,829.79
25	Pek. Pasir Urug t = 10 cm dibawah Pile Cap P1	m3	1.30	133,050.00	172,965.00
27	Pek. Pasir Urug t = 10 cm dibawah Pile Cap P2	m3	1.80	133,050.00	239,490.00
29	Pek. Pasir Urug t = 10 cm dibawah Pile Cap P3	m3	2.88	133,050.00	383,184.00
31	Pek. Pasir Urug t = 10 cm dibawah Pile Cap P4	m3	0.80	133,050.00	106,440.00
33	Pek. Pasir Urug t = 10 cm dibawah Pile Cap P5	m3	4.35	133,050.00	579,208.16
35	Pek. Pasir Urug t = 10 cm dibawah Pile Cap P6	m3	2.40	133,050.00	319,320.00
37	Pek. Pasir Urug t = 10 cm dibawah Pile Cap P7	m3	19.54	133,050.00	2,600,312.17
39	Pek. Pasir Urug t = 10 cm dibawah Pile Cap P10	m3	2.17	133,050.00	288,583.32
SUB TOTAL II.					**41,631,358.60**
III	**PEKERJAAN STRUKTUR BAWAH**				
1	Mob demob peralatan pancang	unit	1.00	28,669,000.00	28,669,000.00
2	Memancang tiang pancang beton bertulang	m'	330.00	292,383.16	96,486,442.14
3	Jumlah tiang pancang	unit	275.00	3,800,000.00	1,045,000,000.00
4	Memotong Tiang Pancang (40x40) cm Beton Bertulang	unit	91.67	197,737.80	18,125,965.00
5	Menyambung Tiang Pancang Beton Bertulang	unit	183.33	866,382.93	158,836,870.13
6	Uji beban langsung	ttk	2.00	17,000,000.00	34,000,000.00
7	Test PDA (Pile Driving Analysis)	ttk	2.00	12,500,000.00	25,000,000.00
8	Pek. Beton Pile Cap P1 K300	m3	10.40	2,597,000.00	27,008,800.00
9	Pek. Beton Pile Cap P2 K300	m3	14.40	2,597,000.00	37,396,800.00
10	Pek. Beton Pile Cap P3 K300	m3	7.77	2,597,000.00	20,187,041.95
11	Pek. Beton Pile Cap P4 K300	m3	6.40	2,597,000.00	16,620,800.00
12	Pek. Beton Pile Cap P5 K300	m3	34.83	2,597,000.00	90,444,410.11
13	Pek. Beton Pile Cap P6 K300	m3	19.20	2,597,000.00	49,862,400.00
14	Pek. Beton Pile Cap P7 K300	m3	156.35	2,597,000.00	406,043,484.67
16	Pek. Beton Pile Cap P10 K300	m3	17.35	2,597,000.00	45,062,811.58
SUB TOTAL III.					**2,098,744,825.59**

IV	**PEKERJAAN STRUKTUR ATAS**				
a	**LANTAI 1**				
1	Pek. Beton Tie Beam TB1 K300	m3	73.87	3,508,000.00	259,142,976.00
2	Pek. Beton Tie Beam TB2 K300	m3	37.32	4,092,000.00	152,718,555.00
3	Pek. Beton Tie Beam TB3 K300	m3	36.43	3,171,000.00	115,525,872.00
4	Pek. Beton Kolom K1 K300	m3	98.00	5,807,000.00	569,086,000.00
5	Pek. Beton Kolom K2 K300	m3	24.50	5,090,000.00	124,705,000.00
6	Pek. Beton Kolom K3 K300	m3	21.25	5,238,000.00	111,307,500.00
7	Pek. Beton Kolom K4 K300	m3	7.20	5,337,000.00	38,426,400.00
8	Pek. Beton Tangga K300	m3	6.13	4,711,000.00	28,878,057.83
9	Pek. Beton Balok Bordes dan Tangga K300	m3	4.46	4,749,000.00	21,192,412.50
8	Pek. Plat Beton dengan Wiremesh m8 , t=10 cm	m3	156.75	1,486,000.00	232,929,014.00
9	Pek. Beton Balok B1 K300 Elv. -0.05	m3	22.18	5,466,000.00	121,214,016.00
10	Pek. Beton Balok BA K300 Elv. -0.05	m3	5.76	4,996,000.00	28,776,960.00
11	Pek. Beton Plat Lantai t = 120 mm K300	m3	20.74	3,870,000.00	80,248,320.00
SUB TOTAL IV					**1,884,151,083.33**
BIAYA TOTAL					**4,318,314,551.53**
				PPn 10%	**431,831,455.15**
				Biaya + PPn	**4,750,146,006.68**
				Pembulatan	**4,750,140,000.00**

4.3.2 Rancangan Anggaran Biaya Pondasi Tiang Bor

Tabel 4.26
Rancangan anggaran biaya pondasi tiang bor

No	Uraian Pekerjaan	Satuan	Volume	Harga Satuan	Total Harga
I	**PEKERJAAN PERSIAPAN**				
1	Pembersihan Lapangan & Pembuangan sampah keluar lokasi	m2	2,133.60	13,250.00	28,270,200.00
2	Pek. Pengukuran dan Pemasangan Bouwplank	m	674.40	50,485.00	34,047,084.00
3	Pek. Air Kerja	ls	1.00	14,570,000.00	14,570,000.00
4	Pek. Listrik Kerja	ls	1.00	16,500,000.00	16,500,000.00
5	Sewa Mobile Crane + Mob Demob	bln	3.00	66,800,000.00	200,400,000.00
SUB TOTAL I.					**293,787,284.00**
II	**PEKERJAAN TANAH**				
1	Pek. Galian Tanah Pile Cap P1	m3	92.80	62,625.00	5,811,600.00
2	Pek. Galian Tanah Pile Cap P2	m3	112.00	62,625.00	7,014,000.00
3	Pek. Galian Tanah Pile Cap P3	m3	228.82	62,625.00	14,330,042.88
4	Pek. Pasir Urug t = 10 cm Bawah Pile Cap P1	m3	11.60	133,050.00	1,543,380.00
5	Pek. Pasir Urug t = 10 cm Bawah Pile Cap P2	m3	14.00	133,050.00	1,862,700.00
6	Pek. Pasir Urug t = 10 cm Bawah Pile Cap P3	m3	28.60	133,050.00	3,805,613.18
7	Pek. Lantai Kerja t = 5 cm Bawah Pile Cap P1	m3	5.80	660,060.00	3,828,348.00
8	Pek. Lantai Kerja t = 5 cm Bawah Pile Cap P2	m3	7.00	660,060.00	4,620,420.00
9	Pek. Lantai Kerja t = 5 cm Bawah Pile Cap P3	m3	14.30	660,060.00	9,439,808.49
10	Pek. Galian Tanah Tie Beam TB1	m3	73.87	62,625.00	4,626,234.00
11	Pek. Galian Tanah Tie Beam TB2	m3	37.32	62,625.00	2,337,243.28
12	Pek. Galian Tanah Tie Beam TB3	m3	36.43	62,625.00	2,281,554.00
13	Pek. Pasir Urug t = 10 cm Bawah Tie Beam TB1	m3	12.31	133,050.00	1,638,111.60
14	Pek. Pasir Urug t = 10 cm Bawah Tie Beam TB2	m3	7.46	133,050.00	993,118.46
15	Pek. Pasir Urug t = 10 cm Bawah Tie Beam TB3	m3	3.17	133,050.00	421,502.40
16	Pek. Lantai Kerja t = 5 cm Bawah Tie Beam TB1	m3	6.16	660,060.00	4,063,329.36
17	Pek. Lantai Kerja t = 5 cm Bawah Tie Beam TB2	m3	3.73	660,060.00	2,463,426.43
18	Pek. Lantai Kerja t = 5 cm Bawah Tie Beam TB3	m3	1.58	660,060.00	1,045,535.04
19	Pek. Urugan Tanah dan Pemadatan Untuk Peninggian Elevasi	m3	1,020.00	122,175.00	124,618,500.00
SUB TOTAL II.					**196,744,467.12**

III	**PEKERJAAN STRUKTUR BAWAH**				
1	Mob demob peralatan bor	unit	1.00	28,669,000.00	28,669,000.00
2	Pengeboran dia. 600 mm - 19 m	m	2,755.00	281,091.75	774,407,771.25
3	Pengeboran dia. 600 mm - 9 m	m	180.00	281,091.75	50,596,515.00
4	Pek. Beton Borepile K300	m3	829.85	1,667,000.00	1,383,362,782.98
5	Uji beban langsung	ttk	2.00	17,000,000.00	34,000,000.00
6	Test PDA (Pile Driving Analysis)	ttk	2.00	12,500,000.00	25,000,000.00
7	Pek. Beton Pile Cap P1 K300	m3	92.80	2,597,000.00	241,001,600.00
8	Pek. Beton Pile Cap P2 K300	m3	112.00	2,548,000.00	285,376,000.00
9	Pek. Beton Pile Cap P3 K300	m3	228.82	2,498,000.00	571,599,953.92
10	Pek. Beton Lantai Pit Lift t=25 cm K300	m3	2.53	4,704,000.00	11,907,000.00
11	Pek. Beton Dinding Pit Lift t=20 cm K300	m3	9.16	4,605,000.00	42,200,220.00
SUB TOTAL III.					**3,448,120,843.15**
IV	**PEKERJAAN STRUKTUR ATAS**				
a	**LANTAI 1**				
1	Pek. Beton Tie Beam TB1 K300	m3	73.87	3,508,000.00	259,142,976.00
2	Pek. Beton Tie Beam TB2 K300	m3	37.32	4,092,000.00	152,718,555.00
3	Pek. Beton Tie Beam TB3 K300	m3	36.43	3,171,000.00	115,525,872.00
4	Pek. Beton Kolom K1 K300	m3	98.00	5,807,000.00	569,086,000.00
5	Pek. Beton Kolom K2 K300	m3	24.50	5,090,000.00	124,705,000.00
6	Pek. Beton Kolom K3 K300	m3	21.25	5,238,000.00	111,307,500.00
7	Pek. Beton Kolom K4 K300	m3	7.20	5,337,000.00	38,426,400.00
8	Pek. Plat Beton dengan Wiremesh m8 , t=10 cm	m3	156.75	1,486,000.00	232,929,014.00
9	Pek. Beton Balok B1 K300 Elv. -0.05	m3	22.18	5,466,000.00	121,214,016.00
10	Pek. Beton Balok BA K300 Elv. -0.05	m3	5.76	4,996,000.00	28,776,960.00
11	Pek. Beton Plat Lantai t = 120 mm K300	m3	20.74	3,870,000.00	80,248,320.00
SUB TOTAL IV.a					**1,834,080,613.00**
BIAYA TOTAL					**5,772,733,207.27**
				PPn 10%	**577,273,320.73**
				Biaya + PPn	**6,350,006,527.99**
				Pembulatan	**6,350,000,000.00**