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BEARING CAPACITY AND SETTLEMENT ANALYSIS OF BOREPILE USING MAYERHOF AND VESIC METHODS FOR LRT JAKARTA PROJECT PHASE 1B

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ABSTRACT

This study presents a comprehensive analysis of the bearing capacity and settlement behavior of bored pile foundations for the Jakarta LRT Phase 1B project, specifically at Pier P64B. Given the challenging soil conditions characterized by soft to stiff clay, silty clay, and sandy silt layers with variable Standard Penetration Test (SPT) values, a deep foundation system using bored piles was adopted to ensure structural stability and minimize excessive settlements. The investigation employs both Mayerhof's and Vesic's empirical methods to evaluate pile bearing capacity, integrating site-specific soil parameters and load combinations modeled in SAP2000 in accordance with Indonesian National Standards (SNI) and American Concrete Institute (ACI) regulations. The analysis includes assessments of single pile and group pile capacities, elastic and consolidation settlements, and pile cap design. Results reveal that while Vesic's method predicts slightly higher pile capacity than Mayerhof's, both methods confirm the adequacy of bored piles with a 0.6 m diameter and 23 m depth for safely supporting the applied loads, with settlements maintained below the allowable limit of 0.16 m. Group pile efficiency and load distribution were also optimized, suggesting 20 piles as the most cost-effective and structurally sound configuration. Construction methodology and reinforcement detailing were developed accordingly. The total estimated foundation cost is approximately IDR 580 million, reflecting improved efficiency over prior designs. This study contributes practical insights for foundation design under similar geotechnical contexts and serves as a reference for ensuring foundation safety and cost-effectiveness in heavy infrastructure projects.

***Keywords** : bearing capacity, bored pile; settlement; Vesic; Mayerhof; LRT*

1. BACKGROUND

The Jakarta LRT Phase 1B (Velodrome–Manggarai) project requires robust foundation solutions due to challenging soil conditions—predominantly clay, silty clay, sandy silt, and organic clay with varying consistency (SPT N-values from 2–4 up to >30). These soils exhibit low to high plasticity and are prone to settlement, making shallow foundations unsuitable for the heavy loads of the viaduct structure. Bore piles are selected to ensure adequate load transfer to deeper, more competent strata, minimize settlement, and enhance structural stability.

"The planned foundation used bored pile because the soil condition and the total construction load are not suitable for a shallow foundation. A suitable dimention design is needed to enhance effectiveness in financing" (Robert Evander, 2020)

A case of significant settlement at bore pile P64B highlighted the importance of accurate bearing capacity and settlement analysis. To ensure safety and performance, the foundation design is reassessed using the Mayerhof and Vesic methods, both well-established for pile capacity and settlement estimation

The Meyerhof and Vesic methods are widely recognized and reliable approaches for calculating the bearing capacity of single piles as well as settlement estimations, both theoretically and numerically, as demonstrated in bridge and infrastructure projects (Bagas Irham Azzahri et al., 2024).

2. METHOD

2.1 Design Load

The loading combinations in this structural design analysis refer to standards such as SNI 1725:2016, SNI 1726:2012, and ACI 343.1R-12, incorporating both static and dynamic loads including dead loads, superimposed dead loads, live loads from LRT vehicles, impact loads, wind loads, braking forces, hunting forces, temperature effects, collision loads, and differential settlement. Seismic analysis for the Jakarta LRT Phase 1B (Velodrome–Manggarai) project follows SNI 2833:2016, with site-specific seismic parameters of Ss = 0.7g and S1 = 0.3g. The analysis also considers directional ground motion effects, response spectrum amplification, and structural response modification factors (R) based on soil classification and structural configuration.

2.2 Bearing Capacity for Borepile

This research for estimate the bearing capacity of bore pile by compare Mayerhof's method and Vesic's method, Then the Ultimate bearing capacity by following Reese and Wright (1977) equation.

$$Q_u = Q_p + Q_s \quad (1)$$

Bore Piles Bearing Capacity based Mayerhoff Method

Based on the result of SPT (Standard Penetration Test) Mayerhof (1976) empirical approach to calculating pile bearing capacity, on cohesive soil (clay) and non-cohesive soil (sand and gravel). This equation is based on bearing capacity (Qp).

a. Bearing Capacity at cohesionless soil (sand)

$$Q_p = A_p \; x \; q' x \; N_q^* \leq A_p \; x \; q_1 \quad (2)$$

b. Bearing Capacity at cohesive soil (clay)

$$Q_p = N_c^* \; x \; c_u \; x \; A_p \approx 9 \; x \; C_u \; x \; A_p \quad (3)$$

Qp	=	Bearing Capacity (kN)
Qs	=	Frictional resistance (kN)
Ap	=	Pile cross-sectional area
N60SPT	=	Number of strokes required from the SPT test
$\overline{(N60)}$	=	(N1 +N2)/2
Li	=	Depth of soil (m)
D	=	Diameter of Pile (m)
P	=	Perimeter of pile (m)
Cu	=	Undrained Cohesion (kN/m2) = NSPT x 2/3 x 10
α	=	Adhesion coefficient between soil and pile

Bore Piles Capacity based on Vesic 1977

Vesic (1977) developed a method to estimate pile tip bearing capacity using cavity expansion theory with effective stress parameters

a. Bearing Capacity at cohesionless soil (sand)

$$Q_p = A_p \; x \; q_p = A_p \; x \; \overline{\sigma}'_o \; x \; N_{\sigma}^* \quad (4)$$

b. Bearing Capacity at cohesive soil (clay)

$$Q_p = A_p q_p = A_p C_u N_c^* \quad (5)$$

Frictional (Skin) Resistance in Sand

The Frictional resistance in sand can be determined by using the formula

$$Q_s = p \; x \; \Delta Li \; x \; f_{av} \quad (6)$$

Meyerhof (1976) showed that average unit frictional resistance for high-displacement piles can be derived from average standard penetration resistance.

$$f_{av} = 0{,}02 \; x \; p_a \; x \; \overline{N60} \quad (7)$$

$\overline{\sigma}'_o$ = mean effective vertical stress for the entire embedment length
c_u = mean undrained shear strength (ø = 0)
p = perimeter of the cross section of each pile
Li = thickness of soil layer
fav = average unit frictional resistance

Frictional Resistance (Qs) in Clay

The Frictional resistance in clay can be determined by using the formula (6) for f_{av} calculation use α method

$$f = \alpha c_u \quad (8)$$

The approximate variation of α based on Terzaqhi et al., 1996

2.3 Elastic Settlement of Group Piles

Vesic (1977) separated the total settlement se, of the pile into three components and suggested expression to determine them from elastic analysis.

a. Elastic shortening of the pile (se(1))

$$s_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p} \quad (9)$$

b. Settlement of the pile due to the working load (Qwp) at the pile point (se(2))

$$s_{e(2)} = \frac{Q_{wp} C_P}{D q_p} \quad (10)$$

c. Settlement of the pile due to the working load (Q_{ws}) along the pile shaft ($s_{e(3)}$)

$$S_{e(3)} = \frac{Q_{wp} C_s}{L q_p} \quad (11)$$

2.4 Consolidation Settlement of Group Piles

Consolidation settlement of a group pile, especially in clay, is a critical aspect of foundation design. The consolidation settlement of a group pile in clay can be estimated by using the 2:1

a. Determine the pile embedment (L) and the total load on the pile group (Qg) minus any soil removed above the pilecap.

b. Equivalent raft concept, assumed the load is distributed to the soil at 2/3L from top of the pile (Terzaqhi and Peck's method)

c. Calculate Stress Increase

$$\Delta\sigma'_i = \frac{Q_g}{(B_g + z_i)(L_g + z_1)} \quad (12)$$

d. Compute settlement for each layer

$$\Delta S_{c(i)} = \frac{C_{c(i)} H_i}{1 + e_{0(i)}} log\left(\frac{\sigma'_{0(i)} + \Delta\,\sigma'_{0(i)}}{\sigma'_{0(i)}}\right) \quad (13)$$

e.The total of consolidation settlement of pile group is the sum of settlements for all affected layers.

$$\Delta S_{c(g)} = \sum \Delta S_{c(i)} \quad (14)$$

3. RESULT AND DISCUSSION

a. The upper structure design at viaduct LRT Jakarta Phase 1B on point P63B-P64B by using SAP2000 is shown at figure

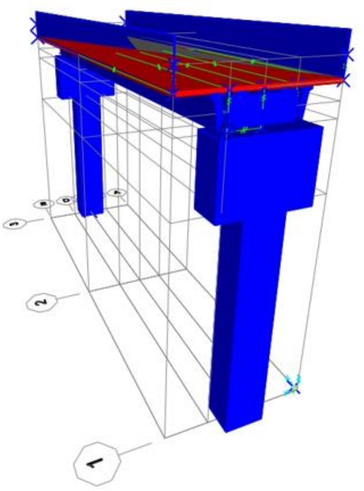


Figure 1. Viaduct upper structure of The Jakarta LRT Phase 1B span P63B-P64B
Resources: SAP2000 Analisis

The output from SAP2000 analysis of the load combination design based on ACI 343.1R-12 is

Axial Load (Qv)	=	18488,45183	kN
Momen x	=	302,930	kN
Momen y	=	19043,81	kN

b. Bearing capacity of a single pile on P64B by using mayerhoff and vesic method. the various depth is 23 m 29 and 41 m m with diameter 0,6 m

$$A_p = \frac{1}{4}\pi D^2 = \frac{1}{4}\pi 0{,}6^2 = 0{,}282\ m^2$$

Mayerhoff method

$$Q_p = 9\ x\ C_u\ x\ A_p = (9)(228)(0{,}25\ x\ \pi\ x\ 0{,}6^2)$$
$$= 580{,}19\ kN$$

Vesic Method

$$Q_p = A_p C_u N_c^* = (0{,}25\ x\ \pi\ x\ 0{,}6^2)(228)(11{,}51)$$
$$= 742\ kN$$

The frictional resistance of single pile on P64B. For clay soil there are three method, but on this calculation, α method is used to find the frictional resistance.

$$f = \alpha c_u = (0{,}57)(228) = 78{,}20$$

$$Q_s = p\ x\ \Delta Li\ x\ f_{av} = (\pi\ x\ 0{,}6)(2)(78{,}20)$$
$$= 294{,}82\ kN$$

Estimation of pile capacity and number of pile, the value of Qp and Qs is commulative from the first soil layer with the safety factor is 4

$$Q_{ult} = Q_p + Q_s = 580{,}19 + 3759{,}91 = 4340{,}10\ kN$$

$$Qall = \frac{Qult}{SF} = \frac{4340{,}10}{4} = 1085{,}03\ kN$$

$$n = \frac{Q_V}{Q_{all}} = \frac{18488{,}452}{1085{,}03} = 17\ piles$$

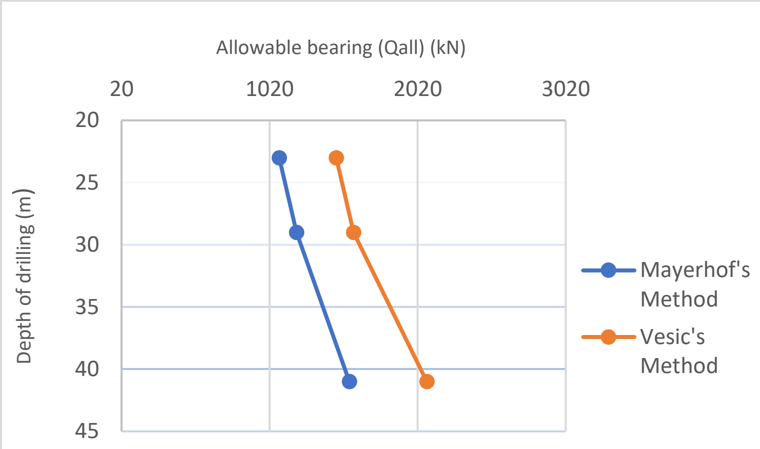
There is the recapitulation of bearing capacity calculation of single pile

Tabel 1. *Recapitulation bearing capacity on single pile by using mayerhof and vesic method*

No	Depth	Layer Thickness	Description	N60	Qp	Qs	Qu	Qall	n
	(m)	(m)			kN	kN	kN	kN	
Mayerhoff									
1	23	2	Clay	38	580,19	3759,913	4340,103	1085,026	17
2	29	2	Clay	12	183,218	4623,917	4807,134	1201,784	15
3	41	2	Clay	24	366,435	5868,246	6234,681	1558,670	12
Vesic									
1	23	2	Clay	38	742,00	3668,780	4410,777	1470,259	13
2	29	2	Clay	12	226,51	4532,783	4759,293	1586,431	12
3	41	2	Clay	24	468,63	5777,112	6245,742	2081,914	9

Resources: Estimation Analysis

Graph 1. *Comperassion result of Qall between Mayerhof's method and Vesic's method*



Resources: Estimation Analysis

Based on the results shown in Table 1 and Graph 1, the conclusion is that Vesic's method consistently yields higher allowable bearing capacities for clay piles compared to Meyerhof's, due to greater calculated end bearing and skin friction. Both methods show decreasing capacity with depth, but Meyerhof's conservative estimates prioritize safety, while Vesic's approach offers optimized design potential.

c. Bearing capacity of group piles Because at LRT Project the bore pile is used to carry the huge self-load (Qv =18488,452 kN) to make sure it is safe, the piles are used in group.

- Design sapcing of pilecap

$$S_{min} = 2{,}5\ x\ D = 2{,}5\ x\ (0{,}6) = 1{,}5\ m$$

$$S_{max} = 3{,}5\ x\ D = 3{,}5\ x\ (0{,}6) = 2{,}1\ m$$

$$S_{edges} = 1{,}5\ x\ D = 3{,}5\ x\ (0{,}6) = 0{,}9\ m$$

- Calculate Pilecap Dimention

$$L_g = (m-1)\ x\ s + D = (4-1)x\ 2{,}1 + 0{,}6 = 6{,}9\ m$$

$$B_g = (n-1)\ x\ s + D = (4-1)x\ 2{,}1 + 0{,}6 = 6{,}9\ m$$

$$L_c = (m-1)\ x\ s + 2\ x\ s\ edge$$

$$= (4-1)x\ 2{,}1 + 2\ x\ 0{,}9 = 8{,}1\ m$$

$$W_c = (n-1)\ x\ s + 2\ x\ s\ edge$$

$$= (4-1)x\ 2{,}1 + 2\ x\ 0{,}9 = 8{,}1\ m$$

$$t_c = \frac{1}{3}\ x\ (8D + 600) = \frac{1}{3}x(8(0{,}6) + 600) = 2\ m$$

- Bearing capacity of groups piles

$$\theta = TAN^{-1}\left(\frac{D}{s}\right) = TAN^{-1}\left(\frac{0{,}6}{2{,}1}\right) = 15{,}945$$

$$Eg = 1 - \theta\ \frac{(n-1)m + (m-1)n}{90mn}$$

$$Eg = 1 - (15{,}945)\frac{\big((4)-1\big)(4) + \big((4)-1\big)(4)}{90(4)(4)}$$

$$= 0{,}7342$$

$$Q_g = E_g\, n\, Q_u$$

$$= (0{,}7342)(16)(29895{,}35) = 534717\ \text{kN}$$

$$Q_{all} = \frac{Q_g}{SF} = \frac{534717}{4} = 133679{,}31\ kN$$

The recapitulation of bearing capacity at group piles is shown at the table 2

Table 2. *Recapitulation bearing capacity on group piles*

Depth Li	Diameneter D	n	Lg	bg	Lc	Wc	tc	Eg	Qg (u)	Q all
(m)	(m)	(piles)	(m)	(m)	(m)	(m)	(m)	(η)	(kN)	(kN)
23	0,6	20	9	6,9	15	9,75	1,8	0,725	390341,412	97585,353
29		16	6,9	6,9	8,1	8,1	1,8	0,734	534717,246	133679,311
41		12	6,9	4,8	8,1	6	1,8	0,749	716951,313	179237,828

Resources: Estimation Analysis

d. The calculation of Elastic Settlement on each variation of pile diameter, and depth.

- Elastic shortening of the pile [se(1)]

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

$$s_{e(1)} = \frac{((8147,90) + (0,67)(3759,91))(23)}{(0,283)(27805575)}$$

$$s_{e(1)} = 0,0312067 \text{ m} = 31,206684 \text{ mm}$$

- Settlement of the pile due to the working load (Qwp) at the pile point [se(2]

$$s_{e(2)} = \frac{Q_{wp} C_P}{D q_p}$$

$$s_{e(2)} = \frac{(8147,90)(0,06)}{(0,6)(28817,31)}$$

$$s_{e(2)} = 0,0282743 \text{ m} = 28,274334 \text{ mm}$$

- Settlement of the pile due to the working load (Qws) along the pile shaft [se(3)]

$$C_s = \left(0,93 + 0,16\sqrt{\frac{L}{D}}\right) C_p$$

$$Cs = \left(0,93 + 0,16\sqrt{\frac{(23)}{(0,6)}}\right)(0,06)$$

C_s = 0,115237

$$S_{e(3)} = \frac{Q_{wp} C_s}{L q_p}$$

$$S_{e(3)} = \frac{(8147,90)(0,115237)}{(230)(28817,31)}$$

$$S_{e(3)} = 0,0006537 \text{ m} = 0,65371 \text{ mm}$$

-Total Settlement for single pile

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

$$s_e = 31,206 + 28,274 + 0,653$$

$$s_e = 60,134 \text{ mm}$$

e. Elastic Settlement of Group Piles based on Mayerhof's Method

$$\overline{N_{60}} = 39,47$$

$$I = 1 - \frac{L}{8 B_g} > 0,5$$

$$I = 1 - \frac{(23)}{8\,(6,9)} = -18,84 < 0,5 \; the\ use\ 0,5$$

$$q = \frac{Q_g}{L_g B_g} = \frac{(18488,45)}{(9)(6,9)} = 297,72$$

$$S_{g(e)} = \frac{0,69\, q\sqrt{B_g I}}{\overline{N_{60}}} = \frac{0,69\,(297,72)\sqrt{(6,9)(0,5)}}{(39,47)}$$

$$S_{g(e)} = 14,53\ mm$$

$$S_{g(e)} = 0,015\ m$$

- Control maximum settlement based on SNI

$$S_{gall} = 15cm + \frac{b}{600} = 15cm + \frac{810}{600} = 16,35\ cm$$

The consolidation elastic settlement at group piles is lower than control, the maximum settlement based on SNI is $15cm + \frac{b}{600}$ and the value of b for the viaduct is wide of the pilecap, and for building is wide of the building itself. That means the consolidation is safe. The recapitulation of elastic settlement estimation is shown in Table 3. Besides estimating the elastic settlement, it need to estimate consolidation settlement if the soil condition is dominated by clay soil.

Table 3. *Recapitulation elastic settlement*

RECAPITULATION DATA FOR ELASTIC SETTLEMENT					
Depth	**Diamter**	**Single pile consolidation**	**Group pile consolidation**	**sg All**	**Checklist**
23	0,6	0,060	0,015	0,16	OK
29		0,079	0,010	0,16	OK
41		0,115	0,014	0,16	OK

Resources: Estimation Analysis

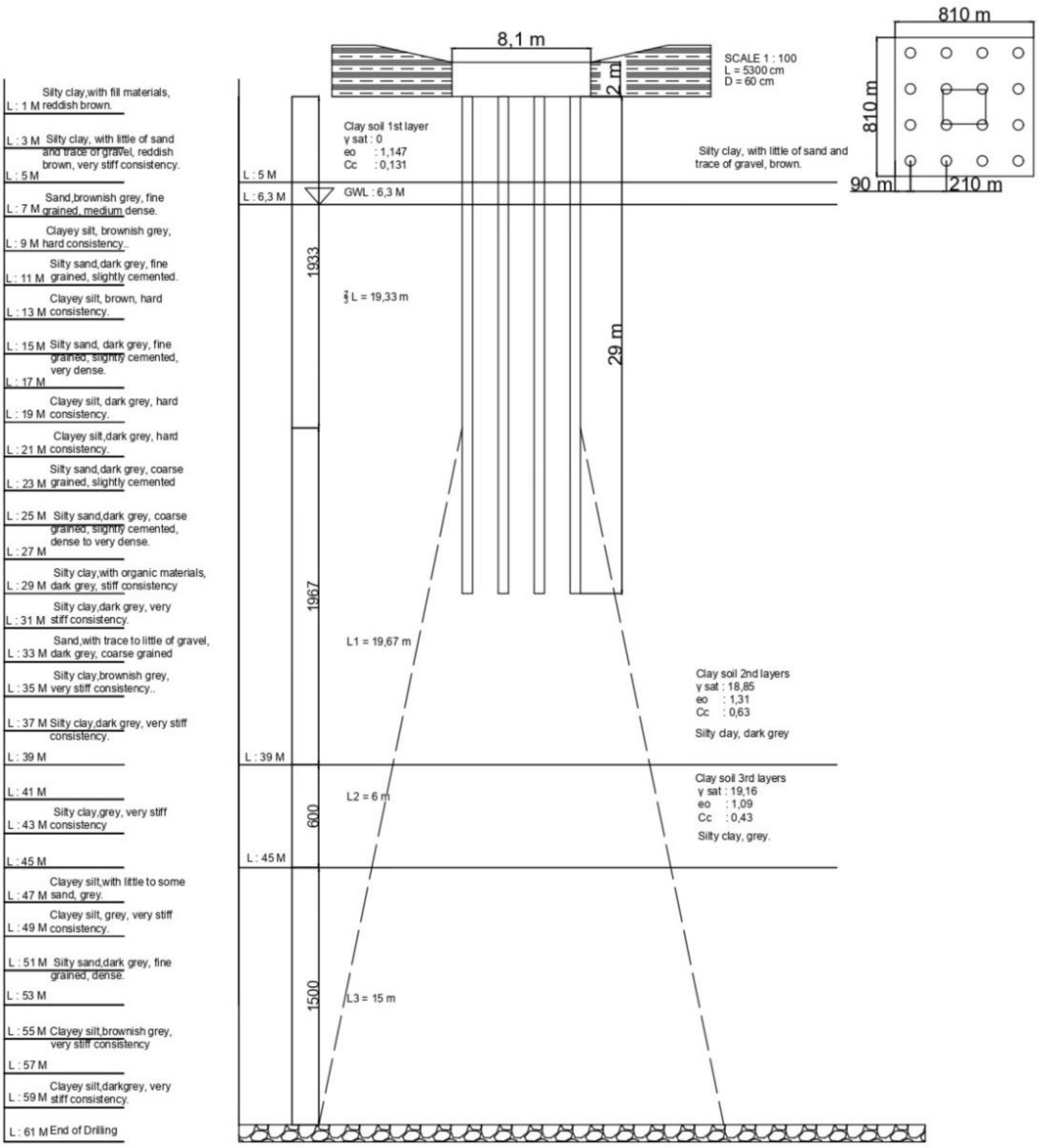


Figure 2. Consolidation Settlement
Resources: CAD Analisis

f. Consolidation Settlement of Group Pile
There are several variations of estimation consolidation at group piles, depending on their diameter (0,6 m) and depth (23m, 29m, and 41m). A group pile with D = 0,6 m and L = 29 m is shown in Figure 4.10.

- First Layer

  L_1 = 6,3m and $\gamma\ sat$ = 0 kN/m^3

- Second Layer

$$\Delta\sigma_2' = \frac{Q_g}{(B_g + z_2)(L_g + z_2)}$$

$$\Delta\sigma_2' = \frac{(18488{,}45)}{((6{,}9) + (9{,}835))((6{,}9) + (9{,}835))}$$

$$\Delta\sigma_2' = 16{,}28\ kN/m^2$$

$$\sigma_2 = (L_1\ x\ \gamma\ dry) + \left(\left(\frac{2}{3}L - L_1\right)x(\gamma\ sat - \gamma\ water)\right)$$

$$= (6{,}3\ x\ 12) + \left(\left(\frac{2}{3}29 - 6{,}3\right)x(18{,}85 - 9{,}81)\right)$$

$$= 193{,}42\ kN/m^2$$

$$\Delta S_{c(2)} = \frac{C_{c(2)}H_2}{1 + e_{0(2)}} log\left(\frac{\sigma_{0(2)}' + \Delta\,\sigma_{0(2)}'}{\sigma_{0(2)}'}\right)$$

$$= \frac{(0{,}131)(19{,}67)}{1 + (1{,}147)} log\left(\frac{(193{,}42) + (16{,}28)}{(193{,}42)}\right)$$

= 0,042 m

- Third Layer

$$\Delta\sigma_3' = \frac{Q_g}{(B_g + z_3)(L_g + z_3)}$$

$$\Delta\sigma_3' = \frac{(18488{,}45)}{((6{,}9) + (22{,}67))((6{,}9) + (22{,}67))}$$

$$\Delta\sigma_3' = 4{,}00\ kN/m^2$$

$$\sigma_3 = (L_1\ x\ \gamma\ dry) + ((L_2 - L_1)x(\gamma\ sat_2 - \gamma\ water)) + \left(\left(\frac{h}{2}\right)x(\gamma\ sat_3 - \gamma\ water)\right)$$

$$= (6{,}3\ x\ 12) + ((39 - 6{,}3)x(18{,}85 - 9{,}81)) + \left(\left(\frac{6}{2}\right)x(19{,}16 - 9{,}81)\right)$$

$$= 339{,}26\ kN/m^2$$

$$\Delta S_{c(3)} = \frac{C_{c(3)}H_3}{1 + e_{0(3)}} log\left(\frac{\sigma_{0(3)}' + \Delta\ \sigma_{0(3)}'}{\sigma_{0(3)}'}\right)$$

$$= \frac{(0{,}63)(6)}{1 + (1{,}310)} log\left(\frac{(339{,}26) + (4{,}00)}{(339{,}26)}\right)$$

= 0,181 m

- Fourth Layer

$$\Delta\sigma_4' = \frac{Q_g}{(B_g + z_4)(L_g + z_4)}$$

$$\Delta\sigma_4' = \frac{(18488{,}45)}{((6{,}9) + (13{,}5))((6{,}9) + (13{,}5))}$$

$$\Delta\sigma_4' = 9{,}73\ kN/m^2$$

$$\sigma_3 = (L_1\ x\ \gamma\ dry) + ((L_2 - L_1)x(\gamma\ sat_2 - \gamma\ water)) + ((L_3 - (L_1 + L_2))x(\gamma\ sat_3 - \gamma\ water)) + \left(\left(\frac{h_4}{2}\right)x(\gamma\ sat_4 - \gamma\ water)\right)$$

$$\sigma_3 = (6{,}3\ x\ 12) + ((39 - 6{,}3)x(18{,}85 - 9{,}81)) + ((45 - (6{,}3 + 39))x(19{,}16 - 9{,}81)) + \left(\left(\frac{60}{2}\right)x(17{,}16 - 9{,}81)\right)$$

$$\sigma_3 = 423{,}53\ kN/m^2$$

$$\Delta S_{c(4)} = \frac{C_{c(4)}H_4}{1 + e_{0(4)}} log\left(\frac{\sigma_{0(4)}' + \Delta\ \sigma_{0(4)}'}{\sigma_{0(4)}'}\right)$$

$$= \frac{(0{,}63)(15)}{1 + (1{,}310)} log\left(\frac{(423{,}52) + (9{,}73)}{(423{,}52)}\right)$$

= 0,030 m

The consolidation settlement at group piles is lower than control. That is means the consolidation is safe. The recapitulation of the consolidation settlement estimation.

Tabel 4. *Recapitulation of consolidatif settlement*

RECAPITULATION DATA FOR CONSOLIDATION SETTLEMENT OF GROUP PILE					
Depth	**Diamter**	**ΔSc1**	**ΔSc2**	**ΔSc3**	**ΔSc (g)**
(m)	(m)	(m)	(m)	(m)	(m)
23	0,6	0,025	0,006	0,012	0,043
29		0,025	0,004	0,018	0,046
41		0,038	0,014	0,030	0,082

Resources: Estimation Analysi

g. Rainforcement for Pilecap and Borepile
Based on the reinforcement calculation for Pilecap the bottom reinforcement is use D25-45mm and top reinforcement is use D25-40mm with additional sheer reinforcement is D10-200mm, for the borepile main reinforcement is use used 6-D25mm with the tranversal reinforcement is D10-130 mm.

h. Cost Estimation
The total estimated budget for foundation works, including bored pile and pilecap construction, amounts to: IDR 580,115,413.90, ir is based on AHSP DKI Jakarta 2024 with the largest component coming from borehole drilling and ready-mix concrete work.

4. CONSLUSION

1. The load analysis design on the LRT Project Jakarta Velodrome-Manggarai Phase 1B at Point 64B by using SAP2000, has a result that the Fz is equal to 18488,452 kN.

2. The most suitable bored pile design, based on the evaluation of single and group pile bearing capacity, settlement control, and cost efficiency, is a bored pile with a diameter of 0.6 m and depth of 23 m, utilizing 20 piles.
   The final group bearing capacity calculation shows the following values:
   Mayerhoff's method
   - Total capacity (Qu) = 4340,10 kN
   - Allowable capacity (Qall) = 1085,03 kN

   Vesic's method
   - Total capacity (Qu) = 4410,78 kN
   - Allowable capacity (Qall) = 1470,26 kN
3. The total settlement, as a combination of elastic and consolidation settlement, for the optimal design (D = 0.6 m, L = 23 m) is approximately: 0.043 m (consolidation settlement). Elastic settlement and total settlement have been analyzed under several dimension variations. The two-way shear control on the pile cap shows that the maximum shear force (Vu = 739225,5955 tons) exceeds the concrete shear capacity (ϕVc = 139,8954 tons), thus requiring additional shear reinforcement in the pilecap to ensure structural safety.
4. The total estimated budget for foundation works, including bored pile and pilecap construction, amounts to: IDR 580,115,413.90, with the largest component coming from borehole drilling and ready-mix concrete work.

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