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Pile Foundation Design for a 9-Story Office Building (Case Study: Mozia Office, South Tangerang)

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Abstract: Building safety is largely determined by the ability of the foundation to transfer structural loads to soil layers with adequate bearing capacity. Therefore, in conditions where the surface soil is weak, pile foundations are used to reach more stable layers. This study evaluates the performance of pile foundations at the Mozia Office Building, a nine-storey office building in Bumi Serpong Damai, through an analysis of bearing capacity, group efficiency, and foundation settlement based on SPT data and laboratory testing with parameters of N-SPT, soil weight, porosity, and soil shear strength. The foundation capacity was calculated by separating the end bearing capacity using the Meyerhof method and the skin friction using the α method according to McClelland (1974) and Reese and Wright (1977), while the pile group efficiency was reviewed conservatively from several empirical methods and the settlement was analysed using semi-empirical methods and Vesic (1977). The analysis results indicate that the effective soil layer for pile tip placement is at a depth of approximately 10 m, with an ultimate bearing capacity of 9,153.55 kN, a permissible bearing capacity of 3,051.18 kN, and a single pile settlement of 4.43 cm and a pile group settlement of 6.27 cm, which still meet the permissible limits. Therefore, the planned pile foundation is considered safe and suitable for use.

Keywords: pile foundation, foundation bearing capacity, SPT, pile group efficiency, foundation settlement

INTRODUCTION

The strength and stability of a building are greatly influenced by the performance of the foundation in transferring structural loads to the supporting soil. Improper foundation design can cause excessive settlement or structural failure, so the selection of foundation types in civil engineering practice must be adapted to the soil conditions in the field. In surface soils with low bearing capacity, the use of deep foundations is a commonly applied solution.

One type of deep foundation that is widely used is pile foundations, which serve to transfer the load of a building to a hard soil layer at a certain depth. Das (2011) states that piles are used to penetrate weak soil layers so that the load of the structure can be borne by soil layers with higher bearing capacity. Therefore, understanding the bearing capacity of the soil is an important aspect in foundation planning.

Soil bearing capacity indicates the ability of soil to withstand loads without experiencing excessive sliding or subsidence, which is influenced by soil type, hard layer depth, and groundwater conditions. With the advancement of technology, foundation analysis and planning are now supported by civil engineering software such as ETABS and SPColumn, which enable a more detailed evaluation of soil-structure interaction, including settlement analysis, pile group effects, and structural response to load variations.

Based on this background, this study examines the pile foundation design for the Mozia Office Building, a nine-storey office building located in Bumi Serpong Damai, South Tangerang. The study focuses on determining the optimal pile type, analysing maximum bearing capacity, calculating settlement, and evaluating the efficiency of the foundation system in relation to structural loads and local soil conditions, with the aim of contributing to the planning of safe, efficient, and sustainable pile foundation design in civil engineering practice.

LITERATURE REVIEW

The foundation planning analysis in this study was based on soil investigation data obtained from Standard Penetration Test (SPT) results and laboratory testing. This data was used to determine soil parameters that affect foundation behaviour, including N-SPT values, soil bulk density, particle density, porosity, and soil shear strength parameters such as cohesion and internal friction angle. These parameters form the basis for evaluating the characteristics of the supporting soil and its suitability for the structural loads acting on it.

The foundation capacity calculation is performed by separating the end bearing capacity and the friction bearing capacity of the pile skin. The end bearing capacity is analysed using the Meyerhof method, which considers the soil conditions around the pile tip and the depth of placement. Meanwhile, the friction bearing capacity is calculated using the α method with the McClelland (1974) and Reese and Wright (1977) approaches, which relate the magnitude of friction to the shear strength properties of the soil along the pile shaft.

In addition to single pile capacity, foundation performance is also reviewed through pile group efficiency analysis using several empirical methods. To ensure safety levels, the smallest efficiency value is selected as the most conservative condition in the planning. Furthermore, foundation settlement analysis is carried out using semi-empirical methods and the Vesic (1977) method to ensure that the amount of settlement that occurs is still within the permissible limits and does not interfere with the overall performance of the structure.

RESEARCH METHOD

The research method used in the preparation of this final project is descriptive-quantitative with *an* applied *research* approach. The research is described as descriptive-quantitative because it utilises numerical data from soil investigations, such as boring logs and SPT values, to describe the characteristics and bearing capacity of pile foundations at the study site. An applied approach is used because the research results are aimed at producing foundation design recommendations that can be directly applied to the buildings under review.

The soil investigation data obtained was then analysed using analytical methods based on pile bearing capacity theory from literature and applicable standards, and compared with calculations based on field data. The results of this analysis were used to determine the ultimate bearing capacity and allowable bearing capacity of pile foundations, as well as to form the basis for developing foundation design recommendations in accordance with local geotechnical conditions.

The data used in this study is secondary data, which is divided into primary secondary data and supporting secondary data. Primary secondary data includes soil investigation results such as boring logs, SPT data, and soil laboratory test results obtained from consultants or project parties, and is used as the main basis for bearing capacity and foundation settlement analysis. Meanwhile, supporting secondary data is obtained from journals, textbooks, previous reports, and other academic literature that serve to reinforce theory and compare calculation methods.

The research instruments used included technical documents, analysis software, standards and reference literature. The technical documents consisted of boring log and SPT data, structural load data from the planning stage, and site maps and preliminary design drawings used to determine the configuration and distribution of the piles. The software used includes Microsoft Excel for calculating bearing capacity, settlement, and pile group efficiency, as well as ETABS 22 as a source of load and force data in structures.

This study also refers to technical standards such as SNI 8460:2017 and SNI 4153:2008, as well as relevant geotechnical literature as guidelines for analysis and planning. In addition, technical interviews with relevant parties may be conducted if necessary to obtain additional information regarding field conditions or soil investigation procedures, so that the data used can be validated and the analysis results become more comprehensive.

Flowchart

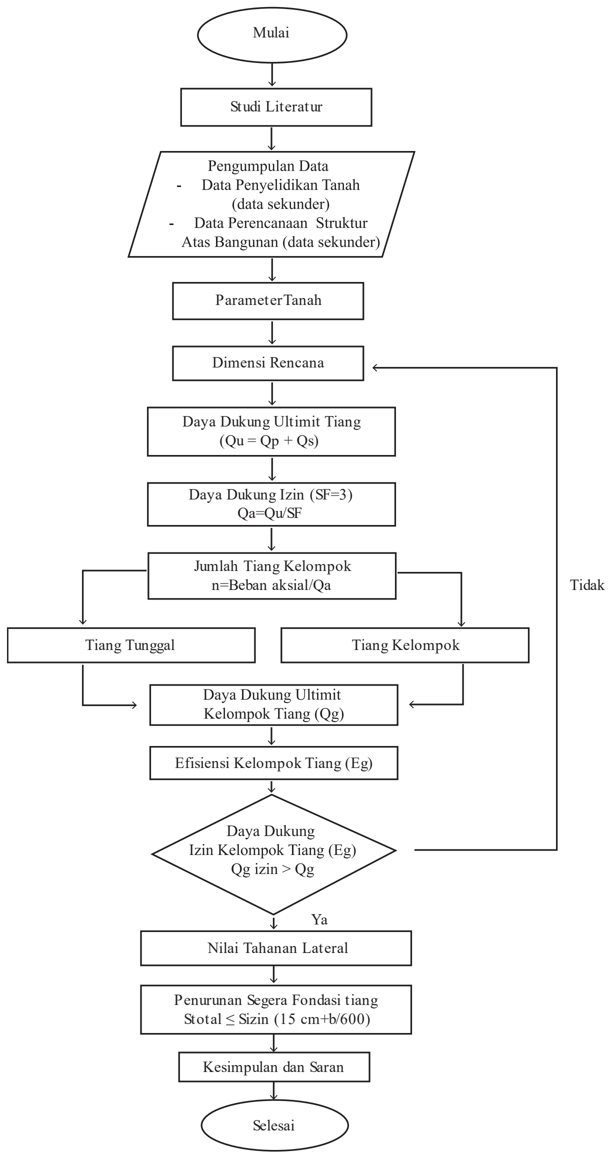


Figure 1. Flowchart
Source: Writer

RESULT AND DISCUSSION

Building Planning Data

The Mozia Office Building is a 9-storey office building located in Bumi Serpong Damai, South Tangerang. The technical data on the axial load of the planned columns for this office building is shown in Table 1. This data is the load output data for each foundation column carried from ETABS, which was provided by the planning consultant. The planned diameter of the columns is 50 cm.

Table 1. Axial load values for columns

Story	Label	Unique Name	Output Case	Case Type	FX tonf	FY tonf	FZ tonf	MX tonf-m	MY tonf-m	MZ tonf-m
Base	1	2	COMB1	Combination	0,8382	2,3097	231,5073	-5,6598	1,3785	0,0422
Base	2	4	COMB1	Combination	1,0382	1,6345	300,2989	-4,8008	1,4474	0,0422
Base	3	6	COMB1	Combination	1,2715	0,2756	314,346	-3,0718	1,5462	0,0422
Base	4	8	COMB1	Combination	0,8822	-3,0888	197,3734	1,2086	0,8613	0,0422
Base	5	10	COMB1	Combination	3,7783	2,8117	344,0927	-6,3838	5,0792	0,0422
Base	6	12	COMB1	Combination	3,5419	2,0121	418,9134	-5,3665	4,5988	0,0422
Base	7	14	COMB1	Combination	3,183	0,8379	519,4057	-3,8727	3,9522	0,0422
Base	8	16	COMB1	Combination	1,8282	-5,2176	305,5315	3,8316	2,0521	0,0422
Base	9	18	COMB1	Combination	-3,4347	0,7467	371,7316	-3,8893	-3,9998	0,0422
Base	10	20	COMB1	Combination	-3,7041	1,6708	401,8534	-5,065	-4,5217	0,0422
Base	11	22	COMB1	Combination	-3,988	1,123	462,7363	-4,3681	-5,0739	0,0422
Base	12	24	COMB1	Combination	-2,3085	-4,8225	275,5575	3,1962	-3,1548	0,0422
Base	13	26	COMB1	Combination	-0,7711	0,9077	253,9422	-4,1796	-0,6472	0,0422
Base	14	28	COMB1	Combination	-0,7364	1,4747	287,1058	-4,9009	-0,7863	0,0422
Base	15	30	COMB1	Combination	-0,757	1,1601	305,8252	-4,5006	-1,0071	0,0422
Base	16	32	COMB1	Combination	-0,6607	-2,8884	182,0269	0,6501	-1,0808	0,0422

Source: Tjendana Structure Consultant

Soil Parameters

In general, the soil profile at this location shows a pattern of increasing soil strength with depth. The upper layer to a depth of approximately 7 metres is dominated by relatively soft clay and silt, while from a depth of 9 metres downwards, the soil is dominated by sandy silt with dense to very dense conditions. Considering the parameters cu, φ, and qu, the effective depth for foundation pile placement is recommended to be at a depth of 10 metres, where the soil has reached high shear strength and stable bearing capacity. Table 2 is a table of soil parameters processed from boring log data.

Table 2. Values Taken in Soil Parameters

No. Layer	Depth (m)	Soil Type	γ (kN/m$^{3)}$	Cu (kPa)	Φu (°)
1	0-4,5	Silty Clay (CH)	16,1	37	30
2	4,5-7,0	Clayey Silt (MH)	14,5	50	31
3	7,0-8,5	Silty Clay (CH)	15,5	143	31
4	8,5-9,5	Clayey Silt (MH)	16,5	202	31

5	9,5-20	Silty Sand (SM)	23	209	36

Source: Writer

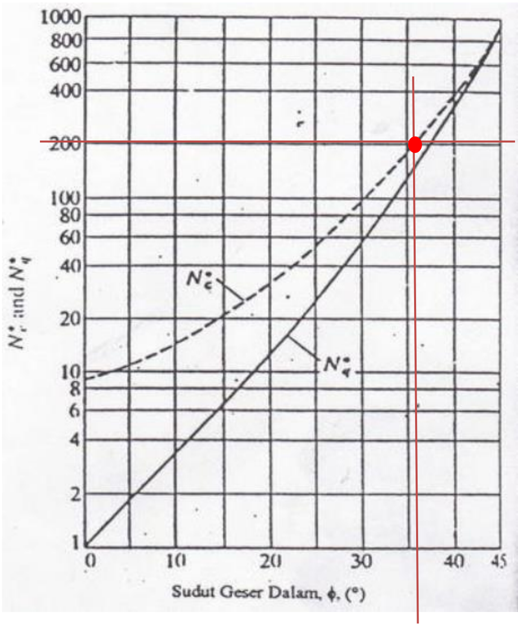
Column Cross-Section Area

Ap $= \frac{1}{4} \times \pi \times (D)^2$ ………………….(1)

$= \frac{1}{4} \times 3{,}14 \times (0{,}50)^2 = 0{,}196\ m^2$

Nc Value

The end bearing is located at layer 5 with a value of φu= 36°. Then, plotting this value as shown in Figure 2, we obtain a value of Nc = 201



Figur 2. Nc Value
Source: Mayerhof, 1976

Ultimate End Resistance

$Qp = Ap \cdot cu5 \cdot Nc$………………….(2)

Qp = 0,196 × 201 × 209 = 8.244,27 kN

Circumference of the Pile Cap

Referring to the pile length of 10 metres and the planned diameter of 0.5 metres, the value of As is obtained from equation (3), and the results can be seen in the table below:

As (π × D × h) ………………….(3)

Table 3. As values per soil layer

No. Layer	Depth (m)	Height of each layer (h)	As (π × D × h) (m^2)
1	0-4,5	4,5	7,07
2	4,5-7,0	2,5	3,93
3	7,0-8,5	1,5	2,36

4	8,5-9,5	1	1,57
5	9,5-20	0,5	0,79

Source: Writer

Adhesion Factor Value (α) for Each Soil Layer

Based on the Cu values collected in Table 4, using the curve according to Tomlinson in Figure 4.3, the adhesion values (α) were obtained:

Table 4. Adhesion Values (α) According to Tomlinson, 1974

No. Layer	cu (kPa)	adhesion value (α)
1	37	0,79
2	50	0,70
3	143	0,25
4	202	1,0
5	209	1,0

Source: Writer

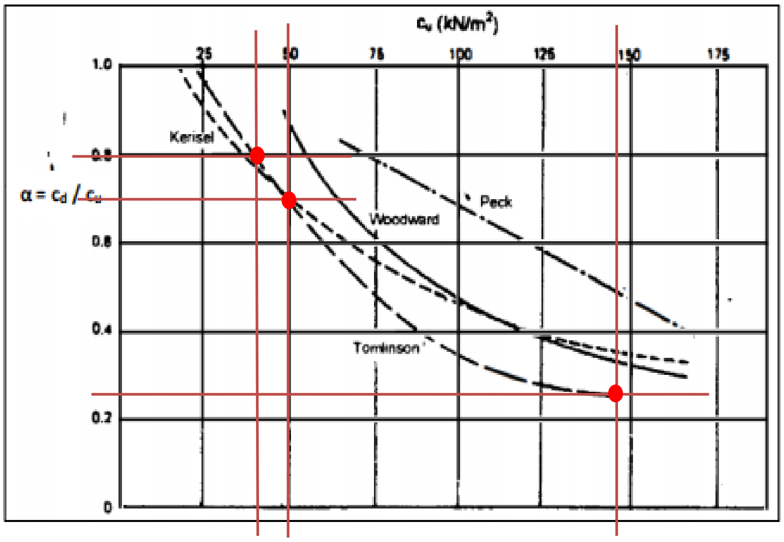


Figure 3. Value of faktor α (Tomlinson,1974)

Friction Resistance of the Pile (fs) for Each Soil Layer

Based on Table 4, using Equation (4), the following is obtained:

$\alpha \times cu$…………………(4)

Table 5. Frictions resistance values of piles (fs)

No. Layer	cu (kPa)	Adhesion value	Friction Resistance (fs)
1	37	0,79	29,23
2	50	0,70	35,00

3	143	0,25	35,75
4	202	1,0	202,00
5	209	1,0	209,00

Source: Writer

Pole Cover Load Capacity (Qs)

Based on Equation 5, then:

Qs = Σ As · fs ………………….(5)

Qs = $(As_1 \times fs_1) + (As_2 \times fs_2) + (As_3 \times fs_3) + (As_4 \times fs_4) + (As_5 \times fs_5)$

= (7,07 × 29,23) + (3,93 × 35,00) + (2,36 × 35,75) + (1,57 ×202,00) + (0,79 ×209,00)

= 909,28 kN

Ultimate Bearing Capacity (Qu)

Based on Equation 6 (Mayerhof, 1976), then:

Qu = Qp + Qs………………….(6)

Qu = 8.244,27 + 909,28 = 9.153,55 kN

Permitted Load Capacity (Qa)

To calculate the Permitted Load Capacity (Qa) of the pile, Equation 7 is used, with a safety factor (SF) of 3. The result obtained is:

$$Qa = \frac{Qu}{SF} = \ldots\ldots\ldots\ldots\ldots\ldots(7)$$

$$Qa = \frac{Qu}{SF} = \frac{9153,55}{3} = \mathbf{3.051,18\ kN}$$

Number of Poles in the Group

To calculate the number of group columns, divide the axial load value of each column by the allowable bearing capacity (Qa). However, the axial load values of the columns in Table 1 must first be converted to kN. The results are shown in Table 6.

Table 6. Number of group columns

Column Number	Axial Load (kN)	Allowed Bearing Capacity, Qa (kN)	Number Grouped Piles (Axial Load/Qa)	Rounded
K1	2.270,32	3.051,18	0,74	1
K2	2.944,94	3.051,18	0,97	1
K3	3.082,70	3.051,18	1,01	2
K4	1.935,58	3.051,18	0,63	1
K5	3.374,41	3.051,18	1,11	2
K6	4.108,16	3.051,18	1,35	2
K7	5.093,66	3.051,18	1,67	2
K8	2.996,16	3.051,18	0,98	1
K9	3.645,46	3.051,18	1,19	2

Column Number	Axial Load (kN)	Allowed Bearing Capacity, Qa (kN)	Number Grouped Piles (Axial Load/Qa)	Rounded
K10	3.940,86	3.051,18	1,29	2
K11	4.537,92	3.051,18	1,49	2
K12	2.702,31	3.051,18	0,89	1
K13	2.490,33	3.051,18	0,82	1
K14	2.815,56	3.051,18	0,92	1
K15	2.999,14	3.051,18	0,98	1
K16	1.785,08	3.051,18	0,59	1

Source: Writer

Nc Value Layer 1

It is known that the soil type in the upper layer is Kelanauan Clay (CH). Based on Figure 4, the internal friction angle obtained for ϕ = 30° is Nc = 98.

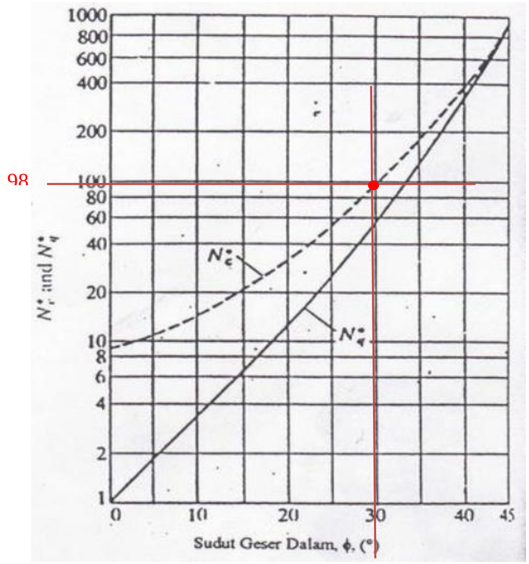


Figure 4. Nc value (Mayerhof,1976)

Ultimate Bearing Capacity of Single Pile Groups

Determining the pile spacing:

Pile spacing to edge = ≤ 1.25 D

= 0.5 m

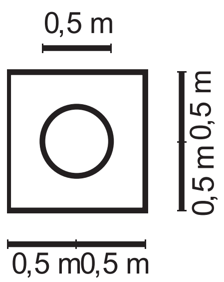


Figure 1. Pile Cap 1 Top View of Pile

Based on Equation 8, we obtain:

$Qg = 2 \cdot Lpile \cdot (B + L) \cdot c + 1{,}3 \cdot cb \cdot Nc \cdot B \cdot L$......................(8)

$Qg = 2 \times 10 \times (1 + 1) \times 37 + 1{,}3 \times 37 \times 98 \times 1 \times 1 = 6.193{,}80$ kN

Therefore, the bearing capacity of the pile group (Qg all) becomes:

$$Qg\ all = \frac{Qg}{SF} = \frac{6.193{,}80}{3} = 2.064{,}60 \text{ kN}$$

Single Pile Group Efficiency

a. General equation

$Eg = \frac{Qg}{N\ x\ Qa}$......................(9)

b. Simple Formula

Calculating the circumference of apole

p = π . D = 3,14 x 0,5 = 1,57 m

$\text{Eg} = \frac{2.(m+n-2).s+4.D}{p.m.n}$ ....(10)

c. *Converse – Labarre* Formula

Menghitung nilai θ

$\theta = arc\ tan\ D/s$ ....(11)

$\text{Eg} = 1 - \theta.\left[\frac{(n-1).m+(m-1).n}{90.m.n}\right]$ ....(11)

d. *Los Angeles* Formula

$\text{Eg} = 1 - \frac{D}{\pi.s.m.n}.[m.(n-1) + n.(m-1).(n-1).\sqrt{2}]$ ....(12)

e. *Seiler – Keeney* Formula

$\text{Eg} = \left[1 - \frac{36.s.(m+n-2)}{(75.s^2-7).(m+n-1)}\right] + \frac{0{,}3}{m+n}$ ....(13)

From the five efficiency values above, the smallest Eg value is taken. The results can be seen in the following table

Table 7. Efficiency of Single Pile Groups

Formula	Result	Criteria	Description
General Equation	0.68	< 1.0	Meets requirements

Simple Formula	1.27	> 1.0	Does not meet requirements
Converse-Labarre Formula	1	< 1.0	Qualified
Los Angeles Formula	1	< 1.0	Meets requirements
Seiler-Keeney formula	1.15	> 1.0	Not eligible

Source: Writer

Single Pole Group Load Capacity

Qgroup = 0,68 x 1 x 3.051,18
= 2.074,80 kN > 2.064,60 kN **(SAFE)**

Ultimate Bearing Capacity of 2-Pole Group

Determining the distance between columns
Pole distance to edge = ≤ 1.25 D
= 0.5 m
Distance between poles (d) = 2.5D - 3D
= 1.25 – 1.5
Pole spacing (s) used = 1 metre

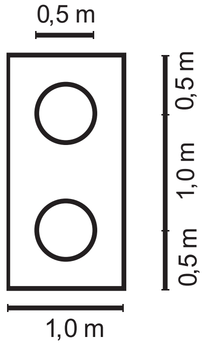


Figure 6. Pile Cap 2 Piles Top View

Based on Equation 8, we obtain:

Qg = 2 × 10 × (2 + 1) × 37 + 1,3 × 37 × 98 × 2 × 1 = 11.647,60 kN

Therefore, the allowable bearing capacity of the pile group (Qg all) becomes:

$$Qg\ all = \frac{Qg}{SF} = \frac{11.647,60}{3} = 3.882,50 \text{ kN}$$

Efficiency of the 2-Pile Group

Table 7. Efficiency of Single Pile Group

Formula	Result	Criteria	Description
General Equation	0.64	< 1.0	Eligibility requirements
Simple Formula	1.43	> 1.0	Does not meet the requirements
Converse-Labarre Formula	0.88	< 1.0	Qualified

Los Angeles Formula	0.94	< 1.0	Meets requirements
Seiler-Keeney formula	0.90	< 1.0	Meets requirements

Source: Writer

From the five efficiency values above, the smallest Eg value is taken

Load-bearing capacity of group 2 columns

Qgroup = 0,64 x 2 x 3.051,18
= 4.149,60 kN > 3.882,50 kN **(SAFE)**

Soil Poisson's Ratio Value (vs)

Based on the soil parameters in Table 2, at a depth of 10 metres, the soil type is silty sand (SM), so the value of 0.3 is taken from Table 8.

Table 8. Poisson's Ratio Values for Soil Types

Type of Soil	***Poisson's Ratio (v)***
Clay, saturated	0,4-0,5
Clay, unsaturated	0,1-0,3
Sand clay	0,2-0,3
Silt	0,3-0,35
Sand, gravelly sand	0,1-1
commonly used	0,3-0,4
Rock (depend somewhat on type of rock)	0,1-0,4
Loess	0,1-0,3
Ice	0,36
Concrete	0,15
Steel	0,33

(source: Bowless, Joseph E., 1988)

Coefficient Value (Cp)

For the type of sand used in pile planning (SM), the coefficient value (Cp) is taken as 0.03, referring to Table 9.

Soil Type	**Pile**	**Bored Pile**
Sand	0.02-0.04	0.09-0.18
Clay	0.02-0.03	0.03-0.06

Table 9. Cp Coefficient Values

Silt	0.03-0.05	0.09-0.12

(source: Vesic, 1977)

Soil Elastic Modulus (Es) Values

For silty sand (SM) soil type, with an N-SPT value of 52, based on Table 10, the following value is obtained:

Es=300(N+6)

Es=300(52+6)

Es=17.400 kN/m2

Table 10. Correlation between N-SPT and Soil Modulus of Elasticity

Soil	***SPT***	***CPT***
Sand *(Normally Consolidated)*	Es = 500 (N+15) Es = (15,000–22,000) In N Es = (35,000–50,000) log N	Es = (2-4)qc Es = $(1\text{-}Dr^2)$qc
Sand *(Saturated)*	Es = 18,000 + 750 N	
Sand *(Overconsolidated)*	Es = 250 (N+15) Es(ocr) = $Es_{(Nc)}(OCR)^{1/2}$	Ice = (6–30)qc
Gravel & Gravelly Sand	Es = 1,200 (N+6) Ice = 600 (N+6) N≤15 Ice = 600 (N+6)+2,000 N≤15	
Clayey Sand	Es = 320 (N+15)	Es = (3-6)qc
Silty Sand	Es = 300 (N+6)	Es = (1-2)qc
Soft Clay	-	Es = (3-8)qc
Clay	Ip > 30 or organic Ip > 30 or stiff $Es_{(ocr)} = Es_{(Nc)}(OCR)^{1/2}$	Es = (100-500)Su Es = (500-1,500)Su

(source: Coduto, 1999)

Settlement Due to Pile Deformation (Ss)

$$Ss = \frac{(Qp+\alpha.Qs)xL}{ApxEp} \text{.....................} (14)$$

$$Ss = \frac{(8.244,27 + 1 \text{ x } 909,28) \text{ x } 10}{0,19625 \; x \; 3,29 \; x \; 10^7}$$

$$Ss = 0,014177 \; m$$

$$Ss = 1,42 \; cm$$

Pole Tip Decline (Sp)

$$Sp = \frac{CpxQp}{Dxqp} \text{.....................} (15)$$

$$Sp = \frac{0,03 \text{ x } 8.244,27}{0,5 \; x \; 42.009}$$
$$Sp = 0,011775 \; m$$
$$Sp = 1,18 \; cm$$

Influencing Factors (Iws)

$$\text{Iws} = 2 + 0,35\sqrt{\text{L/D}} \;......................(16)$$
$$\text{Iws} = 2 + 0,35\sqrt{10/0,5}$$
$$\text{Iws} = 3,565$$

Pole Settlement Due to Load Transfer Along the Pole (Sps)

$$Sps = \left(\frac{Qs}{pxL}\right).\frac{D}{Es}.(1 - \upsilon s^2).Iws \;......................(17)$$
$$Sps = \left(\frac{909,28}{1,57x10}\right).\frac{0,5}{17.400}.(1 - 0,3^2).3,565$$
$$Sps = 0,00540 \; m$$
$$Sps = 0,54 \; cm$$

Total Foundation Pile Settlement (S)

S = Ss+Sp+Sps......................(18)

S = 1,42+1,18+0,54

S = 3,14 cm

Pile Group Foundation Settlement (S)

To calculate the settlement of pile group foundations, the equation according to Vesic (1977) is used, namely:

$$\text{Sg} = \text{S X} \sqrt{\frac{\text{Bg}}{D}} \;......................(19)$$

Spermitted =10%×D ....(20)

Or based on SNI 8460:2017, namely:

Spermitted (15 cm+b/600) ....(21)

Then,

Stotal ≤ Spermitted atau Stotal ≤ Spermitted (15 cm+b/600)

Table 11. Foundation Settlement of Pile Groups

Pile Group	Bg	Sg (cm)	Description
Pile Group 1	1	4.43	Stotal ≤ Spermitted 4.43 cm ≤ 15 cm (Permitted)
Group 2 Pile	2	6.27	Stotal ≤ Spermitted 6.27 cm ≤ 15 cm (Permitted)

(source: Writer)

CONCLUSION AND RECOMMENDATIONS

Conclusion

Based on the results of the pile foundation design for the Mozia Office construction project, the following conclusions can be drawn:

1. The soil investigation results at borehole BH-01 to a depth of 30.50 m show eight soil layers with varying physical and mechanical characteristics based on the N-SPT, γ, cu, φ, and qu parameters from field and laboratory tests, as well as N-SPT correlations.

The upper layer up to 7.00 m is dominated by clay and silt (CH–MH) with N-SPT 6–8, γ 14.5–16.7 kN/m³, and cu 37–50 kN/m² (soft–medium), followed by an increase in strength at a depth of 7.00–9.50 m with N-SPT 19–32 and cu 143–202 kN/m². From 9.50–20.00 m, the soil is dominated by silty sand (SM) with N-SPT 14–52, φ 31°–33°, and qu up to 409 kN/m² (dense–very dense), while up to 29.00 m still shows high N-SPT (±38), cu ±203 kN/m², and ϕ up to 34°. The deepest layer at 29.00–30.50 m consists of CH with N-SPT 20, cu 167 kN/m², and γ 18 kN/m³. In general, bearing capacity increases with depth, so the pile tip is recommended at around 10 m, when the soil has stable shear strength and bearing capacity.

2. The pile foundation design was carried out using the Meyerhof method for end bearing capacity and the α method according to McClelland (1974) and Reese & Wright (1977) for skin friction, resulting in an ultimate bearing capacity of 9,153.55 kN and a allowable bearing capacity of 3,051.18 kN. The efficiency of the pile groups was analysed using several empirical methods, taking the smallest value as the safest condition, which resulted in 9 single pile points and 7 points for groups of 2 piles. Settlement analysis using the semi-empirical method and Vesic (1977) showed a settlement of 4.43 cm for single piles and 6.27 cm for pile groups, which is still within the allowable limit.

Recommendations

In calculating the bearing capacity of the foundation with a planned diameter for the pile foundation of 50 cm, a high value of 9,153.55 kN was obtained. Therefore to obtain a more efficient value, it can be replaced with a planned diameter of < 50 cm.

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