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Analysis of Foundation Bearing Capacity Using Reese & Wright (1977) and Skempton (1966) Methods

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Abstract

Bridge has a structural load from above which is supported by the foundation. The foundation has an important role to holding the load from the superstructure. Therefore, planning the foundation, the bearing capacity must be taken into account so the foundation should withstand the load. In the Bengkulu – Taba Penanjung toll access road construction project, bored piles are used. To determine the bearing capacity of the foundation, NSPT soil data is needed for analysis needs. Based on the results of the bearing capacity analysis, the value of 382 tons with the Reese & Wright method and 349 tons with the Skempton method at a depth of 14 m was obtained. With a 7 x 2 pile configuration, an average efficiency value of 85% is obtained. With this efficiency value, the group pile bearing capacity is 4538,22 tons (Reese & Wright) and 4147,47 tons (Skempton). With the group's pile bearing capacity, the pile is considered capable of withstanding the axial load of the superstructure of 2391.21 tons. The amount of immediate settlement that occurs in a single pile is 13.72 mm using the Vesic method (1977). This reduction is safe where the settlement limit is 65 mm according to Skempton and Mc Donald. Because the dominant soil is clay, the value of the decrease in primary consolidation that occurs is 5 mm. Thus, the drill pile foundation with a length of 14 meters and a total of 14 piles with a configuration of 7 x 2 piles can be said to be feasible and can withstand the axial load transmitted to the drill pile foundation.

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Introduction

The bridge is a construction that has the function of connecting two parts of the road which are separated by lakes, valleys, ravines, railroads, etc. At the end of the bridge it is connected to the abutment, which is the meeting point between the ground and the end of the bridge structure. In order for the abutments to withstand the loads received, a strong foundation is needed to support the abutments [1]. The foundation used in this abutment is a pile foundation. There are two types of pile foundations, namely pile foundation and drilled pile foundation. In this analysis, drilled pile foundations are used because the results of the SPT data found that the soil layer is relatively hard. So that if the piling is done it is feared that the pile will break during the erection process. Therefore the pile foundation used is bored pile foundation [2]. Analysis of the bearing capacity of the foundation is carried out using two calculation methods. The method used is the method of Reese & Wright (1977) and the method of Skempton (1966). The purpose of this paper is to determine the bearing capacity of the foundation and be able to plan the depth of the bored pile foundation so that it can withstand the load on it [3]. The research method used in this analysis has a slight difference, namely in the calculation of the pile end bearing capacity. Skempton added the reduction factor to the pile diameter and added the coefficient Nc = 9. For more details, the following formulas are presented.

Theory

A. Reese & Wright (1977)

Reese & Wright presented an equation to determine the value of carrying capacity based on NSPT data. There are several formulas for determining the bearing capacity of piles according to the type of soil. The following is the formula given by Reese & Wright (1977) [4].

Cohesive Soil :

$$Q_p = 9 \text{ x } c_u \text{ x } A_p \quad (1)$$
$$Q_s = \alpha \text{ x } c_u \text{ x } P \text{ x } Li \quad (2)$$

Grained Soil :

$$Q_p = \frac{40}{3}\ x\ N_{SPT\ rata-rata}\ x\ \frac{Li}{D} \quad (3)$$
$$Q_s = 2\ x\ N_{SPT}\ x\ P\ x\ Li \quad (4)$$

dengan,
Q_p = Load carried at the pile point (t)
A_p = Area of pile tip (m^2)
Q_s = Frictional ressintance (t)
P = Perimeter of the pile section (m)
c_u = N_{SPT} x 2/3 x 10 (kN)
α = Empirical adhesion factor for bored pile is 0,55

B. Skempton (1966)

Skempton presented the bearing capacity equation for bored piles in clay soil conditions. The following is the formula [5].
Point Bearing Capacity, Q_p:

$$Q_p = f_b \text{ x } A_b \quad (5)$$
$$f_b = \mu \text{ x } c_u \text{ x } N_c \quad (6)$$
$$A_b = \tfrac{1}{4} \text{ x } \pi \text{ x } D \quad (7)$$

where,
A_b = Area of pile tip (m^2)
μ = Correction factor
μ = 0,8 (for d < 1 m) and 0,75 (for d > 1 m)
c_u = Mean undrained shear strength (kN/m^2)
N_c = 9

Frictional Resistance, Q_s :

$$Q_s = A_s \text{ x } f_s \quad (8)$$
$$A_s = \pi \text{ x } D \text{ x } L \quad (9)$$
$$f_s = \alpha \text{ x } c_u \quad (10)$$

where,
A_s = Area of pile surface (m^2)
f_s = Unit friction resistance at any given depth z (kN/m^2)
α = Empirical adhesion factor for bored pile is 0,45

Allowable Pile Capacity (Q_{all})

The results of the point bearing capacity and skin friction are added up to become the ultimate bearing capacity (Qult). The value of this carrying capacity must be multiplied by the safety factor to become the allowable carrying capacity [6].

$$Q_{ult} = Q_p + Q_s \quad (11)$$
$$Q_{all} = Q_{ult}/SF \quad (12)$$

TABLE 1 FACTOR OF SAFETY ACCORDING TO REESE & O'NIEL

Structure Classification	Factor of Safety			
	Good Control	Normal Control	Bad Control	Very Bad Control
Monumental	2.3	3	3.5	4
Permanent	2	2.5	2.8	3.4
Temporary	1.4	2	2.3	3

Source: Hardiyatmo, (2011)

By classifying the building that we are planning, we can determine the appropriate safety factor based on Table 1 to obtain the allowable carrying capacity value [7].

C. Group Piles - Efficiency

The bearing capacity of a single pile may decrease after being grouped. The factor that causes a decrease in the bearing capacity of these piles is the distance between the piles that are too close so that the bearing capacity of the piles can be reduced. The following is a formula for finding the amount of efficiency conveyed by several experts.

- Simple Formulas

$$Eg = \frac{2(m+n)s+4D}{Kt\ x\ m\ x\ n} \tag{13}$$

- Los Angeles Group Action Equation

$$Eg = 1 - \frac{D}{\pi Snm}\left[n(m-1) + m(n-1) + \sqrt{2}(n-1)(m-1)\right] \tag{14}$$

- Seiler-Keeney Equation

$$Eg = \left\{1 - \left[\frac{11s}{7(s^2-1)}\right]\left[\frac{n+m-2}{n+m-1}\right]\right\} + \frac{0.3}{n+m} \tag{15}$$

where,
S = Center to center spacing of piles
D = Diameter of piles
m = Number of rows in the group piles
n = Number of piles in a row

The results of the group pile efficiency are taken as an average value.

D. Elastic Settlement of a Pile by Vesic (1977)

The settlement that occurs in the pile under working load on the pile is caused by three factors.:

$$S = S_1 + S_2 + S_3 \tag{16}$$

where:
S = Total pile settlement
S_1 = Settlement of pile shaft
S_2 = Settlement of pile cause by the load at the pile point
S_3 = Settlement of pile caused by the load transmitted along the pile saft.

The immediate/elastic settlement described by Vesic (1977) is the settlement caused by the distortion of the compressed soil mass and occurs at a constant volume [8]. The following is the formula used to calculate the settlement of S_1, S_2, dan S_3 [9].

$$S_1 = \frac{(Q_{wp}+\xi Q_{ws})L}{A_p\ x\ E_p} \tag{17}$$

$$S_2 = \frac{Q_{wp} x\ C_p}{D\ x\ q_p} \tag{18}$$

$$S_3 = \frac{Q_{ws} x\ C_s}{L\ x\ q_s} \tag{19}$$

where,
Q_{wp} = Load carried at the pile point under working load condition (ton)
Q_{ws} = Load carried by frictional (skin) resistance under working load condition (ton)
ξ = Coefficient of skin friction
A_p = Area of pile cross section (m^2)
E_p = Young's modulus of the pile material
L = Length of pile (m)

D = Diameter of pile (m)
C_p = An empirical coefficient (given in Table 2)
q_p = Ultimate point resistance of the pile per unit area (ton/m^2)
q_s = Frictional resistance per unit area (ton/m^2).
C_s = An empiricl constant
$= \left(0{,}93 + 0{,}16\sqrt{L/_D}\right) x C_p$

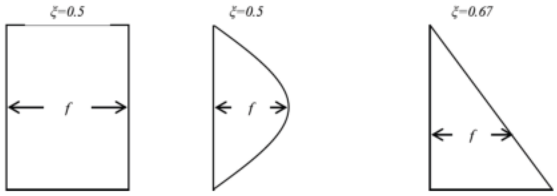

Figure 1. Variation types of unit friction resistance distribution along the pile shaft
Source: Bowless, (1933)

TABLE 2. TYPICAL VALUES OF C_P

Soil Type	Driven Pile	Bored Pile
Sand (dense to loose)	0,02 – 0,04	0,09 – 0,18
Clay (stiff to soft)	0,03 – 0,03	0,03 – 0,06
Silt (dense to loose)	0,03 – 0,05	0,09 – 0,12

Source : Braja M. Das, 1995

The value of ξ depends on the nature of unit frictional resistance distribution along the pile shaft.
ξ = 0,67 (sand soil)
ξ = 0,5 (clay or silt soil)
The maximum settlement limit according Skempton and Mc Donald, 1995 can be seen in Table 3.

TABLE 3. MAXIMUM SETTLEMENT LIMIT

Type of Foundation	Maximum Settlement Limit (mm)
Separate foundation on clay	65
Separate foundation on sand	40
Raft foundation on clay soil	65 - 100
Raft foundation on sand soil	40 - 65

Source: Skempton and Mc Donald 1995

E. Elastic Settlement of Group Pile by Vesic (1977)

Several investigations about settlements of group pile that have been reported in the literature have had very mixed results. The simplest relationship for group pile settlement is given by Vesic (1977) with the following equation.

$$Sg = \sqrt{\frac{Bg}{DS}} \tag{20}$$

where,
Sg = Elastic settlement of group piles
S = Elastic Settlement of each pile at comparable working load
Bg = Width of pile group section
D = Diameter of each pile in the group

F. Consolidation Settlement
Settlement of consolidation generally occurs in clay soils and occurs 1/3 of the pile length from the bottom of the pile on long pile foundations. To determine the amount of consolidation settlement, the OCR value of the soil must be calculated first using the following formula.

$$OCR = \frac{\sigma_o'}{\sigma_c'} \tag{21}$$

$$\Delta\sigma'o = \frac{F_x}{(B_g+z)(L_g+z)} \quad (22)$$

Method

In this research, the first stage is collecting the required data. The data needed are soil data and load of upper structure. After obtaining the required data, then proceed to the analysis stage of the data. For more details, see the flowchart in Figure 2.

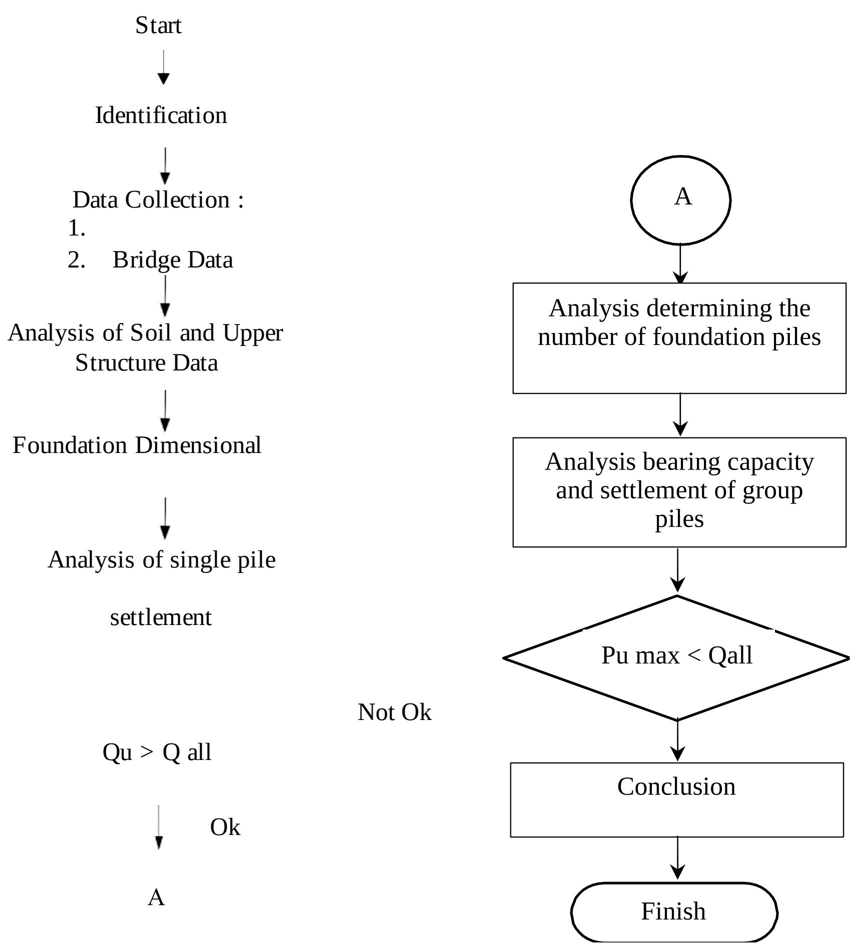

Figure 2. Flow chart

Results and Discussion

A. Soil Data Analysis and Upper Structure Loading

The soil data needed in this analysis is the NSPT data collected at the analysis location. The soil parameters used are the result of a correlation with the NSPT value. The Cu value was obtained from the correlation according to Terzaghi and Peck's suggestion with a c_u value of 0.6 x NSPT. While the value of φ is used the correlation proposed by Lambe and Whitman (1969).

The loading data used in this analysis is data that has been calculated by the consultant. After that, load combinations are carried out in accordance with SNI 1725: 2016 concerning Loading for Bridges. The combination used is a combination of strong 1 and extreme 1 [10]

Table 4 contains loading data which is reprocessed into Strong 1 and Extreme 1 load combinations. The following is the result of load combination analysis.

TABLE 4 LOAD COMBINATION RECAPITULATION

N	Load Combination	Vz (kN)	Hx (kN)	My (kN)
1	Strong 1	23.912	2984	14624
2	Extreme 1	18.660	9527	29758

The results of the strong load combination 1 produce the greatest strong axial bearing capacity 1 and the extreme combination 1 produces the greatest lateral bearing capacity which will later be used to control the load on the pile bearing capacity.

B. Bored Pile Axial Bearing Capacity Analysis

The general formula for the bearing capacity of a pile is the total end resistance (Qp) and total frictional resistance (Qs). Then the result of the sum becomes the value of the ultimate carrying capacity (Qu). The results of the analysis of the ultimate bearing capacity of the piles are presented in the Table 5.[11]

TABLE 5 SINGLE BEARING CAPACITY RECAPITULATION

Method	L	Qu		Qall
	(m)	*(kN)*	*(ton)*	*(ton)*
Reese & Wright (1977)	10	6904.53	690.45	276.18
	12	8227.58	822.76	329.10
	14	9550.62	955.06	382.02
Skempton (1966)	10	6519.12	651.91	260.76
	12	7623.70	762.370	304.95
	14	8728.29	872.83	349.13
	16	9832.87	983.29	393.31

The results of the pile bearing capacity analysis (Qu) must be multiplied by the safety factor according to Reese & Wright with a moderate control value of 2.5. The results of the excavation become the allowable pile carrying capacity (Qall) value which is used for the next stage of analysis.[12] Bearing capacity analysis is calculated to a depth of 14 m. Using the Reese & Wright method we get 382 kN and the Skempton method we get a bearing capacity of 349 kN. It can be said that with the same analytical data, the calculation of the Reese & Wright method has a greater carrying capacity than Skempton.

C. Elastic Settlement of Single Pile by Vesic Method

Elastic settlement or immediate pile settlement is settlement caused by the distortion of the compressed soil mass, and occurs at constant volume. This settlement can occur in the pile stem, because there is a load at the end of the pile and the load is distributed along the pile. Table 6 is a recapitulation of the pile settlement that occurred.

TABLE 6 RECAPITULATION OF SINGLE PILE SETTLEMENT

Method	L	Se(1)	Se(2)	Se(3)	Se Total
	(m)	*(mm)*	*(mm)*	*(mm)*	*(mm)*
Reese & Wright (1977)	10	0.97	9.42	2.10	12.49
	12	1.32	9.42	2.38	13.13
	14	1.73	9.42	2.60	13.76
	16	2.19	9.42	2.79	14.41
Skempton (1966)	10	0.86	9.42	2.64	12.93
	12	1.17	9.42	2.86	13.45
	14	1.51	9.42	3.04	13.98
	16	1.91	9.42	3.19	14.52

The limit for pile settlement according to Skempton and Mc Donald (1995) where the settlement of piles in clay is 65 mm. Thus it can be said that the pile settlement that has occurred is still within reasonable limits.

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D. Bearing Capacity of Axial Group Pile

In determining the axial bearing capacity of group piles, it is necessary to take into account the efficiency of group piles first. The efficiency method used in this analysis is a simple formula, the Los Angeles Group Action Equation, and the Seiler – Keeney Equation. Table 7 is the result of group pile efficiency analysis. [13],[14}

TABLE 7 EFFICIENCY OF GROUP PILES

D	(m)	1	1	1
S	(cm)	200	250	300
m		9	7	6
n		2	2	2
Simple Formula	%	71	89	106
Los Angeles	%	74	80	83
Seiler Keeney	%	81	86	90
Average	%	75	85	93

Based on Table 7 it can be concluded that the farther the space between the piles, the more the efficiency value of the pile is closer to 100% where the less the bearing capacity is reduced. After the efficiency value is obtained, then the allowable bearing capacity of the group pile is calculated. Table 8 is the result of group pile bearing capacity analysis.

TABLE 8 RECAPITULATION OF BEARING CAPACITY GROUP PILE

Method	**L**	**Qall**	**Efficiency**	**N**	**Qall Group**
	(m)	***(mm)***	***(%)***	***(Pile)***	***(ton)***
Reese & Wright (1977)	10	281.82	85	14	3347.82
	12	335.82	85	14	3989.33
	14	389.82	85	14	4630.84
	16	443.82	85	14	5272.35
Skempton (1966)	10	260.76	85	14	3097.73
	12	304.95	85	14	3622.60
	14	349.13	85	14	4147.47
	16	393.31	85	14	4672.34

From the results of the calculation of the pile carrying capacity taken at a depth of 14 meters with a group carrying capacity value of 4,630.22 tonnes (Reese & Wright) and 4,147.47 ton (Skempton).

E. Control of Static and Dynamic Axial Load

Control of Static and dynamic axial load is carried out with the aim of knowing the maximum load that can occur on the pile. Static axial load using strong combination 1 loading. The following is the result of the analysis of the maximum load on the pile.

Pmax = 314,68 ton

TABEL 9 CONTROL OF STATIC AXIAL LOAD

Method	**Pmax**	**Qall single pile**	**Qall>P**
	(ton)	***(ton)***	
Reese & Wright	314,68	382,02	OK
Skempton	314,68	349,13	OK

If the results of the comparison between the Pmax and Qall values meet the requirements, then the pile is declared stable, which means it can withstand the maximum load that occurs. Then proceed with dynamic axial load control with extreme load combinations 1.

Pmax = 379,98 ton

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TABLE 10 CONTROL OF DYNAMIC AXIAL LOAD

Method	Pmax (ton)	Qall single pile (ton)	Qall>P
Reese & Wright	379,98	382,02	OK
Skempton	379,98	349,13	OK

Judging from the results of calculating the Pmax value compared to a single Qall that meets the requirements, it can be stated that the pile is stable and can withstand the maximum load that occurs.

F. Consolidation Settlement on Clay Soil

Settlement of consolidation generally occurs in clay soils. Consolidation settlement is settlement that occurs due to the release of water on the clay soil layer after being loaded from above. On deep foundations, consolidation settlement occurs at 1/3 of the pile length from the bottom of the pile. The following is a layer of soil that needs to be reviewed with a pile length of 14 m.[12]

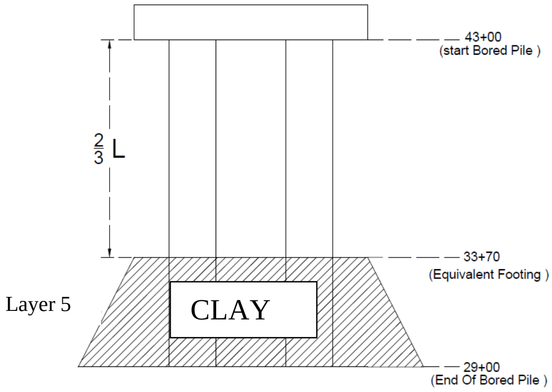

Figure 3 Consolidation Settlement

n determining the formula for consolidation settlement, the OCR value of the land must be determined in advance. Obtained OCR value on the soil under review is:

OCR = 1,07 > 1 => From the OCR value, the soil is an overconsolidation soil, so it must be reviewed by calculating the added stress.

$\boldsymbol{\sigma'_o + \Delta\sigma'_o}$ = 386,96 kN/m^2
$\boldsymbol{\sigma'_c}$ = 361,46 kN/m^2
$\boldsymbol{\sigma'_o + \Delta\sigma'_o > \sigma'_c}$
386,96 kN/m2 > 361,46 kN/m^2

With the added stress value greater than the compression stress, equation (22) is used to determine the consolidation settlement. [15]

Sc =5,03134 mm

The settlement limit on the foundation according to Skempton and Mc Donald (1995) is 65 mm. So the reduction can be said to be safe.

Conclusions

From the calculation of the bearing capacity with the Reese & Wright and Skempton methods, both of them were able to withstand the superstructure load that occurred. With the criteria for a pole diameter of 1 meter, a pile length of 14 meters and a number of piles of 14 poles with a configuration of 7 x 2 poles. Against this decline, the foundation design is safe and can be used as a reference for foundation design.

Based on the results of the two calculation methods used, it produces the same pile configuration value, namely 7 x 2 piles with a diameter of 1 meter. Thus, it can be concluded that the coefficient value of the pile diameter and the Nc value in the Skempton method only has a slight influence on the calculations which makes Skempton's calculations produce relatively smaller values.

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