BEARING CAPACITY AND SETTLEMENT BEHAVIOUR OF COASTAL SOIL FOR THE PLANNED BALONGAN PORT DEVELOPMENT, WEST JAVA

DAYA DUKUNG DAN PERILAKU PENURUNAN TANAH PESISIR UNTUK RENCANA PENGEMBANGAN PELABUHAN BALONGAN, JAWA BARAT

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ABSTRACT: The planned development of Balongan Port in West Java requires a comprehensive geotechnical evaluation to support foundation planning at the proposed site, which is underlain by soft, clay-rich sediments. This study aims to assess the bearing capacity of the subsurface and predict potential settlement behaviour by integrating field and laboratory investigations. Standard Penetration Test (SPT) data from two boreholes, BH-3 (14 m) and BH-4 (18 m), and were complemented by laboratory analyses of soil physical and mechanical properties. The site is primarily composed of high-plasticity clay, known for its low strength, high compressibility, and variable geotechnical characteristics. Calculations based on SPT results yielded allowable loads of 53.1 tons at BH-3 and 39.0 tons at BH-4, respectively, while laboratory analyses indicated significantly higher bearing capacities of 265.9 tons and 884.4 tons, respectively. Settlement predictions based on SPT and laboratory data were 0.61 cm and 2.07 cm, with an estimated 90% consolidation period of about 12.9 years. These findings emphasize the variability and compressibility of the soft clay strata as well as the importance of employing multiple assessment methods. The study provides essential input for foundation planning and highlights the need for integrated geotechnical assessment methods to ensure the safety, reliability, and long-term performance of pile-supported structures at the proposed port site.

Keywords: Pile foundation, SPT, bearing capacity, settlement prediction, Balongan Port

***ABSTRAK**: Rencana pengembangan Pelabuhan Balongan di Jawa Barat memerlukan evaluasi geoteknik yang komprehensif untuk mendukung perencanaan fondasi pada lokasi yang diusulkan, yang didominasi oleh sedimen lempung lunak. Studi ini bertujuan untuk menilai daya dukung tanah dan memprediksi perilaku penurunan melalui integrasi data uji lapangan dan laboratorium. Data Uji Penetrasi Standar (SPT) diperoleh dari dua titik bor, yaitu BH-3 (14 m) dan BH-4 (18 m), dan dilengkapi dengan analisis laboratorium terhadap sifat fisik dan mekanik tanah. Lokasi yang dikaji terutama terdiri dari lempung plastisitas tinggi yang dikenal memiliki kekuatan rendah, daya mampat tinggi, serta karakteristik geoteknik yang bervariasi. Perhitungan berdasarkan data SPT menunjukkan beban izin sebesar 53,1 ton (BH-3) dan 39,0 ton (BH-4), sedangkan analisis laboratorium menghasilkan nilai yang jauh lebih tinggi yaitu 265,9 ton dan 884,4 ton secara berturut-turut. Prediksi penurunan berdasarkan data SPT dan laboratorium masing-masing sebesar 0,61 cm dan 2,07 cm, dengan estimasi waktu konsolidasi 90% sekitar 12,9 tahun. Temuan ini menyoroti variabilitas dan kemampatan tanah lempung lunak, serta pentingnya penggunaan metode asesmen yang beragam. Studi ini memberikan masukan penting bagi* *perencanaan fondasi dan menekankan perlunya pendekatan geoteknik yang terintegrasi guna menjamin keamanan, keandalan, dan kinerja jangka panjang struktur fondasi tiang pada lokasi pelabuhan yang direncanakan.*

Kata Kunci: *Fondasi tiang pancang,* SPT*, daya dukung tanah, prediksi penurunan tanah, Pelabuhan Balongan*

INTRODUCTION

The development of maritime infrastructure is essential for supporting regional logistics, economic integration, and national connectivity. A strategic coastal zones identified for a new port facility is located in the Balongan area of Indramayu Regency, West Java. The area lies near major transportation routes, including toll roads and railway lines, providing logistical advantages for future port operations.

However, the subsurface conditions of the Balongan coastal plain apredominantly composed of soft, clay-rich alluvial and marine sediments. Such soils are known for their low bearing capacity, high compressibility, and long consolidation periods, posing major geotechnical challenges for foundation design, particularly for heavy port structures such as container yards, piers, and storage terminals (Bo et al., 2015; Shah, 2021). Inadequate consideration of these conditions during early planning stages may result in excessive settlement, differential movement, or even structural instability in the long term.

To support the planning and feasibility assessment of the proposed Balongan Port, a preliminary geotechnical study was conducted to evaluate subsurface conditions, bearing capacity, and potential settlement behaviour of pile foundations. The investigation involved Standard Penetration Tests (SPT) at two borehole locations, complemented by laboratory testing of soil physical and mechanical properties. The combined approach is intended to provide reliable and practical input for decision-making in early-stage design and land-use planning for the proposed site.

This paper presents the findings of that investigation, discusses the implications of the subsurface characteristics on foundation behaviour, and offers recommendations for future geotechnical evaluation and engineering design strategies

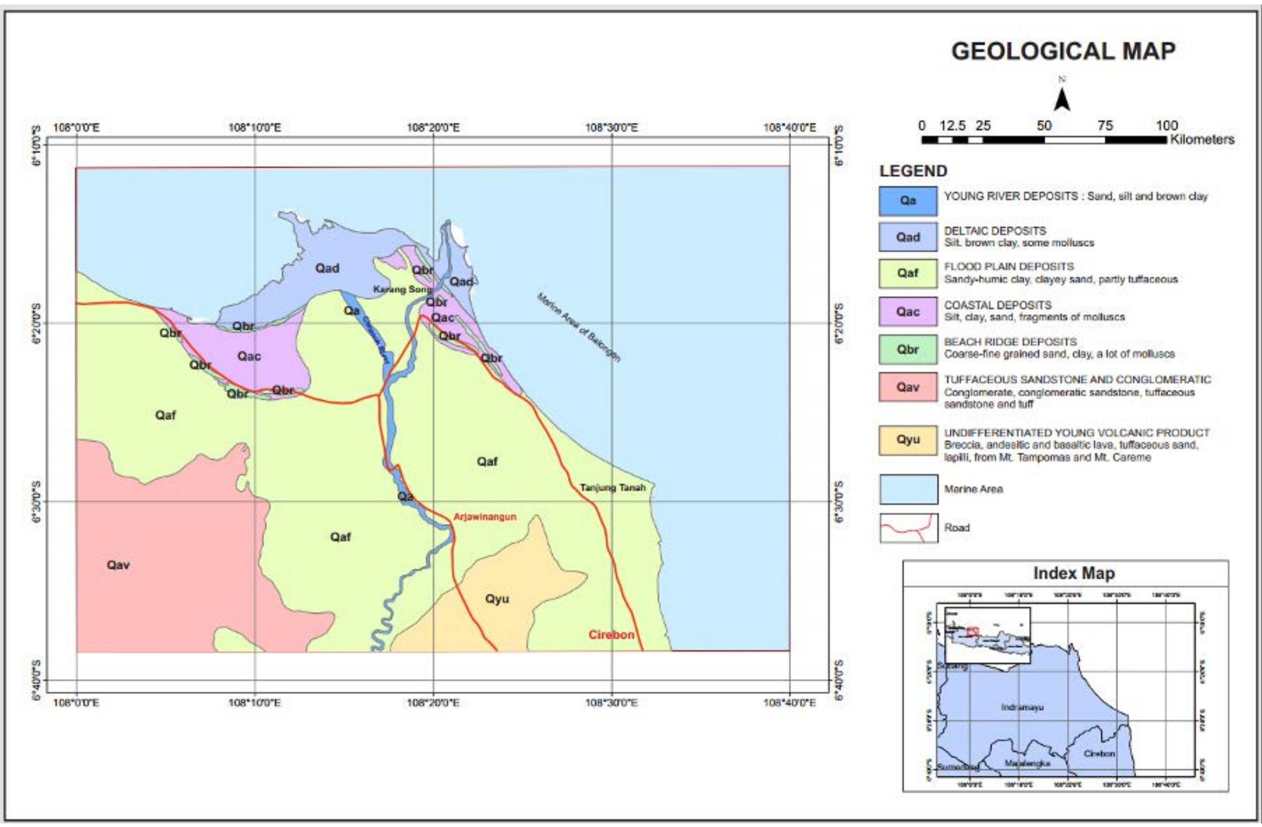

Figure 1. Geological map of the Balongan coastal area, West Java

appropriate for soft clay-dominated port development sites.

Geological Setting

The morphology of the study area is generally characterized as coastal environment. The morphological characteristics of the Balongan coastal area consist of sandy beaches and a delta. The sandy beach morphology stretches from the tip of Tanjung Tanah to the Karang Song area, which is strongly influenced by marine energy processes, including both waves and currents.

The study area (Figure 1) is an alluvial plain composed of beach ridge deposits, floodplain deposits, beach deposits, delta deposits, and river deposits (Achdan & Sudana, 1992). At the mouth of the Cimanuk River, there are delta deposits consisting of a mixture of terrestrial and marine sediments. These sediment distributions form an alluvial fan, resembling a birdfoot delta. Along the coastline, various forms of aquaculture have been developed, including shrimp ponds, salt ponds, and settlements (Ilahude and Usman, 2009).

The beach ridge deposits consist of coarse to fine sand and clay, and are rich in mollusks. The height of the ridges reach up to 5 meters. These ridges are generally limited to the coastal area, with some of them positioned parallel to one another, while others radiate from a single point. This area is home to settlements and serves as a transportation route. Part of the Jakarta–Cirebon highway runs along these beach ridges.

The floodplain deposits consist of sandy clay, humus clay, and range in color from brownish-gray to black. Southward, these deposits become more compact, and their color gradually shifts to a reddish hue. This unit overlying older strata is characterized by erosional surfaces, as observed in the Cibogor River and the upstream section of Kali Kandanghaur. These deposits extend broadly into the Cirebon and Arjawinangun areas as alluvium.

The delta deposits consist of silt and clay, with a brownish-black color, containing small amounts of mollusks, ostracods, planktonic foraminifera, and benthic foraminifera. This unit is typically found in areas designated for the cultivation of milkfish, shrimp ponds, and mangrove forests.

The river deposits consist of sand, silt, and clay, with a brown color, and are predominantly found along the Cimanuk River.

METHODS

Preliminary Stage

The preliminary stage of this research involved a comprehensive review of literature and secondary data to support early-stage geotechnical evaluation for the proposed Balongan Port site. This included the compilation and analysis of regional geological information, topographic and thematic maps, as well as previously published studies relevant to coastal soft soil behaviour, pile foundation design, and port infrastructure on lowland marine deposits. Some important secondary data include:

1. Geological maps at a scale of 1:100,000, particularly the Indramayu Sheet compiled by Achdan and Sudana (1992), which provide stratigraphic and lithological context for the site.
2. Topographic maps and satellite imagery to assess site accessibility, surface morphology, and potential drainage patterns.
3. Location map of the drilling points

This stage was critical in determining the representative locations for borehole drilling and field testing. Borehole locations BH-3 and BH-4 were selected based on a combination of geological considerations, site accessibility, and anticipated variability in subsurface conditions. The information gathered during the preliminary stage served as the foundation for designing the subsequent field investigation program and for contextualizing the geotechnical data within the broader geological framework of the site.

Data Collection Stage

The data collection stage consisted of systematic field investigations and laboratory testing aimed at characterize the geotechnical properties of the proposed Balongan Port site. This stage was conducted in accordance with internationally recognized geotechnical standards to ensure the quality, accuracy, and reproducibility of the data.

1. Field Investigation

Subsurface exploration was conducted through rotary core drilling at two borehole (BH-3 and BH-4) locations (Figure 2), reaching depths of 50 meters. These locations were selected during the preliminary study based on geological context and accessibility. Drilling employed a single-tube rotary system suitable for retrieving cohesive soil samples in soft ground conditions. Undisturbed soil samples were extracted at selected depths using Shelby tubes, following the ASTM D1587 standard, to preserve the in-situ structure of the soft clay for laboratory testing.

SPT were performed at 2 meter intervals throughout the borehole depth, following ASTM D1586. The N-values obtained from SPT were used to assess soil strength profiles and to estimate the allowable bearing capacity of pile foundations using empirical correlations.

Where W is the weight of water and Ws is the weight of the dry soil after oven drying.

b. Bulk density (ASTM D7263)

Wet bulk density (γ_b) is the ratio of the weight of the soil particles, including water and air (W), to the volume of the soil (V). It can be expressed using the following formula:

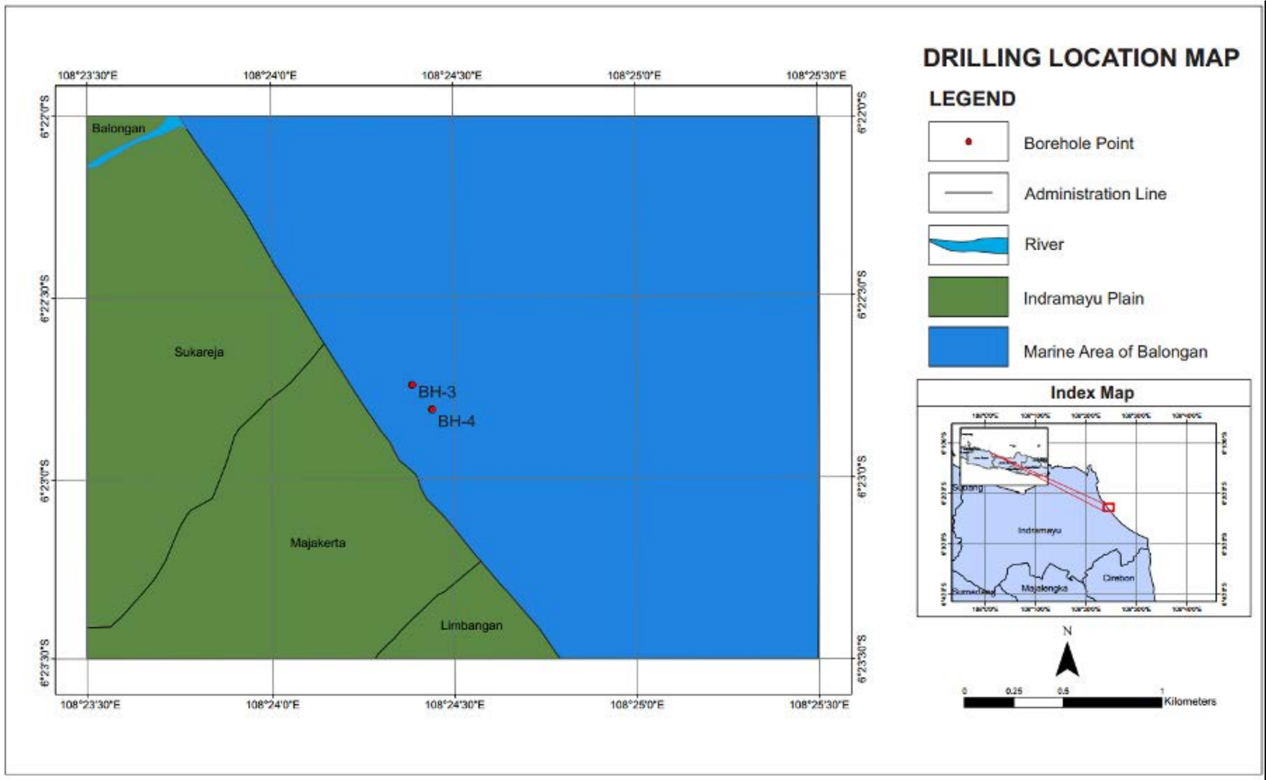

Figure 2. Drilling Location Map

2. Laboratory testing

Laboratory tests were carried out on both undisturbed and disturbed soil samples obtained from field drilling. The tests were conducted at the Soil Testing Laboratory of Politeknik Negeri Bandung. The following tests were performed:

a. Water content (ASTM D2216)

The water content test is used to determine the percentage of water in the sample. This test was conducted twice for each undisturbed sample (UDS).

Water content (W_c) can be calculated using the following equation:

$$w_c = \frac{W_w}{W_s} x\ 100\%$$

$$\gamma_b = \frac{W}{V}$$

Dry Bulk Density (γ_d) is the ratio of the weight of the soil particles (Ws) to the total volume of the soil (V). It can be expressed using the following formula:

$$\gamma_d = \frac{W_s}{V}$$

Solid Particle Density (γs) is the ratio of the weight of the solid soil particles (Ws) to the volume of the solid soil particles (Vs). It can be expressed using the following formula:

$$\gamma_s = \frac{W_s}{V_s}$$

c. Specific Gravity (ASTM D854)

The Specific Gravity (GS) test is conducted twice for each UDS. The purpose of this test is to obtain the value of Gs. Specific gravity (or specific weight) is the ratio between the mass (or weight) of dry soil particles (γ_s) and the mass (or weight) of distilled water occupying the same volume as the soil particles (γ_w). The parameter was computed as:

$$G_s = \frac{\gamma_s}{\gamma_w}$$

d. Atterberg Test (ASTM D4318)

Atterberg provided a method to describe the consistency limits of fine-grained soils by considering their water content. These limits include the liquid limit (LL), plastic limit (PL), shrinkage limit (SL), and plasticity index (LI).

Furthermore, Atterberg (1911, as cited in Hardiyatmo, 2002) divided the plasticity index limits and soil types (Table 1).

e. Direct Shear Test (ASTM D3080)

Table 1. Plasticity index limits

PI	Soil property	Soil types	Cohesion
0	Non-plastic	Sand	Non-cohesive
< 7	Slightly plastic	Silt	Partially cohesive
7-17	Medium plastic	Silty Clay	Cohesive
>17	Highly plastic	Clay	Cohesive

This test is conducted to examine UDS specimens obtained from drilling. The purpose of this test is to determine the cohesion parameter (c) and the internal friction angle (Φ). The testing is carried out using three test samples for each undisturbed sample, with different loading conditions applied to each.

f. Consolidation Test (ASTM D2435)

This test is conducted to obtain the compression parameters for the magnitude of settlement from a consolidation parameter, to estimate the rate of settlement.

All test procedures adhered to standard protocols, and multiple trials were conducted to verify data reliability. Results from field and laboratory data were integrated to evaluate the geotechnical behaviour of the subsurface materials and to support bearing capacity and settlement analyses.

Data Processing Stage

The data processing stage focused on interpreting the results of field and laboratory investigations to evaluate the geotechnical performance of the proposed port site. Two primary aspects were analyzed in this stage: (1) allowable bearing capacity of pile foundations, and (2) settlement estimation of soft clay layers. Analyses were conducted using both empirical correlations from SPT data and numerical calculations based on soil properties derived from laboratory tests.

Bearing Capacity Based on SPT Values

The ultimate bearing capacity of a pile (Q_u) is the sum of the ultimate tip resistance (Q_b) and the skin friction resistance (Q_s) between the pile shaft and the surrounding soil, expressed by the following equation (Meyerhof, 1976):

$$\begin{aligned} Q_u &= Q_b + Q_s \\ &= A_b\, f_b + A_s\, f_s \end{aligned}$$

Where:

A_b = Cross-sectional area of the pile tip (m² or ft²)

A_s = Surface area of the pile shaft (m² or ft²)

f_b = Average unit tip resistance (kN/m² or tons/m²)

f_s = Average unit skin friction (kN/m² or tons/m²)

The ultimate bearing capacity of a pile can be calculated empirically based on the SPT N-values. The tip resistance of the pile was calculated from SPT N-values using Meyerhof's equation;

$$Q_b = A_b\,(40N'')\, L_b/B \leq A_b\,(380N'')$$

Where:

N” = The statistical average value of the SPT numbers in the area approximately 8B above to 3B below the pile tip.

B = The width or diameter of the pile.

L_b/B = The ratio of the average depth of a point.

The skin friction resistance of the pile was calculated from SPT N-values using Meyerhof's equation (Bowles, 1993), which is:

$$Q_s = X_m\, N\, p\, L_i$$

Where:
X_m = 0.1 for small displacement pile
L_i = Thickness/length of the soil layer (m)
P = Perimeter of the pile (m)
N = Average SPT blow count (statistical mean)

Bearing Capacity Based on Laboratory Test Result

To determine the bearing capacity based on laboratory parameters, several factors need to be considered (Meyerhof, 1963, as cited in Hardiyatmo, 2010), as follows:

1. Overburden Pressure at the Foundation Base

$$P_o = D_f \gamma$$

Where:
P_o = Overburden pressure
D_f = Foundation depth
γ = Unit weight of the soil

2. Foundation Shape Factor

The value of s_c for any φ is:

$$s_c = 1 + 0.2\,(B/L)\,tg^2\,(45 + \phi/2)$$

The value of $s_q = s_\gamma$ for $\varphi \geq 10^o$ is:

$$s_q = s_\gamma = 1 + 0.1\,(B/L)\,)\,tg^2\,(45 + \phi/2)$$

The value of $s_q = s_\gamma$ for $\varphi = 0^o$ is 1

Where:
B = Diameter of the foundation (m)
L = Length of the foundation base (m)
φ = Internal friction angle
B/L = 1 (if the foundation is a pile)

3. Foundation Depth Factor

The value of d_c for any depth is:

$$d_c = 1 + 0.4\,(D/B)$$

The value of $d_q = d_\gamma$ for any depth is:

$$d_q = 1 + 2\,(D/B)\tan\phi\,(1-\sin\phi)^2$$

The value of d_γ for $\varphi = 0^o$ is 1.

Where:
D = Depth of the foundation (m)
B = Diameter of the foundation (m)

If D/B is greater than 1, then D/B should be replaced with arc-tan (D/B)

4. Load Inclination Factor

The value of $i_c = i_q$ for any φ is:

$$i_c = i_q = [1-(\delta^o/90^o)]^2$$

The value of i_γ for $\varphi \geq 10^o$ is:

$$i_\gamma = [1-(\delta^o/\phi)]^2$$

The value of i_γ for $\varphi = 0^o$ is 1

Where:
δ = Load inclination angle relative to the vertical axis.

By considering the shape of the foundation, the load inclination, and the shear strength of the soil beneath the foundation base, Meyerhof provided the following bearing capacity equation (Hardiyatmo, 2010):

$$Q_u = s_c\,d_c\,i_c\,cN_c + s_q\,d_q\,i_q\,P_o\,N_q + s_\gamma\,d_\gamma\,i_\gamma\,0.5\,B'\gamma N_\gamma$$

Where:
Q_u = Ultimate bearing capacity
N_c, N_q, N_γ = Bearing capacity factors for strip foundation
s_c, s_q, s_γ = Foundation shape factors
d_c, d_q, d_γ = Foundation depth factors
i_c, i_q, i_γ = Load inclination factors
B' = Effective foundation width
P_o = Overburden pressure at the foundation base
D_f = Foundation depth
γ = Unit weight of the soil

Settlement Analysis

Settlement was evaluated using two approaches: (1) calculations based on field-test-derived allowable bearing capacity, and (2) calculations based on laboratory-derived soil parameters. For allowable bearing capacity, Prakash et al. (1990, as cited in Budiono and Rahardjo, 2008) proposed:

$$S_t = (B/100) + ((Q_a\,L)/(A_p\,E_p))$$

Where:
S_t = Amount of settlement (m)
B = Pile diameter (m)
Q_a = Allowable bearing capacity (ton)
L = Pile length (m)
A_p = Pile cross-sectional area (m^2)
E_p = Elastic modulus

For the settlement calculation based on laboratory test analysis data, Terzaghi (as cited in Hardiyatmo, 2010) provides the following equation:

$$\sum Sc_n = \sum(\frac{Cc}{1+e_0}.H.\log\left(\frac{P_0+\Delta P}{P_0}\right))$$

Where:

S_c = Settlement (cm or mm)

C_c = Compression index (from laboratory consolidation tests)

H = Thickness of the compressible soil layer (m)

e_0 = Initial void ratio (from laboratory tests)

P_0 = Initial effective stress (kN/m²)

ΔP = Change in effective stress due to the applied load (kN/m²)

The magnitude of settlement that still meets the allowable bearing capacity criteria is less than 1 inch or 2.54 cm (Meyerhof, 1956, as cited in Hardiyatmo, 2010). Generally, clay soils experience much greater settlement compared to sand layers. Settlement is closely related to the thickness of the soft soil layer; the thicker the soft soil, the higher the settlement that will occur (Jusi et al., 2024).

RESULTS

Water Content

The laboratory water content results are presented in Table 2. The highest water content value at BH3 (88.41%) occurs at a depth of 30.00-35.55 m, while the lowest value (36.85%) is found at a depth of 20.00-20.55 m. The highest water content value at BH4 (94.97%) occurs at a depth of 15.00-15.55 m, while the lowest value (28.37%) is observed at a depth of 20.00-20.55 m. Both highest and lowest values are in clay lithology.

Table 2. Laboratory test results for water content

Depth	BH3	BH4
	w_c	
(m)	(%)	
2.00 - 2.55	62.17	81.21
5.00 - 5.55	85.74	90.69
10.00 - 10.55	80.84	78.27
15.00 - 15.55	67.85	94.97
20.00 - 20.55	36.85	28.73
25.00 - 25.55	42.47	34.37
30.00 - 30.55	88.41	35.93
35.00 - 35.55	65.81	35.75
40.00 - 40.55	65.81	42.46

Unit Weight

The laboratory unit weight results are shown in Table 3. The highest unit weight value at BH3 (1.950 ton/m³) occurs at a depth of 40.00-40.55 m, while the lowest value (1.527 ton/m³) is found at a depth of 2.00-2.55 m. The highest unit weight value at BH4 (1.908 ton/m³) is at a depth of 40.00-40.55 m, while the lowest value (1.513 ton/m³) is at a depth of 10.00-10.55 m.

Table 3. Laboratory test results for unit weight

Depth	BH3	BH4
	γ_{sat}	
(m)	(ton/m³)	
2.00 - 2.55	1.565	1.518
5.00 - 5.55	1.576	1.519
10.00 - 10.55	1.527	1.513
15.00 - 15.55	1.896	1.835
20.00 - 20.55	1.8	1.862
25.00 - 25.55	1.867	1.713
30.00 - 30.55	1.818	1.815
35.00 - 35.55	1.93	1.895
40.00 - 40.55	1.95	1.908

Specific Gravity

The results of the calculations show that the specific gravity of the soil/sediment at BH3 ranges from 2.13 to 2.83, while the specific gravity of the soil/sediment at BH4 ranges from 2.41 to 2.64 (Table 4).

Table 4. Laboratory test results for specific gravity

Depth	BH3	BH4
	Gs	
(m)	-	
2.00 - 2.55	2.52	2.52
5.00 - 5.55	2.48	2.48
10.00 - 10.55	2.45	2.45
15.00 - 15.55	2.46	2.46
20.00 - 20.55	2.57	2.57
25.00 - 25.55	2.83	2.83
30.00 - 30.55	2.13	2.13
35.00 - 35.55	2.66	2.66
40.00 - 40.55	2.51	2.51

Atterberg Limits

The plastic limit values in the study area vary with depth. At BH3, the range is between 28.87% and 36.46%, while at BH4, the range is between 28.88% and 43.42%. The plasticity index values in the study area also vary with depth. At BH3, the plasticity index ranges from 27.87% to 49.83%, while at BH4, the range is from 37.33% to 64.89%. According to Atterberg's classification, the soil in the study area can be categorized as having high plasticity and cohesive properties.

Based on the Unified Soil Classification System (USCS), it can be observed that most of the soils in the study area are inorganic clays with high plasticity (CH). However, some samples are classified as organic clays with high plasticity (OH).

Table 5. Laboratory test results for Atterberg test and soil classification

Depth	BH3				BH4				BH3	BH4
	Wn	PL	LL	PI	Wn	PL	LL	PI	Classification	
(m)					(%)				-	
2.00 - 2.55	62.17	32.03	59.9	27.87	81.21	43.14	93.17	50.03	CH	CH
5.00 - 5.55	85.74	36.96	75.13	38.17	90.69	31.03	86.89	55.85	CH	CH
10.00 - 10.55	80.84	35.52	76.37	40.85	78.27	32.76	97.64	64.89	CH	SM
15.00 - 15.55	67.85	35.58	64.11	28.53	94.97	43.42	87.39	43.97	CH	SM
20.00 - 20.55	-	-	-	-	28.73	32.08	67.64	35.56	SM	CH
25.00 - 25.55	42.47	36.46	86.29	49.83	34.37	31.81	90.89	59.08	CH	CH
30.00 - 30.55	88.41	32.56	88.95	56.39	35.93	38.12	94.61	56.49	CH	CH
35.00 - 35.55	65.81	28.87	68.88	40.01	35.75	32.15	82.32	50.17	CH	CH
40.00 - 40.55	65.81	30.93	74.88	43.95	42.46	28.88	66.2	37.33	CH	CH

Consolidation Test Results

From the Table 6, the average compression index value at BH3 is 0.382, with an average coefficient of consolidation (Cv) of 0.006 cm²/s. At BH4, the average compression index is 0.321, with an average coefficient of consolidation (Cv) of is 0.007 cm²/s.

Table 6. Laboratory test results for consolidation test

Depth	BH3 Cv	BH4 Φ	BH3 c	BH4 Φ	BH3 Cc	BH4 Cv	BH3 Cc	BH4 Cv
(m)	(kPa)	(°)	(kPa)	(°)		(cm^2/s)		(cm^2/s)
2.00 - 2.55	2.52	2.52	0.20	7.07	3.24	5.51	0.431	0.008
5.00 - 5.55	2.48	2.48	0.20	3.42	8.83	5.71	0.750	0.006
10.00 - 10.55	2.45	2.45	10.69	6.45	10.59	5.13	0.840	0.004
15.00 - 15.55	2.46	2.46	30.40	6.92	20.69	13.31	0.229	0.005
20.00 - 20.55	2.57	2.57	69.63	20.70	68.75	30.59	0.256	0.007
25.00 - 25.55	2.83	2.83	68.65	19.50	66.59	14.56	0.155	0.005
30.00 - 30.55	2.13	2.13	96.60	0.22	96.11	11.39	0.350	0.003
35.00 - 35.55	2.66	2.66	107.88	22.71	77.28	9.18	0.170	0.008
40.00 - 40.55	2.51	2.51	94.15	18.20	106.90	18.93	0.240	0.007

Bearing Capacity Calculation Based on SPT Values

Depth (D) / (L_b) = 18 meters
Diameter (B) = 0.4 meters
N SPT = 14
Average N SPT (N") = 21.65
Safety Factor (F_s) = 3

Area (A_b) = (π B²) / 4
= ((3.14) (0.4)²) / 4
= 0.1256 m²

Perimeter (P) = π B
= (3.14) (0.4)
= 1.256 m

End Bearing Resistance
$Q_b = A_b$ ((40) N") (L_b / B) ≤ A_b ((380) (N)) = 1033.46

Skin Friction Resistance
$Q_s = X_m$ N p L_i = 135.77

Ultimate Bearing Capacity (Q_u)
$Q_u = Q_b + Q_s$ = 116.92 tons

Allowable Bearing Capacity (Q_{all})
$Q_{all} = Q_u / F_s$ = 38.97 tons

Bearing Capacity Calculation Based on Soil Physical Properties

Depth (D) = 18 meters
Diameter (B) = 0.4 meters
N SPT = 14 N
Safety Factor = 3

Area (A_b) = (π (B²)) / 4
= ((3.14) (0.4)²) / 4
= 0.1256 m²

Perimeter (P) = π B
= (3.14) (0.4)
= 1.256 m

Soil Properties:
γ = 1.862 g/cm³ = 18.261 kN/m³
ϕ = 30.592°
c = 0.701 kg/cm² = 68.747 kN/m²
Nc = 32.67; Nq = 20.63; Nγ = 18.56

Overburden Pressure (P_o) = $D_f \gamma$
= (18) (18.261)
= 328.691 kN/m²

Foundation Shape Factor
sc = 1 + 0.2 (B/L) tan²(45 + ϕ/2)
= 1 + 0.2 (1) tan²(45 + 30.592/2)
= 1.4

sq = 1 + 0.1 (B/L) tan²(45 + ϕ/2)
= 1 + 0.1 (1) tan²(45 + 30.592/2)
= 1.637

Foundation Depth Factor
dc = 1 + 0.4 (D/B)
= 1 + 0.4 arc tan(D/B)
= 1 + 0.4 arc tan(18/0.4)
= 1.628

$d_q = d_\gamma = 1 + 2\,(D/B)\tan(\phi)\,(1 - \sin(\phi))^2$
= 1 + 2(18/0.4) tan(30.592) (1 - sin(30.592))²
= 1.91

Load Inclination Factor
$i_c, i_q, i_\gamma = 1$

Ultimate Bearing Capacity (Q_u)
$Q_u = s_c\, d_c\, i_c\, c\, N_c + s_q\, d_q\, i_q\, P_o\, N_q + s_\gamma\, d_\gamma\, i_\gamma\, 0.5\, B'_\gamma\, N_\gamma$
= 7977.88 kN/m² = 797.79 tons

Allowable Bearing Capacity (Q_{all})
$Q_{all} = Q_u / F_s$
= 797.79 / 3
= 265.93 tons

Port Design Calculation
Port Length (L): 285 m
Port Width (B): 161 m
Port Area (A): (285) (161) = 45,885 m²
Pile Diameter: 0.4 m
Pile Length (L_{pile}): 18 m
Pile Area (A_{pile}): 0.1256 m²
Slab Thickness (T): 0.3 m
Concrete Density ($\gamma_{concrete}$): 2.4 ton/m³
Pile Spacing: 5 m
Number of piles (J): (58) (4) = 232 piles
Vertical Load Assumption (V): 1000 ton
Slab Load Calculation:
$T\,L\,B\,\gamma_{concrete}$ = 33,037 ton
Pile Load Calculation:
$J\,A_{pile}\,L_{pile}\,\gamma_{concrete}$ = 979.08 ton
Total Load Calculation:
V_{tot} = 33,037 + 979.08 = 34,016 ton
Load per Unit Area (q):
V_{tot}/A = 34,016/45,885 = 0.741 ton/m^2

Settlement Calculation

The settlement calculation is performed to determine the amount of settlement if a port is constructed with a pile depth of 18 m and a diameter of 0.4 m. This calculation uses two methods;

1. Based on Field Test Results

The first calculation uses the minimum allowable bearing capacity (Qa) obtained from the field test results. Using this approach, the calculated settlement is 0.61 cm.

2. Based on Laboratory Test Results

The second calculation uses the settlement value derived from the laboratory tests. In this case, the calculated settlement is 1.2445 cm.

These results indicate the estimated settlement for the proposed pile foundation at the site based on the two different testing methods.

The combined data from field and laboratory investigations confirm that the Balongan site exhibits geotechnical conditions typical of soft coastal clay, which is characterized by low strength, high compressibility, and extended consolidation periods. The results emphasize the need for integrated design approaches that combine empirical and laboratory methods to more accurately evaluate foundation behaviour and performance.

DISCUSSIONS

Overall, the subsurface profiles at are predominantly composed of clay lithology. At BH-3, the depth from 0.00 to 16.45 meters consists of grayish-green clay, and from this depth to 50.00 meters, brownish-gray clay is found. Similarly, at BH-4, the depth from 0.00 to 18.45 meters consists of grayish-green clay, and from 18.45 meters to 50.00 meters, brownish-gray clay is encountered. In relation to the regional geology, grayish-green clay is classified as coastal deposits, while brownish-gray clay is classified as river deposits.

Correlation with the SPT values (Figure3) indicates a clear relationship between BH-3 and BH-4. The depth from 0.00 to 11.00 meters consists of clay with a very soft to soft consistency, with an SPT value interval ranging from 1/30 to 4/30. From 11.00 meters to 18.00 meters, the clay has a medium to very stiff consistency, with SPT values ranging from 5/30 to 30/30. From 18.00 meters to 50.00 meters, the clay becomes hard, with SPT values ranging from 35/30 to 61/30.

From the graphs (Figures 4 and 5), as well as Table 7 and Table 8, it can be observed that the maximum load capacity from the SPT values tends to be lower compared to the maximum load capacity based on physical and mechanical property values. In the latter, a sharp increase can be seen after reaching a depth of 18.00 meters. This can be explained by a significant increase in the internal friction angle and a change in the consistency of the SPT values from stiff to hard.

The substantial difference between the SPT values and the physical—mechanical property values could be attributed to several factors (Mitchell & Soga, 2005; Fernando et al., 2021), including the potential influence of external factors on the soil before analysis or inaccuracies during the laboratory analysis process (Agustian et al., 2024). On the other hand, the SPT values tend to be more accurate since they are directly tested in the field (Allen & Psarris, 2024).

Given this explanation, the allowable bearing capacity through the SPT values is considered more reliable as a reference. In addition, the SPT has proven reliable for sandy soil and well suited for cohesive soils (Truong & Duong, 2020).

The magnitude of soil settlement (Table 9) was calculated from the SPT values and laboratory analysis. The potential settlement of the soil or sediment, based on field testing with the SPT, is 0.61 cm. The planned port dimensions have been calculated, and the total load acting on the port is 0.741 tons/m². The total settlement of the soil layer beneath the foundation base, based on the laboratory test data, is 2.07 cm, with a consolidation time of 90% occurring over 12.91 years (Table 10). This settlement still meets the criteria for the allowable bearing capacity.

The settlement that still meets the allowable bearing capacity criteria is less than 1 inch or 2.54 cm (Meyerhof, 1956; Hardiyatmo, 2010). Therefore, the calculation results above still comply with the allowable bearing capacity criteria, meaning the settlement that will occur is very small.

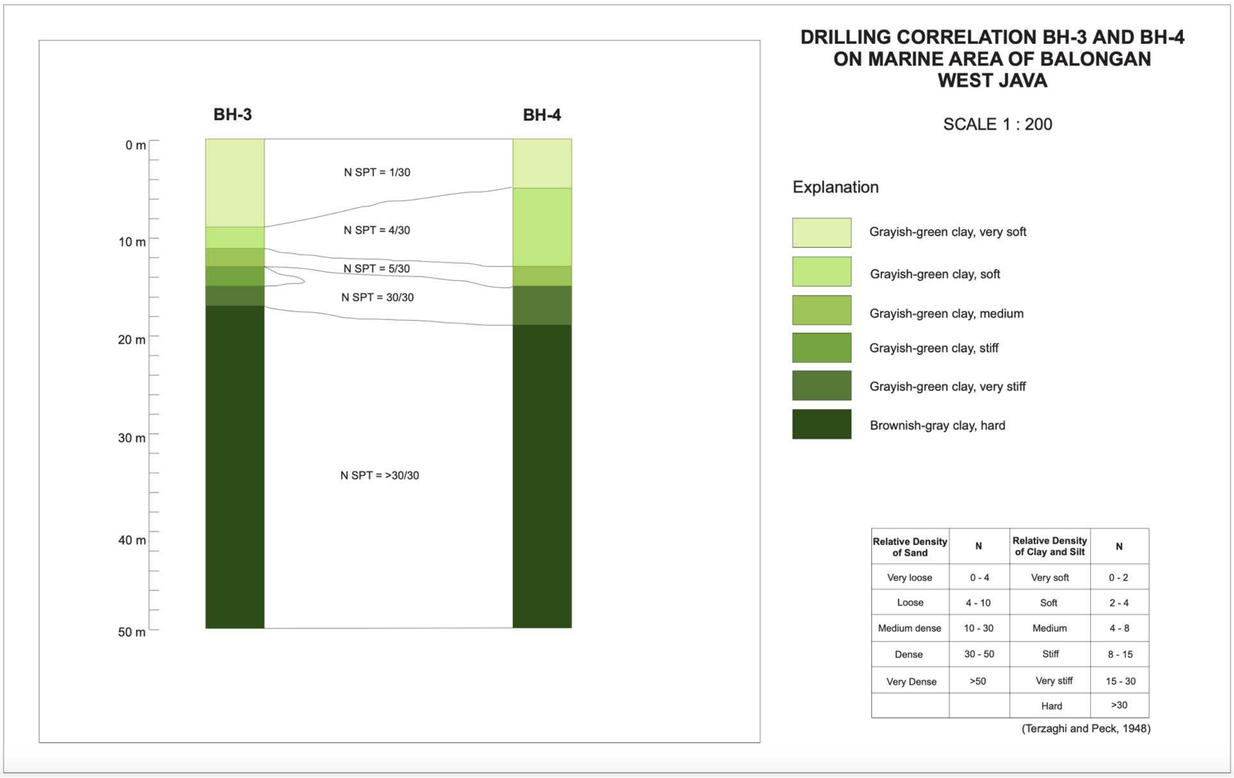

Figure 3. Drilling data correlation for BH-3 and BH-4 (PPPGL, 2006)

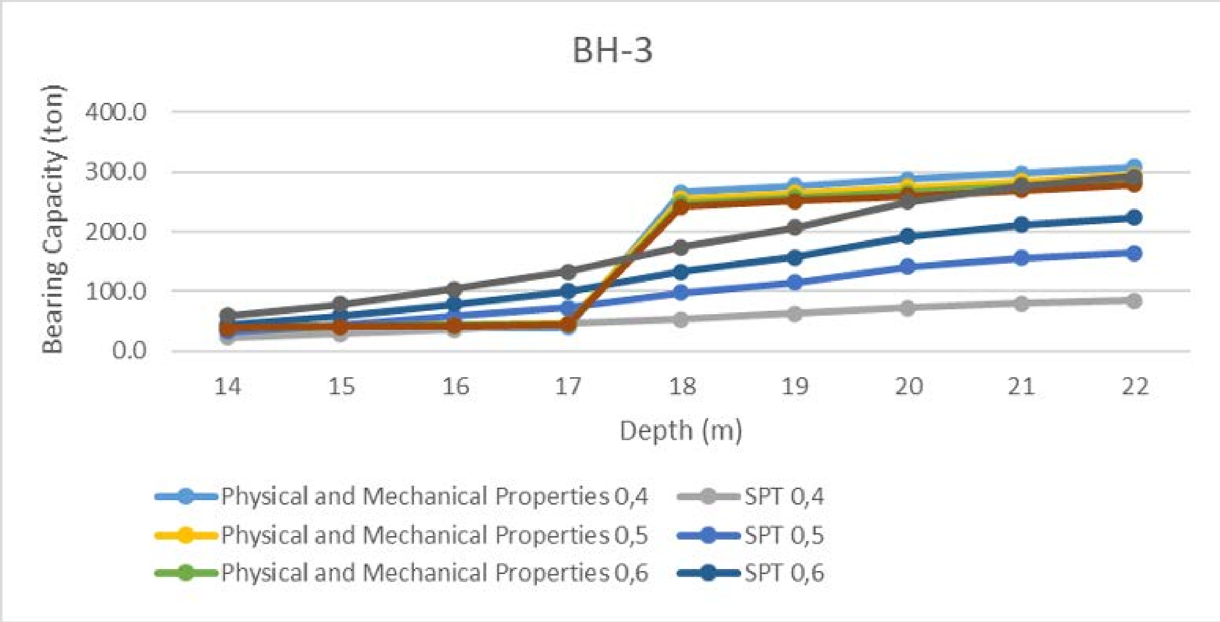

Figure 4. The graph of the allowable bearing capacity (BH3) versus depth

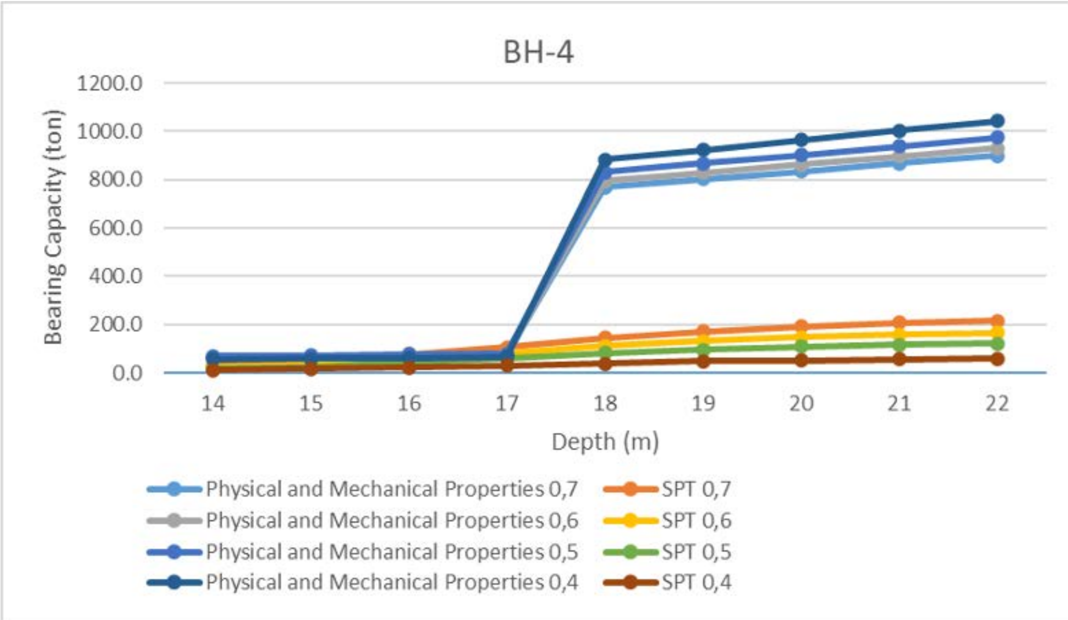

Figure 5. The graph of the allowable bearing capacity (BH4) versus depth

These findings underline the importance of employing a dual approach in geotechnical evaluation, integrating empirical field data with laboratory-derived parameters (Zdravković et al., 2021). Relying solely on either method may lead to underdesign or overdesign. To ensure reliable and efficient foundation performance, designers should consider combining both methods, complemented by probabilistic analysis or safety margins based on site conditions.

In addition, for areas subjected to high structural loads, such as quay walls or cargo terminals, ground improvement techniques may be necessary (Ruggeri et al., 2019). Preloading with vertical drains, vacuum consolidation, or staged construction can accelerate consolidation and reduce settlement-related risks (Indraratna et al., 2012; Zhang et al., 2018). Long-term instrumentation and monitoring are also recommended to validate design assumptions and accommodate post-construction behaviour.

Table 7. Maximum load capacity of BH3

Depth (m)	Maximum Load Capacity (Qall)							
	Diamater 0.4 m		Diamater 0.5 m		Diamater 0.6 m		Diameter 0.7 m	
	Physical & Mechanical Properties (ton)	N SPT (ton)	Physical & Mechanical Properties (ton)	N SPT (ton)	Physical & Mechanica l Properties (ton)	N SPT (ton)	Physical & Mechanical Properties (ton)	N SPT (ton)
14.00	35.664	22.989	40.488	33.170	39.688	45.124	39.117	58.852
15.00	37.219	29.591	42.387	43.290	41.529	59.509	40.917	78.248
16.00	38.774	35.869	44.286	57.931	43.371	79.197	42.717	103.690
17.00	40.330	45.589	46.185	73.236	45.212	100.972	44.517	133.071
18.00	265.929	53.068	254.697	98.240	247.256	133.546	241.980	174.071
19.00	276.432	63.372	264.566	115.192	256.701	157.516	251.123	206.269
20.00	286.934	72.150	274.435	141.282	266.147	191.881	260.266	249.927
21.00	297.436	80.311	284.303	155.462	275.593	211.721	269.409	276.369
22.00	307.939	84.382	294.172	164.401	285.038	223.739	278.552	291.895

Table 8. Maximum load capacity of BH4

Depth (m)	Maximum Load Capacity (Qall) (ton)							
	Diamater 0.4 m		Diamater 0.5 m		Diamater 0.6 m		Diameter 0.7 m	
	Physical & Mechanical Properties (ton)	N SPT (ton)	Physical & Mechanical Properties (ton)	N SPT (ton)	Physical & Mechanical Properties (ton)	N SPT (ton)	Physical & Mechanical Properties (ton)	N SPT (ton)
14.00	56.475	11.669	68.105	17.134	66.201	23.616	64.849	31.118
15.00	59.546	16.791	72.001	25.058	69.959	34.951	68.507	46.473
16.00	62.617	22.270	75.896	44.435	73.716	60.164	72.165	78.175
17.00	65.688	30.833	79.792	58.417	77.473	80.060	75.824	105.023
18.00	884.393	38.974	830.292	82.112	794.540	111.452	769.274	145.099
19.00	923.651	47.148	866.485	96.061	828.688	131.163	801.959	171.561
20.00	962.910	51.977	902.679	107.929	862.836	147.120	834.644	192.181
21.00	1002.168	56.385	938.872	116.151	896.984	158.546	867.329	207.328
22.00	1041.426	57.837	975.065	120.909	931.132	164.706	900.013	215.040

Table 9. Calculation of settlement value

Layer	Equilibrium Point	N	M	I	I	L	B	A	A/A1	ΔP (t/m^2)	ΔP (kg/m^2)	Po (t/m^2)	Po (kg/m^2)	Sc (m)	Sc (cm)
1	20	7.13	4.03	0.248	0.992	297	173	51381	0.893	0.657	656.744	37.24	37240	0.0124	1.24
2	24	5.94	3.35	0.247	0.988	305	181	55205	0.831	0.609	608.787	40.67	40666	0.0026	0.26
3	28	5.09	2.88	0.243	0.972	313	189	59157	0.776	0.559	558.916	47.72	47722	0.0018	0.18
4	32	4.45	2.52	0.24	0.96	321	197	63237	0.726	0.516	516.401	54.98	54982	0.0014	0.14
5	36	3.96	2.24	0.237	0.948	329	205	67445	0.680	0.478	478.129	62.40	62402	0.0015	0.15
6	40	3.56	2.01	0.232	0.928	337	213	71781	0.639	0.440	439.770	70.01	70008	0.0010	0.10
	Quantity														2.07

Table 10. Results of settlement duration calculation

%	T	Hdr	$(Hdr)^2$	Cv	t_s	t_m	t_{hour}	t_{day}	t_{year}
10	0.008	2400	5760000	0.012	3840000	64000	1066.67	44.44	
20	0.031	2400	5760000	0.012	14880000	248000	4133.33	172.22	
50	0.197	2400	5760000	0.012	94560000	1576000	26266.67	1094.44	3.00
90	0.848	2400	5760000	0.012	407040000	6784000	113066.67	4711.11	12.91

This study has certain limitations. The number of boreholes was limited to two, and the lateral variability of subsurface conditions across the broader site could not be fully assessed. Furthermore, advanced in-situ testing such as cone penetration testing (CPT), pressuremeter tests, or dilatometer tests was not conducted, which could have provided a more detailed and continuous soil strength profile (Robertson, 2012; Singh et al., 2016). Future investigations should incorporate these methods and increase spatial coverage to reduce uncertainty in geotechnical characterization.

CONCLUSSIONS

This geotechnical study offers valuable insights into the subsurface conditions and foundation behaviour at the proposed Balongan Port site in West Java. The site is predominantly underlain by soft, high-plasticity clay with low shear strength and high compressibility, which presents significant challenges for the design of deep foundations, especially for pile-supported port infrastructure.

Allowable bearing capacities derived from Standard Penetration Test (SPT) data were substantially lower than laboratory-based analyses, confirming the conservative nature of empirical methods and highlighting the need for integrated assessment. Settlement predictions from both SPT and laboratory methods fell within acceptable serviceability limits, but the projected time to reach 90% primary consolidation, approximately 12.9 years, raises concerns regarding long-term performance and the potential need for ground improvement interventions.

The study recommends that future foundation design at the Balongan site incorporate both field and laboratory data, and that designers consider consolidation control strategies such as preloading with vertical drains or vacuum consolidation. Long-term settlement monitoring should also be implemented as part of the construction and operational phase.

Despite limitations in spatial coverage and advanced in-situ testing, this study establishes a technical foundation for further investigation and design refinement. The findings contribute to early-stage decision-making and risk management strategies for developing safe, reliable, and sustainable port infrastructure in soft soil environments.

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