TECHNICAL AND INVESTIGATION OF SOIL MECHANICS IN THE SEI BALAI CLIFF REINFORCEMENT PROJECT, BATUBARA DISTRICT

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ABSTRACT

Cliff reinforcement is one of the important steps in overcoming geotechnical problems, especially in areas prone to landslides or slope instability. One of the areas experiencing this threat is Sei Balai, Batubara Regency. Soil mechanics testing and cliff reinforcement are important elements in the planning and implementation of construction involving cliffs or slopes, especially in areas prone to landslides. This research was conducted in the Sei Balai area, Batubara Regency at coordinates S-1: 3.211228˚ N, 99.593272˚ E and S-2: 3.210962˚ N, 99.592608˚ E. The soil in the area is dominated by sandy silt (silty Sand to Sandy Silt), which can affect slope stability. Medium to dense soil layers were obtained at each depth of the sounding point, where at point S-1, at a depth of 130.0 m to 25.0 m, the cone tip resistance (qc) was 40 to 175 kg/cm2, and at point S-2, at a depth of 17.8 m to 24.0 m, the cone tip resistance (qc) was 40 to 105 kg/cm2. The test results also showed that the soil bearing capacity with hand drill No. HB 1 at a depth of 2 m to 2.5 m was obtained at 3,748 tons/m2, and the soil bearing capacity with hand drill No. HB 2 at a depth of 2 m to 2.5 m was obtained at 4,253 tons/m2. This study forms the basis for planning cliff reinforcement to ensure the stability and safety of infrastructure around Sei Balai.

Keywords: ***C****one penetration test, slope reinforcement, soil bearing capacity.*

1. INTRODUCTION

The cliff strengthening project is one of the important steps in overcoming geotechnical problems, especially in areas that are prone to landslides or slope instability. One of the areas that experienced this threat was Sei Balai, Batubara Regency. This region, located along the river flow, has geological and hydrological characteristics that have the potential to cause cliff collapse, which can threaten the infrastructure and safety of the surrounding population. Planning for cliff reinforcement with soil mechanics needs to be carried out because the analysis of the physical and mechanical properties of the soil will provide an understanding of the bearing capacity of the soil, slope stability and appropriate reinforcement methods. Sondir testing and hand drill tests were carried out to obtain in-depth data related to cone tip pressure (qs) and soil bearing capacity (fs) at several strategic points. This data is a reference in determining the right reinforcement method, such as the use of soil retaining walls, drainage systems, or other reinforcement methods that are in accordance with soil conditions in Sei Balai. With proper studies, cliff reinforcement is expected to be able to maintain slope stability, minimize the risk of landslides, and protect infrastructure and communities around the area.

Based on the context of the problem, several problem formulations that need to be analyzed are obtained as follows:

1. What is the mechanical character of the soil in the Sei Balai area, especially at a critical depth for cliff reinforcement?
2. How much of the cone tip pressure (qs) and ground bearing capacity (fs) were at the location with sondir testing?
3. Is the dominant type of soil in this area able to withstand the load of the planned cliff reinforcement structure?
4. What is the most effective method of cliff reinforcement to apply in the site based on the results of the investigation of soil mechanics?

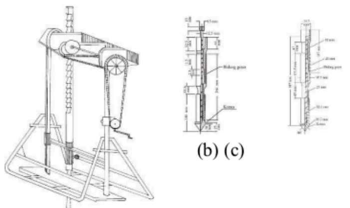
2. LITERATURE REVIEW

2.1. Common

The method of implementation of this soil investigation work includes observation in the field and conducting soil testing on site. From this field data, an evaluation was carried out to be presented in a report containing the results of the soil investigation. The field work consists of field observation and the implementation of static cone penetration tests. Field observation aims to find out the condition of the field and information from the surrounding residents, especially related to the depth of the hard soil at the location. Meanwhile, the penetration test was carried out to obtain the value of the edge thanan (qc) and friction resistance (fs) of the soil as parameters in the calculation of soil carrying capacity.

2.2. Sondir/cone penetration (CPT)

Sondir, or better known as Cone penetration test (CPT), is a soil testing method used to investigate the mechanical properties of soil below the surface, detect hard soil layers that will support construction loads, determine soil profiles for foundation design and planning, assess soil shear strength and measure edge resistance *(Cone Resistance)* and adhesion resistance *(Friction Resistance)* and determine the liquefaction potential of sandy soils in earthquake-resistant construction designs. The standard used for testing (CPT) in Indonesia refers to SNI 2827:2008 concerning sondir testing methods in the field for soil investigation and several 2008 international standards. The sondiri used has a capacity of 2.50 tons with a steel cone-shaped tool tip that has an angle of 60°, the sondir has a diameter of 35.7 mm with a cross-sectional area of 10 cm2. The use of this tool is less efficient for soil with dense characteristics such as sand, gravel, or rocky, because the cone will experience resistance when penetrating these types of soil (Hardiyatmo, 2020a). The testing process using a sondir is carried out by pressing the pipe and the sondir eye separately, either with mechanical or manual pressing, with a penetration speed of less than 10 mm/second. Measurements of tip resistance and friction were measured using a manometer every 20 cm penetration interval. The test will be stopped if the tip resistance value has exceeded 200 kg/cm2 (Hardiyatmo, 2020a, 2020b).



(b) (c)

(a)

Figure 2-1 Penetration Test Equipment: a) Sondir Test Equipment (CPT); b) Sondir Conus in Stressful Conditions; c) Conus Sondir on the Unfolding State (National Standardization Agency, 2008; Silitonga and Qarinur, 2022).

2.3. Relative density level

The relative density of sondir test results can be categorized based on the classification listed in table 2.1.

Table 2-1 Relative Density of Soil on Sondir Testing

Cone Tip Resistance Value (kg/cm2)	Relative Density
0 – 16	Very loose
16 – 40	Loose
40 – 120	Keep
120 – 200	Dense
> 200	Very dense

2.4. Supporting Capacity of Shallow Foundations

To calculate the bearing capacity of a shallow foundation, some common formulas that are often used are as follows:

a. **L'Herminier (1953)** conducted field loading experiments on dense sandy soils. The results of the experiment were compared with the conical pressure of the sondir test. Based on the experiment, it can be concluded that the bearing capacity of a shallow foundation located at a depth of 1 meter is about one-tenth of the pressure of the sondir test cone. However, this formula does not apply to foundations that are too shallow, too deep, or have unusual dimensions.

b. **Mayerhof (1956)** gave a formula for calculating the bearing capacity of foundations in sandy soils as follows:

$$q_{all} = \frac{q_c}{40} B\left(1 + \frac{D}{B}\right) \quad (1)$$

Clay soil:

$$q_{all} = \frac{q_c}{80} B\left(1 + \frac{D}{B}\right) \quad (2)$$

c. Terzaghi formulated the calculation of the bearing capacity of a shallow foundation laid on clay as follows:
Column foundation:

$$q_{ult} = 5{,}7\, c_u \quad (3)$$

Local foundations :

$$q_{ult} = 6{,}8\, c_u \quad (4)$$

2.5. Bearing Capacity of the Pole Foundation

2.5.1. Bearing Capacity of the Pole Foundation

Since the resistance of the sondir tip changes with depth, the *qc value* at the end of the pole is taken from a certain range. Here are some equations used to calculate the bearing capacity of a pile foundation.

a. Schertmann Method

$$Q_b = \left(\frac{q_{c1} + q_{c2}}{2}\right) \quad (5)$$

b. Van der Veen's Method

$$Q_b = \frac{1}{2}\left(\frac{1}{2}(q'_{kd} + q''_{kd}) + q'''_{kd}\right) \quad (6)$$

c. Al Alusi HR Method (1977)

$$Q_c = \frac{1}{4} q_{c1} + \frac{3}{4} q_{c2} \quad (7)$$

d. Meyerhof Method (1976)

$$fb = \omega 1 \omega 2 qc \quad (8)$$

e. Begemann (1965)

$Qc = cuNk$ (9)

f. deRuiter and Berigen

$fb = 5\ Cu$ Limited $Fb \leq 150$ kg/cm2 (10)

$cu = qc / nk$ (11)

2.5.2. Bearing capacity of the pole blanket (Side Friction)

For the non-cohesive, Scmertmann (1975) put forward the following equation:

$$Qs = K\left[\sum_{z=0}^{8d}\left(\frac{z}{8d}\right) f_s A_s + \sum_{z=8d}^{L} f_s A_s\right] \quad (12)$$

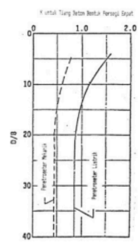


Figure 2-2 Chart K

If the length of the pole is equal to or greater than 8d, then the equation can be simplified to:

$$Qs = K\left[\frac{1}{2}(f_s A_s)_{0-8d} + (f_s A_s)_{8d-L}\right] \quad (13)$$

For cohesive soil, the equation can be used:

$Qs = \alpha fsA$ (14)

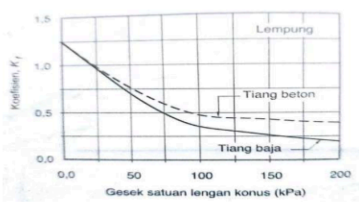


Figure 2-3 Kf coefficient in clay

Meyerhof (1976) recommended a calculation of the bearing capacity of the pile blanket for piles, where the friction resistance of the unit can be determined using one of the following equations:

$Fs = Kfqf$ with $Kf = 1$ (15)

or, if the cone side friction resistance measurement is not performed:

$Fs = KCQC$ with $Kc = 0.005$ (16)

The friction bearing capacity of piles according to the Begemann Method (1965) is as follows:

$Qf = Fs$ (17)

The unit friction resistance in cohesive soils according to the deRuiter and Beringen methods is determined from the value of undrained cohesion ($cu = qc / 20 = 0.05qc$) namely:

$$f_s = \alpha c_u = \left(\frac{q_c}{N_k}\right) = 0{,}05\, q_c \quad (18)$$

2.5.3. Pile support capacity

In general, the ultimate net bearing capacity (*Qu*) can be calculated using the following equation:

$Qu = Qb + Qs = Abfb + Asfs - Wp$ (19)

There are various formulas that can be applied to calculate the bearing capacity of piles, including:

1. In Indonesia, basic equations are usually used to calculate the bearing capacity of poles, namely:

$$Q = \frac{Q_b A_p}{3} + \frac{T_f \times R}{5} \quad (20)$$

2. Based on the explanation of Meyerhof (1956), the calculation of soil carrying capacity can be done using the following equation:

$$Q = \mathrm{mQbA} + \mathrm{nTfR} \quad (22)$$

2.6. Bearing Capacity of Drill Pile Foundation

The calculation of the bearing capacity for the foundation of the drill pile is carried out by the end *bearing method* and does not include the total soil adhesion factor (*Tsf*) where the soil adhesion is temporarily considered to be not working due to the implementation of drilling. The equations used are as follows:

$$Qa = qcA \quad (24)$$

$$Q_{a\,ijin} = \frac{q_c A}{SF} \quad (25)$$

3. IMPLEMENTATION METHOD

3.1. Research Location

The administrative location of the research is in Lima Laras Village, Nibung Hangus/Tanjung Tiram District in Batubara Regency in the North Sumatra Province area, precisely which can be reached by road with a distance of 110 km with a 2-hour trip from Medan as seen in figure 4 below.± ±

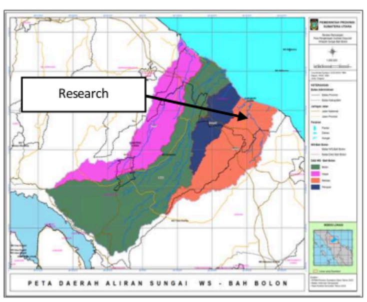


Figure 3-4 Map of the research location

Figure 3-5 Soil research location (sondir)

Table 3-2 Sondir Test Point Coordinates

Yes	Point Name	Coordinates
1	S-1	3,2112228LU° 99,593272BT°
2	S-2	3,210962LU° 99,592608BT°

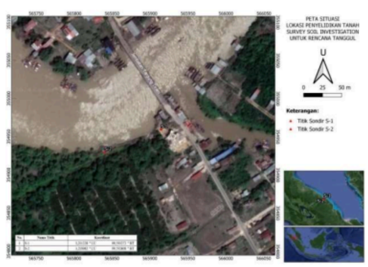
3.2. Sampling Process

3.2.1. Using Hand Bore

The implementation of Hand Bore work was carried out to a depth of 4.00 meters from the original land surface, carried out at 2 (two) points. The equipment used is a spiral type bore bore tool. At a depth of 2.00 meters - 2.50 meters, undisturbed soil samples (UDS) and disturbed soil samples (DS) were taken, and groundwater level observations were also carried out manually. The results from the undisturbed soil sampling (UDS) were taken to the laboratory for testing.

3.2.2. Undisturbed Soil Sampling

Undisturbed soil sampling (UDS) was carried out using a thin steel tube with a diameter of 73.00 mm, a length of 50.00 cm, carried out as many as 1 (One) tube at each point. At a depth of 2.00 meters - 2.50 meters, undisturbed soil sampling (UDS) was carried out using a thin steel tube (Thin Tube Sampler) lowered by connecting to the bore



handlebar. to maintain no change in the original condition of the groundwater content. Undisturbed soil samples (UDS) are taken to the laboratory for testing to determine the description and properties of the characteristics and classification of the soil.

3.3. Laboratory Tests

3.3.1. Testing of Property Index

1. Granular Gravity Index

   ASTM Standard D – 854-72, soil specific gravity is the ratio between the weight of soil grains and the weight of distilled water that has the same volume. This measurement is usually done with tools such as a picnometer, balance sheet, and other tools, which are denoted by the GS symbol.

2. Sieve Analysis Test

   ASTM D – 421-72 standard, the properties of soils are generally determined by their grain size, which is also the basis in the classification and naming of soil types. The grain size can be depicted through a graph known as a gradation curve graph or a grain divider curve graph.

3. Atterberg Consistency (Atterberg Limit Test)

   Liquit limit, Plastic limit, Plastic index, ASTM Standard D – 421, D – 423, D-2217, if the sample of soil that has fine grains, such as clay or silt, is mixed with water until it reaches a liquid state, then allowed to dry, the soil will experience some of the following conditions:

   - Liquid Limit
   - Plastic Limit
   - Semi-plastic condition (Plastic Index)

4. Moinsture Content

   According to ASTM D-2216 standard, the water content in the soil varies depending on the size and number of poro-pores present. The percentage of water contained affects the ability of the soil to withstand the given load. If the amount of water in the soil is too high, this can lead to problems, such as water seepage when the soil is loaded.

3.3.2. Engineering Properties Testing

1. Unit Density (Natural Density/Unit Weight)

   The unit weight of the soil is very important to determine the amount of soil required in a given unit of compaction. This calculation is useful for analyzing the strength of the soil based on its contents. Soil that has a higher unit weight shows better conditions in supporting the load applied to it.

2. Unconfined Compression

   Based on the ASTM D-2166-72 standard, free compressive strength is defined as the wide unity axial load received by the axial force when it is subjected to collapse, or when the axial strain reaches 20%.

3. Direct Shear Test

   Standard ASTM Shear Test, shear strength measurement can be done directly. A sample of the tested yanakan will be installed in the appliance and subjected to a constant vertical voltage (normal voltage).

4. Consolidation Test

Sntandard ASTM D-2435, in general, the soil has a high compression rate, which is due to the large pore size. Therefore, if the soil is subjected to heavy loads, this can result in a drop in the foundation, which in turn can lead to damage or even collapse to the building structure. From this experiment, it can be determined:

- Consolidated Efficiency , CV (cm/ sec).
- Volume Reduction Coefficient, mv.
- Koeffisien Permeability, K (cm/sec).
- Compression Index, Cc.

4. RESULTS AND DISCUSSION

4.1. Groundwater Level

The condition of the soil investigation location topographically and observation is a relatively flat area. Based on the data from the results of field tests in the form of Sondir testing at two points, it was found that in general the classification of the soil is Silty Sand to Sandy Silt. Complete information about the results obtained can be seen in table 4.3.

Table 4-3 Groundwater Surface Depth

No.	Point Name	Depth (m)
1	S-1	3,00
2	S-2	2,20

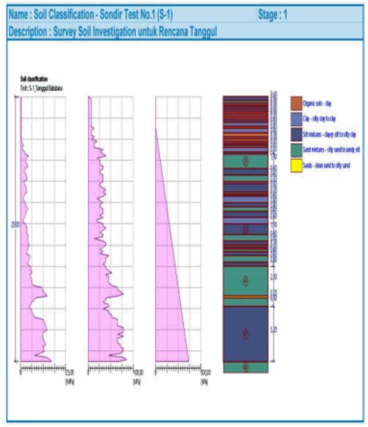


Figure 4-6 Soil Classification On Sondir No. 1 (S1) Testing

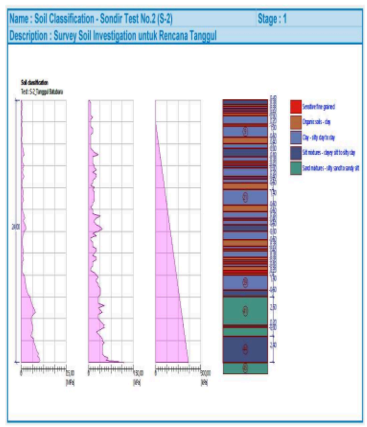


Figure 4-7 Land Classification in Sondir Permit No.2 (S2)

4.2. End Resistance and Total Shear Resistance

The results of the Sondir test showed that the tip resistance value (qc) reached a value greater than 200 kg/cm2 and the highest total friction resistance (Tf) at the shallowest position at a depth of 23.8 m (S-2) and the deepest at a depth of 25.0 m (S-1).

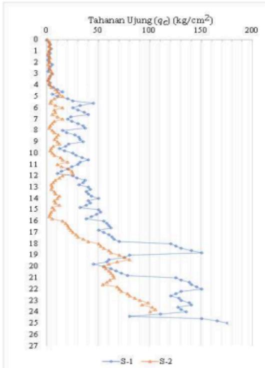


Figure 4-8 Graph of Edge Resistance (QC) of sondir test results

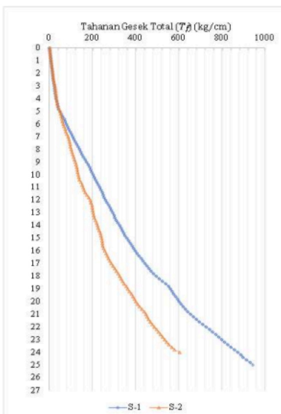


Figure 4-9 Graph of Friction Resistance (Tf) of sondir test results

Table 4-4 Maximum Total Edge Resistance and Friction Resistance

Yes	Point Name	S-1	S-2
1	Depth (m)	25	23,8
2	Prisoner End (*qc*) (kg/cm2)	175	105
3	Depth (m)	25	24
4	Prisoner Gesek (*Tf*) (kg/cm)	941,33	601,33

4.3. Relative Density

Soil with relatively dense density conditions is found at depths of more than 21 meters.

Table 4-5 Maximum Total Edge Resistance and Friction Resistance

Kedalaman	S-1		S-2	
(m)	Tahanan Ujung (q_c) (kg/cm^2)	Kepadatan Relatif	Tahanan Ujung (q_c) (kg/cm^2)	Kepadatan Relatif
0,2	2	Sangat Lepas	2	Sangat Lepas
1	2	Sangat Lepas	2	Sangat Lepas
2	2	Sangat Lepas	3	Sangat Lepas
3	5	Sangat Lepas	5	Sangat Lepas
4	2	Sangat Lepas	3	Sangat Lepas
5	10	Sangat Lepas	15	Sangat Lepas
6	25	Lepas	15	Sangat Lepas
7	20	Lepas	15	Sangat Lepas
8	15	Sangat Lepas	5	Sangat Lepas
9	35	Lepas	10	Sangat Lepas
10	25	Lepas	5	Sangat Lepas
11	30	Lepas	10	Sangat Lepas
12	25	Lepas	15	Sangat Lepas
13	40	Sedang	5	Sangat Lepas
14	50	Sedang	10	Sangat Lepas
15	50	Sedang	7	Sangat Lepas
16	55	Sedang	15	Sangat Lepas
17	55	Sedang	25	Lepas
18	120	Padat	50	Sedang
19	80	Sedang	70	Sedang
20	55	Sedang	55	Sedang
21	125	Padat	65	Sedang
22	150	Padat	70	Sedang
23	130	Padat	90	Sedang
24	135	Padat	100	Sedang
25	175	Padat		

4.4. HAND BORE (BOR TANGAN)

Based on the results of the Hand Bore test, the structure of the soil layer contained in the research location area is as follows:

1. Hand Drill Location No : HB-1

• Layer 1 (One)
Depth 0.00 – 1.80 meters
Description : Clay berlunau
Colour : Bright gray
Stength :Soft
Plasticity :Keep
Moist Content :Keep

• Layer 2 (Two)
Depth 1.80 – 2.40 meters
Description : Sandy clay lanau mud inserts battered wood
Colour : Bright gray
Stength :Soft
Plasticity :Keep
Moist Content :Keep

• Layer 3 (Three)
Depth 2.40 - 3.00 meters
Description : Sand with scallop shards inserted
Colour : Dark grey
Stength :Low
Plasticity :Keep
Moist Content :Keep

• Layer 4 (Four)
Depth 3.00 - 4.00 meters
Description : Sand with scallop shards inserted
Colour : Dark grey
Stength :Low
Plasticity :Low
Moist Content :Low

2. Hand Drill Location No: HB-2

• Layer 1 (One)
Depth 0.00 - 2.20 meters
Description : Clay berlunau
Colour : Bright gray
Stength :Soft
Plasticity :Keep
Moist Content :Keep

• Layer 2 (Two)
Depth 2.20 - 3.00 meters
Description : Sandy clay with weathered wood inserts and shell fragments
Colour : Dark gray blackish
Stength : Very Soft
Plasticity :Keep
Moist Content :Keep

• Layer 3 (Three)
Depth 3.00 - 3.50 meters
Description : Sand with scallop inserts
Colour : Dark Brown
Moist Content :Low

• Layer 4 (Four)
Depth 3.50 - 4.00 meters
Description : Coarse sand with shellfish flake inserts
Colour : Dark grey
Moist Content :Low

4.5. LABORATORY TESTING

The results of laboratory tests on undisturbed soil samples can be seen in the appendix of the Summary of Laboratory Test Results (attached).

SUMMARY OF LABORATORY TEST RESULT

Bore Hole	BH	No	HB - 1	HB - 2			
Sample Depth	(M)	(Meter)	2.00 - 2.50	2.00 - 2.50			
Sample Type	UDS	UDS	UDS	UDS			
Natural Density Test							
- Wet Density	(Wet γ)	(gr/cc)	1.675	1.650			
- Dry Density	(Dry γ)	(gr/cc)	1.285	1.289			
- Void Ratio	(e)	(%)	1.0078	0.9904			
- Porosity	(n)	(%)	0.5019	0.4976			
- Saturation	(Sr)	(%)	77.70	72.62			
Specific Gravity Test		(Gs)	2.580	2.565			
Moisture Content Test		(%)	30.35	28.04			
Atterberg Limit Test							
- Liquit Limit	(L L)	(%)	36.15	35.22			
- Plastic Limit	(PL)	(%)	16.05	13.09			
- Plastic Index	(PI)	(%)	20.10	22.13			
Soil Classification							
- AASHTO Standard			A-6	A-6			
- U S C S Standard			CL	CL			
Sieve Analysis Test							
- No 4 Sieve Size	Passing	(%)	100	100			
- No 10 Sieve Size	Passing	(%)	98.20	98.04			
- No 40 Sieve Size	Passing	(%)	94.05	93.25			
- No 200 Sieve Size	Passing	(%)	89.55	88.10			
Uncon Fined Test							
- Qu Maximum	(Qu)	(kg/cm2)	0.180	0.195			
- Strain Maximum	(Str)	(%)	6.00	6.50			
Direct Shear Test							
- Internal Friction	(Ø)	(Degree)	7° 36' 59.38"	8° 3' 11.61"			
- Cohesion	(C)	(kg/cm2)	0.143	0.157			
Consolidation Test							
- Coef of Consol	(Cv)	10^{-3}	1.69E-03	1.75E-03			
- Coeffisien Index	(Cc)	(-)	0.4038	0.2422			
- Permeability	(K)	(10^-7)	7.85E-08	1.02E-07			
Pekerjaan : SURVEY SOIL INVESTIGASI UNTUK RENCANA TANGGUL				Lokasi : DESA LIMA LARAS KECAMATAN TANJUNG TIRAM, KABUPATEN BATUBARA, SUMATERA UTARA			Page :

4.6. BEARING CAPACITY OF THE FOUNDATION

The bearing capacity of the foundation was obtained through data analysis from static cone testing (sondir). This calculation of carrying capacity value considers the type of foundation to be applied, both shallow and deep foundations, as well as the characteristics of the soil at the construction site, including granular soil and cohesive soil.

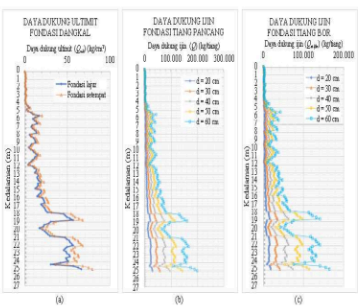


Figure 4-10 Sondir Point Bearing Capacity S-1: a), Ultimate Bearing Capacity of Shallow Foundation; (b) The Carrying Capacity of the Pile Foundation Permit; c) The Carrying Capacity of the Drilling Pole Foundation Permit.

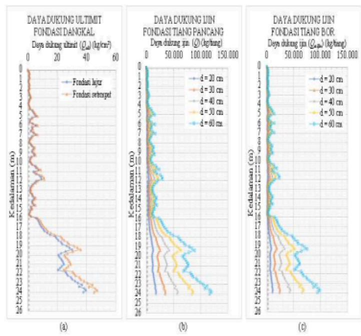


Figure 4-11 Sondir Point Bearing Capacity S-2: a), Ultimate Bearing Capacity of Shallow Foundation; (b) The Carrying Capacity of the Pile Foundation Permit; c) The Carrying Capacity of the Drilling Pole Foundation Permit.

4.7. CARRYING CAPACITY OF LOCAL FOUNDATIONS

Testing has been carried out in the laboratory using an undisturbed soil sample (Undisturbed Sample). From the test results, the bearing capacity of the foundation, such as the local foundation (square footing), is calculated using the Terzaghi formula which is explained as follows:

$$q_{ult} = 1{,}3 \,.\, C \,.\, Ne + Dt \,.\, Nq + 0{,}4 \,.\, y \,.\, B \,.\, Ny$$

$$qa = q_{ult} \,/\, FK$$

Where:

q ult = Ultimate bearing capacity (t/m2)
q a = Permit carrying capacity (t/m2)
C = Cohesion (t/m2)
O = Ground sliding angle (degrees)
y = Volumetric weight of the soil (t/m3)
Dt = Depth of foundation (meters)
B = Foundation width (meters)
N.N.Ny = Carrying capacity factor
FK = Safety Factor (value 3 to 5)
Dw = Groundwater level (m)

The carrying capacity of the local foundation can be seen in the following attachment:

Bearing Capacity of Soil for Square Footing, 1.00 M x 1.00 M

Bore no	Depth (m)	∂ (ton / m3)	C (ton / m2)	Ø (deg)	Nc (-)	Nq (-)	N∂ (-)	Df (m)	B (m)	Dw (m)	F Dw (-)	Sf	Qu (ton/m2)	q ult (ton/m2)
HB-1	2.00-2.50	1.675	1.430	7°	8.22	2.04	1.38	2.00	1.00	0.20	0.50	3.0	22.488	**3.748**
HB-2	2.00-2.50	1.650	1.570	8°	8.68	2.26	1.32	2.00	1.00	0.20	0.50	3.0	25.517	**4.253**

5. CONCLUSIONS AND SUGGESTIONS

5.1. Conclusion

Based on the results of field tests and analyses that have been carried out, the following are conclusions and suggestions that can be taken into consideration in making decisions regarding soil conditions in the research location:

1. The conditions at the work site topographically and observation are relatively flat areas.
2. In general, the soil is dominated by Silty Sand to Sandy Silt. The soil layer from the surface has a very low conical value and begins to increase in height gradually at a depth of 21 m.
3. Medium to dense soil layers are found at each depth of Sondir point as follows:
4. Hand Drill No. HB 1 obtained a soil carrying capacity at a depth of 2 m to a depth of 2.5 m of 3,748 tons/m2

Yes	Name Point	Depth (m)	Prisoner End (*QC*) (kg/cm2)
1	S-1	13.0 to 25.0	40 to 175
2	S-2	17.8 to 24.0	40 to 105

5. Hand Drill No. HB 2 obtained a soil carrying capacity at a depth of 2 m to a depth of 2.5 m of 4,253 tons/m2

5.2. Suggestion

1. The main building may be recommended to use a deep foundation such as a pile foundation or drill post by placing the foundation foundation on a solid or very dense layer of soil.
2. It is necessary to perform calculations to obtain the maximum construction load value (Pmax) as follows:
   - Perform construction static load calculations.
   - Perform dynamic load calculations.
   - Taking into account the influence of earthquakes (Seismic force load).

3. In the selection of foundation alternatives, the following are recommended:
   - Perform construction static load calculations.
   - Perform dynamic load calculations.
   - Taking into account the influence of earthquakes (Seismic force load).

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