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ANALISIS PENGGUNAAN PONDASI *BORE PILE* PADA LAPISAN PASIR ALUVIAL DAN TUF PASIRAN DI PROYEK BENDUNGAN MARGATIGA

ANALYSIS THE USE OF BORE PILE FOUNDATION ON ALLUVIAL SAND AND TUFFACEOUS SANDSTONES AT MARGATIGA DAM PROJECT

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ABSTRACT

The elevation of the dam building foundation plan is at an elevation of +06.00 and +08.00, the results of sub-surface tests including the Standard Penetration Test (SPT) and Pressure Meter Test (PMT) show that at these elevations the SPT value is < 20 or categorized as loose rock and the PMT results shows a lateral pressure of <3 – 10 MPa or categorized as very soft rock - soft rock. The results of the sub-surface test show that the type of rock constituent in the dam building consists of layers of alluvial sand and tuffaceous sandstone. Based on the technical specifications, the foundation must be at SPT value > 50 or in very dense rock layer, if the dam building foundation is still built at the elevation of the foundation plan, it can cause the building to become unstable due to the rock layer at that elevation not able to support the weight of the building. SPT values >50 was found at an elevation of +01.00 - -02.00 or 7 to 10 meters from the design elevation of the foundation. Furthermore, an analysis is carried out based on the results of sub-surface test to determine the method of strengthen the foundation. The results of the analysis show that the bore pile foundation is effective as a method of foundation strengthen. The analysis includes the analysis of the ultimate bearing capacity, the axial allowance bearing capacity of the pile foundation and the bearing capacity of the pile group foundation. Based on the analysis, strengthening the foundation of intake area requires 156 piles with a depth of 19 meters, the spillway area requires 196 piles with a depth of 17 meters and the retaining wall area requires 64 piles with a depth of 15 meters.

Keywords: *Sub-surface test; alluvial sand; tuffaceous sandstone; bore pile; bearing capacity*

ABSTRAK

Elevasi rencana pondasi bangunan bendungan berada di elevasi +06.00 dan +08.00, hasil pengujian bawah permukaan meliputi Standard Penetration Test (SPT) dan Pressure Meter Test (PMT) menunjukan bahwa pada elevasi tersebut nilai SPT < 20 atau masuk dalam kategori batuan lepas dan hasil PMT menunjukan tekanan lateral sebesar <3 – 10 MPa atau masuk dalam kategori batuan sangat lunak - batuan lunak. Hasil pengujian bawah permukaan menunjukan jenis batuan penyusun pada bangunan bendungan terdiri atas lapisan pasir aluvial dan tuf pasiran. Berdasarkan spesifikasi teknis, pondasi harus berada pada nilai SPT >50 atau pada lapisan batuan yang sangat padat, apabila pondasi bangunan bendungan tetap dibangun pada elevasi rencana pondasi maka dapat menyebabkan bangunan menjadi tidak stabil karena lapisan batuan pada elevasi tersebut tidak mampu menahan beban bangunan di atasnya. Nilai SPT >50 ditemukan pada elevasi +01.00 - -02.00 atau 7 hingga 10 meter dari elevasi rencana pondasi. Selanjutnya dilakukan analisis berdasarkan hasil pengujian bawah permukaan untuk menentukan metode perkuatan pondasi. Hasil analisis menunjukan bahwa pondasi bore pile efektif sebagai metode perkuatan pondasi. Analisis yang dilakukan meliputi analisis daya dukung pondasi tiang, daya dukung ijin aksial dan daya dukung pondasi kelompok tiang. Berdasarkan analisis yang dilakukan, perkuatan pondasi area pengambilan membutuhkan 156 tiang dengan kedalaman 19 meter, area spillway membutuhkan 196 tiang dengan kedalaman 17 meter dan area dinding penahan tanah membutuhkan 64 tiang dengan kedalaman 15 meter.

Kata Kunci: *Pengujian bawah permukaan; pasir aluvial; tuf pasiran; bore pile; kapasitas daya dukung*

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INTRODUCTION

The construction of Margatiga Dam is part of the utilization of Way Sekampung River from upstream to downstream or known as Cascade dam. The Margatiga Dam is located in Trisinar Village and Negeri Jemanten Village, Margatiga District, East Lampung Regency, Lampung Province (Figure 1). The Margatiga Dam is designed as a combination of gravity concrete dam and rock fill with an upright core. Gravity concrete dam is located in riverbed and embankment dams with upright core are located in river floodplains on the right and the left side of the gravity concrete dam (Figure 2).

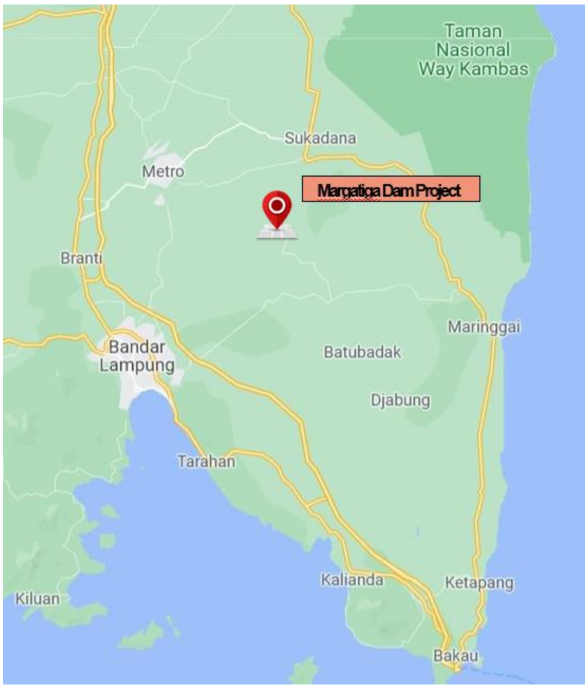

Figure 1 Location of Margatiga Dam Project

Figure 2 BIM 3D Model of Margatiga Dam Project

Based on the initial design by consultant, elevation of concrete dam foundation of spillway, intake and retaining wall will be built at elevation +06.00 and +08.00 (Figure 4). Water level condition at Margatiga is various, normally water level at elevation +09.00 (dry season) – +11.00 (rainy season). After conducted the sub surface test included Standard Penetration Test (SPT) and Pressure Meter Test (PMT), the result explained that at elevation +06.00 and +08.00 or at the initial foundation design, SPT value (NSPT) is 4 - 20 or categorized as loose rock and medium dense (Terzaghi and Peck, 1976). The result is not suitable according to technical specification from the Consultant that recommend SPT value >50 or categorized as very dense for foundation. Due to the geological sub surface test results show that the SPT value is lower than specification. Dam Advisory Service (DAS) Team suggested that soil foundation is replaced by the lean concrete (Figure 5). After further analysis, authors found that the recommendation from DAS team could not been implemented due to:

1. Soil replacement with lean concrete is about 8 – 12 meters thick because the NSPT >50 is at elevation -2.00 and the bottom structure is at elevation +6.00 - +10.00.
2. The water level at the project is very high so when the excavation happens it could inundate a large area and dangerous for workers and equipment.
3. Some areas were already built and the excavation work with 8 meters depth eventually could causes collapse for structure around (Figure 3).
4. The budget calculation of lean concrete method is very expensive because the amount of the soils that needed to replace with concrete is very large and thick, beside that the covering method using soldier pile to prevent the collapse of the building that were already built is also expensive.

Figure 3 Actual condition of Margatiga Dam Project

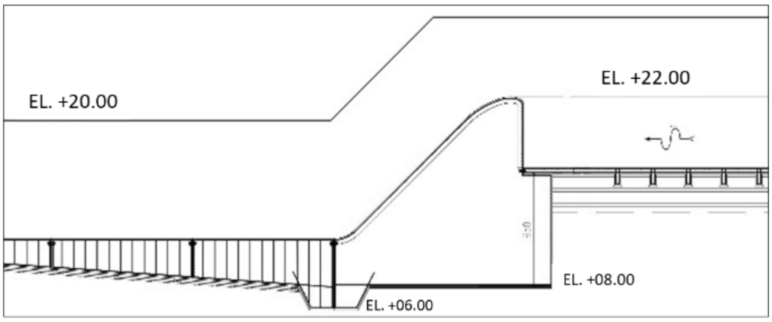

Figure 4 The initial spillway foundation design at elevation +6.00 and +08.00 without considering sub surface condition

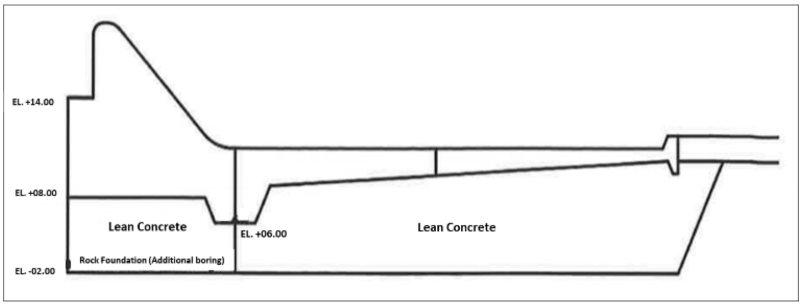

Figure 5 Lean concrete foundation design suggested by the Dam Advisory Service (DAS) Team

The sub-surface condition of the dam structure was composed of thick layer of alluvial sand (Figure 6) and the position of the foundation at SPT < 20 and PMT value < 3 MPa, if the dam building foundation is still built at the elevation of the foundation plan, it can cause the dam building to become unstable due to the rock layer condition that is unable to support the weight of the building. The purpose of this study is to analyze the method of strengthening the foundation to replace the initial design of the foundation due to the problem of bearing capacity of the foundation because of the geological sub-surface condition.

Figure 6 The condition of spillway area that dominantly consist of alluvial sand

METHODOLOGY

To determine the method of the foundation strengthening, authors need to collect the data about the geological condition especially about the geological structure and rocks. Data and information obtained from geological sub surface investigation that provide the information about the types of rock and rock conditions including the level of weathering and rock structure (Hencher, S., 2013). The data from sub surface test later will be calculated to determine the bearing capacity of the foundation.

1. Sub Surface Test

The sub surface test conducted at Margatiga includes Standard Penetration Test (SPT) and Pressure Meter Test (PMT) (Figure 7), the objectives is to provide the data and information about the types of rocks, rock condition, lateral pressure, and soil/rock layer penetration resistance parameter.

Figure 7 PMT & SPT location (view from downstream)

a) Standard Penetration Test (SPT)

Based on Indonesian National Standard (SNI) 4153:2008 regarding Field Penetration Test

Method with SPT, this standard stipulates the method of field penetration test using SPT to obtain soil layer penetration resistance parameters in the field using SPT. These parameters are obtained from the number of blows against cone penetration which can be used to identify soil layers that are part of the foundation design (Badan Standarisasi Nasional, 2008).

Figure 8 Standard Penetration Test in the intake area of Margatiga Dam Project

Figure 9 Core sample from boring activity at Margatiga Dam Project

Beside the SPT value, core sample obtained from this drilling process (Figure 9). This core sample is needed to ascertain condition, type and properties of underground rock, such as its porosity and permeability (Handy and Spanger, 2007).

b) Pressure Meter Test (PMT)

Pressure Meter Test is used to determine the modulus of elasticity of soil/rock and shear modulus of soil/rock. This test is carried out by applying lateral pressure to the soil through a drill hole and then recording the behavior of the soil layer when a lateral load is applied. The advantage of shear modulus and elastic modulus in geotechnical modeling is that it can determine how much deformation occurs when a load is applied (Handy and Spanger, 2007).

The pressure meter test is carried out on a new drill hole with the condition of the hole wall being intact or not collapsing. This test uses a rubber probe as a pressure medium on the borehole wall, for pressure obtained from a water pump that has a pressure manometer. Pressure is applied gradually until the earth wall collapses or there is no more resistance. The recorded value is the diameter of the rubber to the ground and the amount of pressure applied. The parameter value of the bearing capacity of the soil is obtained from the graph formed from the pressure value and the diameter of the rubber expansion.

Figure 10 Pressure Meter Test in the spillway area of Margatiga Dam Project

2. Analysis of the Bearing Capacity Foundation

Structural buildings in compressible soil locations are generally designed to use pile foundations to provide sufficient bearing capacity (Das, 2011). Some conditions that require pile foundations as pile foundation systems, including the following:

- The surface soil layer is a highly compressible layer and has a low bearing capacity
- The upper structure receives considerable horizontal force
- Upper structure receives tensile or uplift

Pile foundations resist axial compression loads and end resistance, tensile loads are resisted through blanket resistance, and lateral loads are resisted by the stiffness of the pile and the surrounding soil (Tomlinson and Woodward, 2014)).

a) The Ultimate Bearing Capacity (Q_u)

The ultimate bearing capacity of the pile foundation (Q_u) is the sum of the skin friction (Q_s) and end bearing capacity (Q_p) as illustrated in Figure 11. The ultimate bearing capacity can be calculated using Meyerhof (1976) equation below.

$$Qu = Qs + Qp \quad (1)$$

Where:
Qu : ultimate bearing capacity
Qs : skin friction
Qp : end bearing capacity

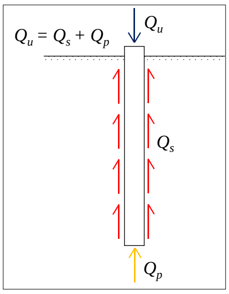

Figure 11 Ultimate bearing capacity of pile foundation

Based on geological sub-surface test, Margatiga Dam Project consist of alluvial sand (coarse grained soil) and tuffaceous sandstone (cohesive soil), it means that the analysis needs to consider the type of soil and rock (Fadilah and Tunafiah, 2018).

1) Cohesive Soil
The end bearing of pile foundations (Q_p) in cohesive soils, such as clays, can be estimated using Resse and Wright (1977) equation below.

$$Qp = Nc \times cu \times Ap \quad (2)$$

where:
Nc : bearing capacity factor = 9
cu : undrained shear strength (6 x N-SPT)
Ap : pile area

2) Coarse Grained Soil
Reese and Wright (1988) stated that the skin friction (Q_s) of piles in coarse-grained layers, such as sand, can be obtained through N-SPT using the following equation.

$$Qs = 2 \times N - SPT \times P \times L \quad (3)$$

Where:
P : perimeter
L : length of pile

The end bearing (Q_p) in the coarse-grained layer can be evaluated using Reese and Wright (1988) equation below.

$$Q_p = 40/3 \cdot N \cdot L/D \quad (4)$$

Where:
N : value of N-SPT
L : length of pile
D : diameter of pile

b) The Axial Allowance Bearing Capacity (Q_{all})

The axial allowance bearing capacity of the pile foundation (Q_{all}) condition can be obtained by dividing the ultimate bearing capacity by the safety factor/number (SF) according to Meyerhof (1976).

$$Qall = \frac{Qu}{SF} \quad (5)$$

The value of the safety factor used depends on various considerations including:

1) Reliability of soil parameters
2) Soil layer uniformity
3) Pile construction method used
4) Quality control and quality assurance during construction

The safety factor used is between 2.0 and 4.0 for service/operation loads. Safety Factor requirements based on Indonesian National Standard (SNI) 8460-2017 for group pile carrying capacity is 2.5 (Badan Standarisasi Nasional, 2017).

c) Bearing Capacity Pile Group Foundation (Q_{ug})

In general, the pile foundation structure is designed as a pile group to be able to provide a bearing capacity that meets the requirements (Hardiyatmo, 2015). In pile foundation group, when the pile foundations are close to each other, the stresses that occur will be spread by each pile foundation into the soil and rocks, thereby reducing the bearing capacity of the single pile foundation (Pham et al, 2018). To calculate the effect of this overlapping stress on the axial bearing capacity of the pile foundation, it is necessary to evaluate the efficiency of the pile foundation in the group (E_g) using Converse Labare (1941) [see Moorhouse and Sheehan (1968)] equation below.

$$Eg = 1 - \left[\frac{(n_1-1)n_2+(n_2-1)n_1}{90 n_1 n_2}\right] tan^{-1}(D/s) \quad (6)$$

where:
n : number of foundations in a row
D : diameter of pile
s : distance between poles

The ultimate group bearing capacity (Q_{ug}) can be estimated using Converse Labare (1941) [see Moorhouse and Sheehan (1968)] equation below.

$$Q_{ug} = Eg \sum_{i=1}^{n} Q_u \quad (7)$$

RESULTS AND DISCUSSION

1. Geological Sub Surface Investigation

The results of geological mapping shown that the Margatiga Dam is composed of alluvial or river sand deposits, vesicular basalt and tuffaceous sandstone. The left main dam and right main dam areas are consisting of basal vesicular and tuffaceous sandstone (Figure 13). The spillway area is composed of alluvial deposits which have the characteristics of loose sand and are very porous (Figure 6).

Based on the core sample obtained from coring activity and further analysis, rock masses are categorized into three according to the weathering degree (Kikuchi et all, 1982) (Figure 12). The upper layer from elevation +12.00 - +05.00 consists of alluvial or river sand deposits categorized as Class D with characteristic of loose material with the joints tensile strength and fractures are almost non-existent. The middle layer from elevation +05.00 - +1.00 categorized as Class CL consists of tuffaceous sandstone with characteristic of weathered rock and soft rock. And the last layer tested from elevation +01.00 - -02.00 categorized as Class CM consists of tuffaceous sandstone with classification of medium rock.

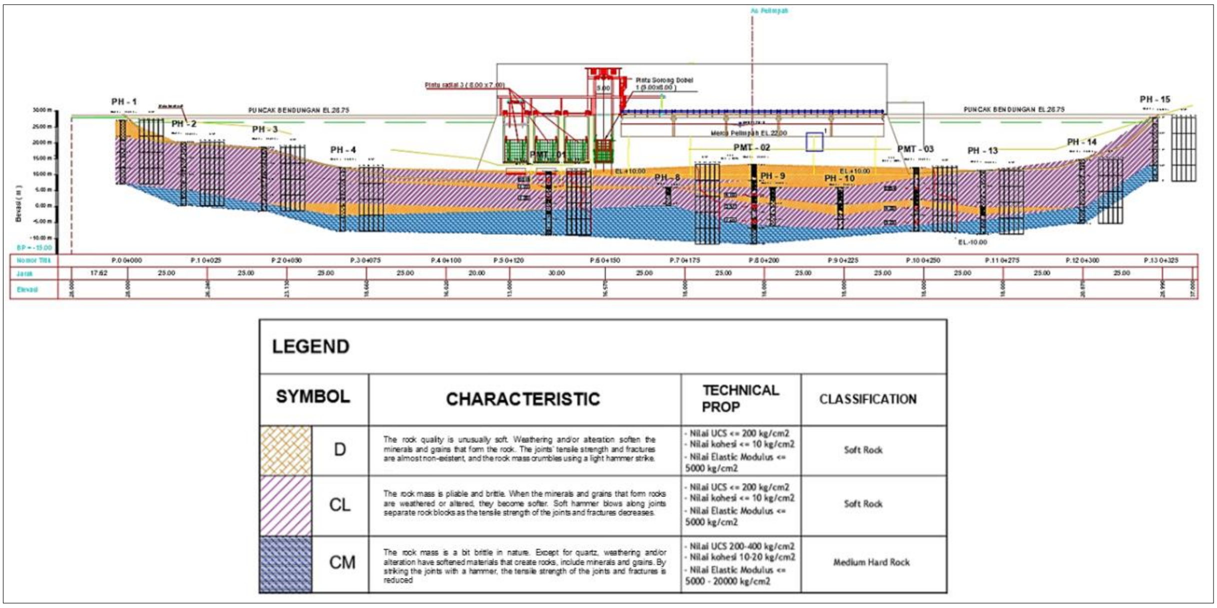

Figure 12 Engineering geology cross section of Margatiga Dam Project

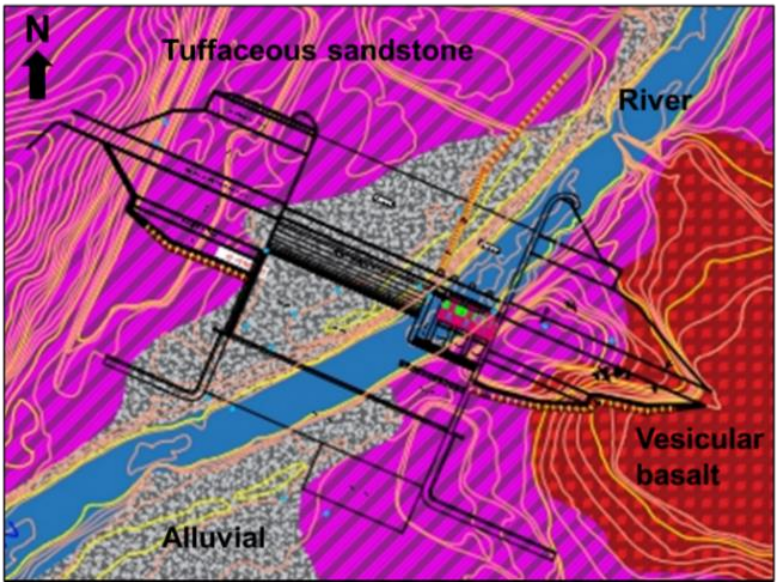

Figure 13 Geological map of Margatiga Dam Project

Based on stratigraphic analysis from Geology Regional map of the Tanjung Karang quadrangle, Sumatera (Mangga et al, 1993), alluvial or river sand deposit is part of Alluvium Unit of Holocene Era or Quarter, vesicular basalt is part of Sukadana Basalt Formation of Pleistocene Era and tuffaceous sandstone is part of Lampung Formation of Pliocene – Pleistocene Era. The geological age could affect the compaction of the rock, the younger the age of the rock (the geological age), the compaction process has not occurred optimally, in this case it happened on alluvial or river sand deposit with geological age of Quarter and has characteristic of loose sand.

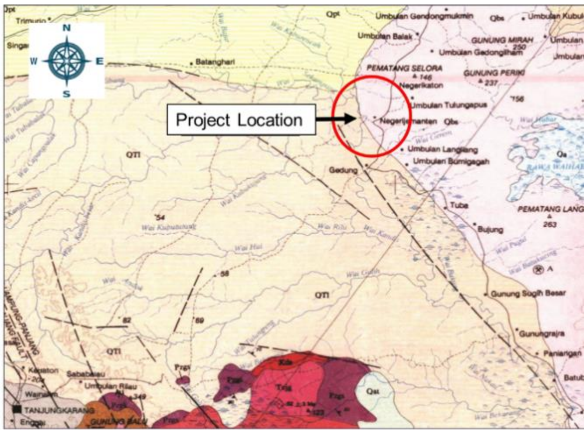

Figure 14 Geology Regional map of the Tanjung Karang quadrangle, Sumatera (Mangga et al, 1993)

Regarding the sub-surface test or investigation, PMT results provide an illustration of how much deformation occurs when the rock layer below the surface is subjected to a load or structure. The PMT results show that the PMT 1 or the location of the intake area has a maximum pressure value of 0.16 – 1.22 MPa, at the PMT 2 or at the spillway area has a maximum pressure value of 2.57 – 7.40 MPa, while at PMT 3 or the location of the retaining wall has a maximum pressure value of 0.63 – 5.2 MPa. According to Bieniawski (1989) classification and its relation to PMT outcomes, layers of constituent rock in the intake, spillway and right-side wall/retaining wall area are categorized as very soft rock to soft rock (Figure 15). Based on the technical specifications, the foundation must be in very dense rock layer, if the dam building foundation is still built at the elevation of the foundation plan, it can cause the building to become unstable due to the rock layer at that elevation not able to support the weight of the building (Terzaghi and Peck, 1967).

Meanwhile, from the results of the SPT, it is known that compact/dense rock with the type of tuffaceous sandstone has SPT value >50 and alluvial or river sand has SPT value <10 or categorized as loose sand. The intake area with SPT >50 is at elevation of -2.159, the spillway area with SPT > 50 is at elevation of +1.23 and the right-side wall/retaining wall area with SPT > 50 is at elevation 0.0. Alluvial with SPT <10 found at elevation +11.00 - +08.00 (Figure 16). The value of the SPT is very important because the foundation stable must be in a rock layer with SPT value >50 according to the Technical Specification. In addition, the results of the SPT test show that the rock in the foundation design plan is NSPT <50, to get a rock layer with an NSPT >50, the structure foundation must be lowered to 5 - 8 meters from the design plan. The initial design by consultant, elevation of concrete dam foundation of spillway, intake and retaining wall will be built at elevation +06.00 and +08.00, regarding the sub-surface test conducted earlier, it showed that SPT and PMT value at spillway, intake and retaining wall were lower than specification. New method or design to strengthen structural foundation is needed so that construction failure does not occur due to sub-surface rock conditions that do not meet the requirements.

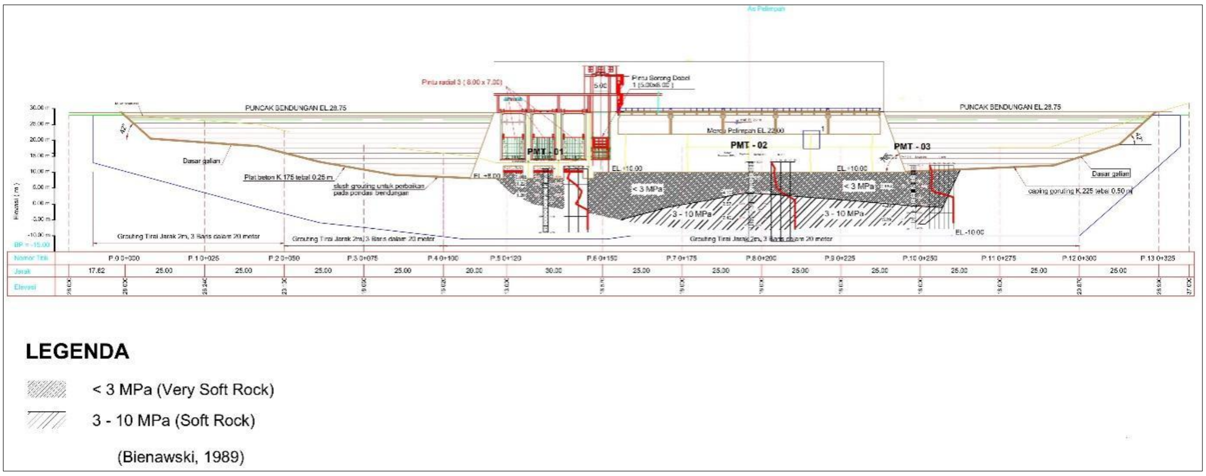

Figure 15 Cross section of Pressure Meter Test (PMT) result at Margatiga Dam Project

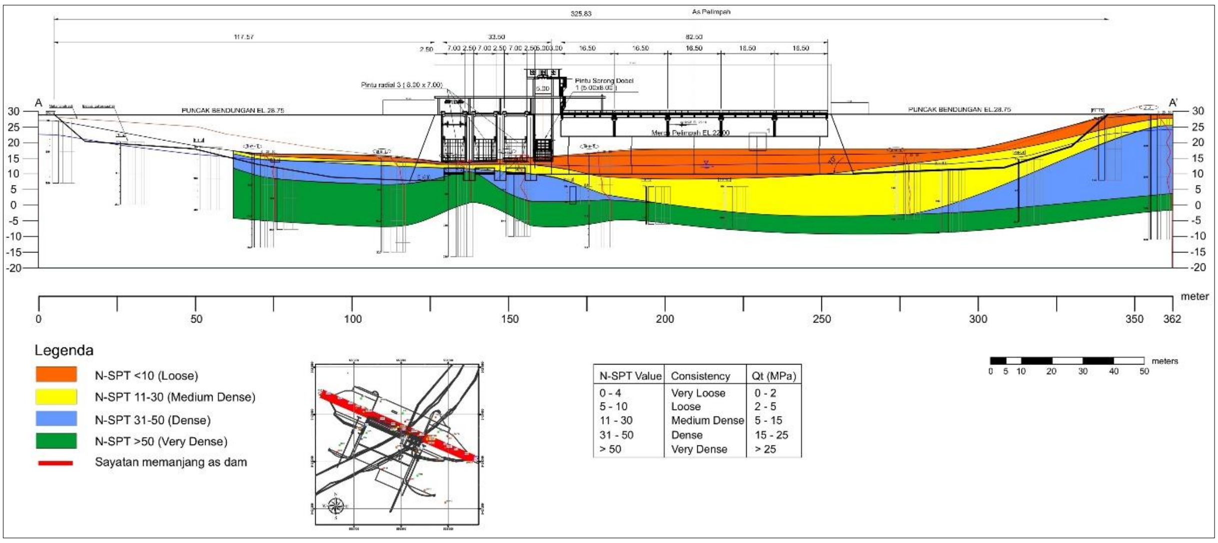

Figure 16 Cross section of Standard Penetration Test (SPT) result at Margatiga Dam Project

2. Bore Pile Foundation

Bore pile foundation in the spillway, intake and retaining wall area using pile with diameter 0.8 m (Figure 19). The selection of this foundation has been carried out by considering the limited space, the existing soil conditions, the structure load, and the maximum reinforcement limit. Bore pile is a type of foundation with reinforced concrete elements inserted into the borehole. This type of foundation is a type of deep foundation with a tubular design. Its function is to transmit the building load to the hard soil layer (Varghese, 2012).

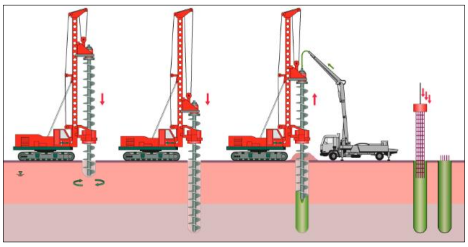

Figure 17 Illustration of work sequence of bore pile foundation *(source: https://www.arsitur.com/2017/10/pengertian-pondasi-bored-pile-dan.html)*

Figure 18 Documentation of boring activity at Margatiga Dam Project

The work sequence of bore pile foundation is start with setting out/survey of pile position, after that start with drilling, because of the type of river deposits that easily slide, casing and bentonite are used in this case to protect the bore hole. Verticality measurement/verticality control is very important in conducted the drilling. After that, installation of reinforcement cages into the bore hole and pouring of pile concrete. After finish all the work sequence, the post-installation testing using Pile Driving Analyzer (PDA) or using Static Loading Test (SLT) should be conducted to ensure the piles are successfully installed (Wahyudiono and Sulik, 2018).

The bore pile configuration recommendation is based on the individual failure type. Group collapses in configurations tend to be safer than individual collapses therefore the type of individual collapse will be decisive (Pham et all., 2018). Calculation of design recommendations is based on the following criteria:

a) Safety Factor requirements based on Indonesian National Standard (SNI) 8460-2017 for group pile carrying capacity is 2.5 (Badan Standarisasi Nasional, 2017).
b) The soil correlation from the UU triaxial test results for undrained shear strength to the SPT value is as follows. Based on the correlation results, a correlation value of 6 N was chosen. This is based on the correlation results and the uncertainty factor that is accommodated from the safety factor (Dharmayasa, 2017).
c) Calculation of the bearing capacity of the foundation in the bore pile refers to the calculation formula of Reese and Wright (1988) and Meyerhof (1976).

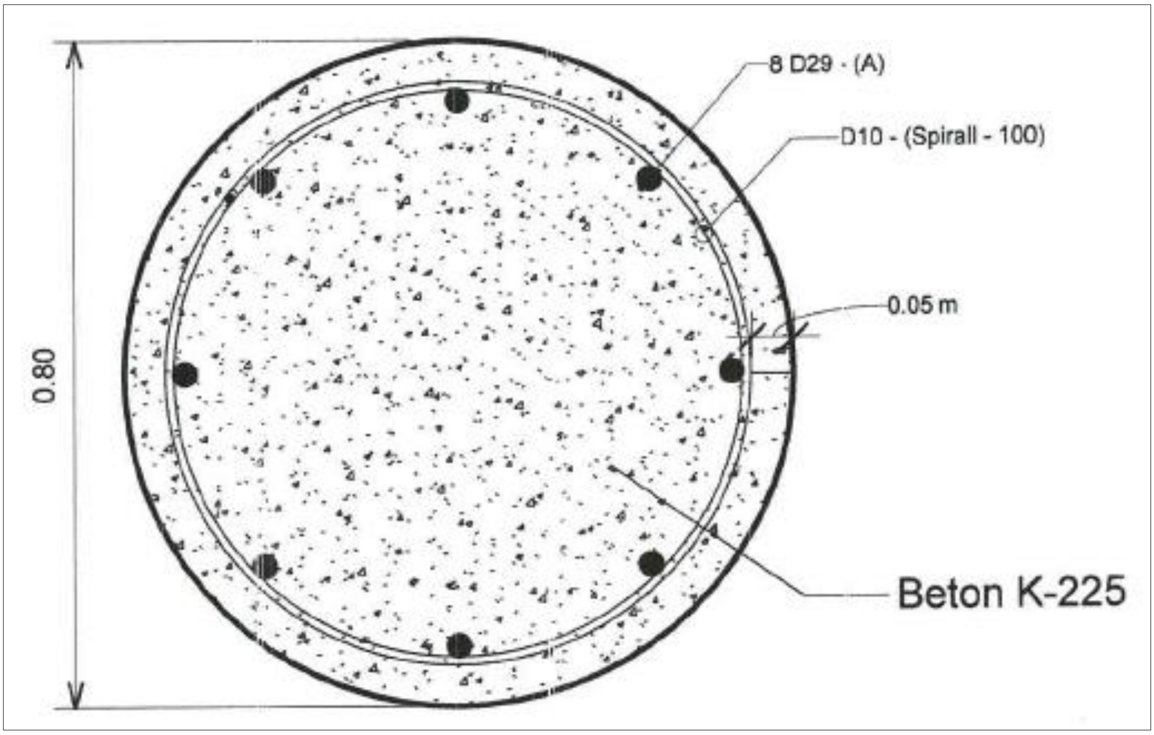

Figure 19 Detail of pile with diameter 0.8 m used at Margatiga Dam Projec

The calculation of bearing capacity in intake, spillway and retaining wall area is as follows:

Table 1 Summary of bore pile foundation design at Margatiga Dam Project

DESCRIPTION	UNIT	LOCATION			
		INTAKE	SPILLWAY 1	SPILLWAY 2	RETAINING WALL
Structural Load	Ton	29517.8	18686.40	18686.40	10249.69
Bore Pile Diameter	m	0.8	0.8	0.8	0.8
Bore Pile Length	m	19	17	17	15
Safety Factor	-	2.5	2.5	2.5	2.5
Ultimate Bearing Capacity of Single Pile (Q_{ult})	kN	5608.87	5593.54	5558.86	4729.48
	ton	**571.75**	**570.19**	**566.65**	**482.11**
Allowable Bearing Capacity of Single Pile (Q_{all})	kN	2243.55	2237.42	2223.54	1891.79
	ton	**228.70**	**228.08**	**226.66**	**192.84**

The results of the recommendations for the analysis of bore pile calculations are as follows:

Table 2 Summary of recommendations for the calculation of bore pile foundation design at Margatiga Dam Project with 3 meters distance configuration

Structure	P Ultimate	L Pile	Qall Pile	*Row*	*Column*	Total piles	Space	Group Efficiency	Qall Group (kN)	Checking
	[ton]	(m)	[ton]				(m)			
Intake	29518	19	229	12	13	**156**	3.00	0.86	30578	OK!
Spillway 1	18686	17	228	14	7	**98**	3.00	0.86	19238	OK!
Spillway 2	18686	17	227	14	7	**98**	3.00	0.86	19118	OK!
Retaining Wall	10250	15	193	8	8	**64**	3.00	0.87	10711	OK!

Based on the Table 2, in intake area with structure load of 29517.8 ton, pile diameter of 0.8 m and safety factor of 2.5, the calculation of ultimate bearing capacity of single pile (Q_{ult}) is 571.75 ton and allowable bearing capacity of single pile (Q_{all}) is 228.70 ton. The total piles needed on this area is 156 piles consist of 12 row and 13 columns with 19 m length and space of each pile is 3 m.

In spillway 1 area with structure load of 18686.40 ton, pile diameter of 0.8 m and safety factor of 2.5, the calculation of ultimate bearing capacity of single pile (Q_{ult}) is 570,19 ton and allowable bearing capacity of single pile (Q_{all}) is 228.08 ton. The total piles needed on this area is 98 piles consist of 14 row and 7 columns with 17 m length and space of each pile is 3 m. In Spillway 2 area with structure load of 18686.40 ton, pile diameter of 0,8 m and safety factor of 2.5, the calculation of ultimate bearing capacity of single pile (Q_{ult}) is 566.65 ton and allowable bearing capacity of single pile (Q_{all}) is 226.66 ton. The total piles needed on this area is 98 piles consist of 14 row and 7 columns with 17 m length and space of each pile is 3 m.

In Retaining wall area with structure load of 10249.69 ton, pile diameter of 0.8 m and safety factor of 2.5, the calculation of ultimate bearing capacity of single pile (Q_{ult}) is 482.11 ton and allowable bearing capacity of single pile (Q_{all}) is 192.84 ton. The total piles needed on this area is 64 piles consist of 8 row and 8 columns with 15 m length and space of each pile is 3 m.

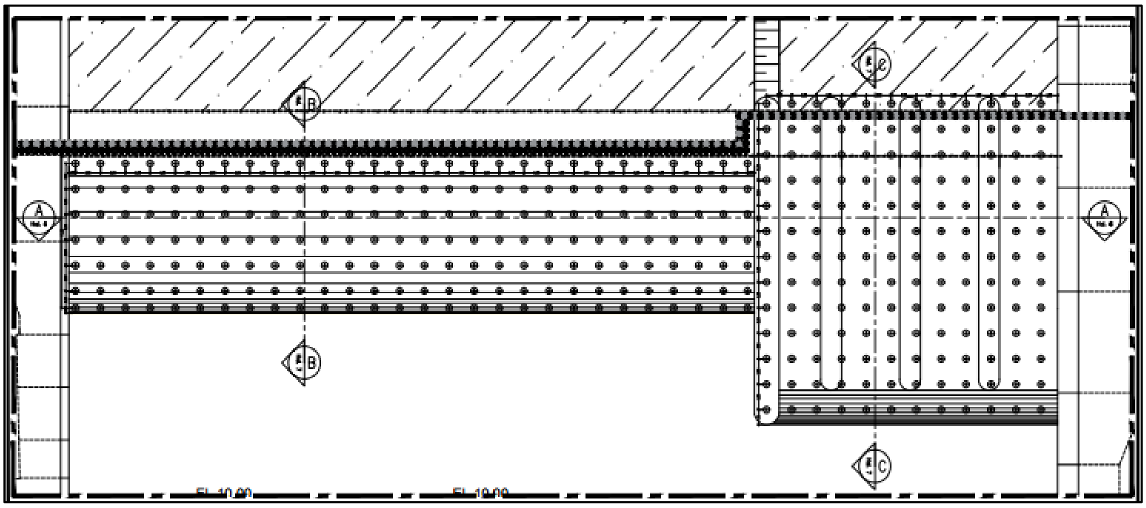

Figure 20 Bore pile foundation plan in spillway area of Margatiga Dam Project

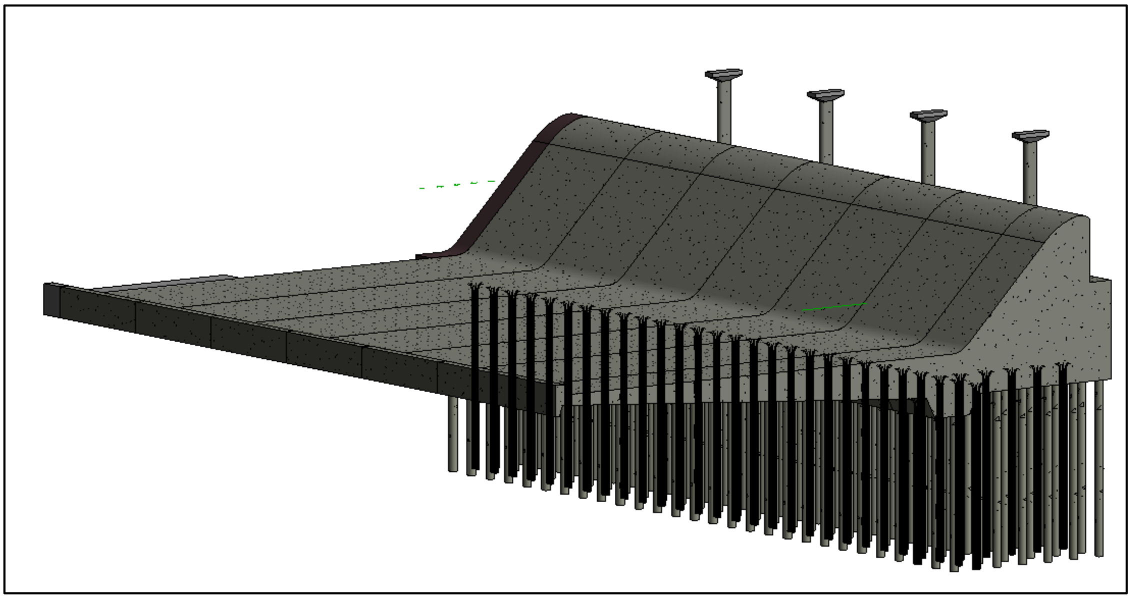

Figure 21 3d view of bore pile foundation at spillway area

CONCLUSIONS

Based on the analysis of the foundation of the Margatiga Dam Project, the conclusions of this study are as follows:

The results of geological mapping shown that the Margatiga Dam is composed of alluvial or river sand deposits, vesicular basalt and tuffaceous sandstone. The left maindam and right maindam areas are consists of basal vesicular and tuffaceous sandstone. The spillway area is composed of alluvial deposits which have the characteristics of loose sand and are very porous. Based on the core sample obtained from coring activity and further analysis, rock masses are categorized into three according to the weathering degree (CM, CL and D). The data from sub surface test including Standard Penetration Test (SPT) and Pressure Meter Test (PMT) used for the calculation of bearing capacity on area at Margatiga Dam Project. The data used to include n value SPT, cohesion, type of soil, friction, PMT modulus and shear modulus. The PMT results shown that the PMT 1 or the location of the intake area has a maximum pressure value of 0.16 – 1.22 MPa, at the PMT 2 or at the spillway area has a maximum pressure value of 2.57 – 7.40 MPa, while at PMT 3 or the location of the right-side wall/retaining wall has a maximum pressure value of 0.63 – 5.2 MPa. According to Bienawski's (1989) classification and its relation to PMT outcomes, layers of constituent rock in the intake, spillway and right-side wall/retaining wall area are categorized as very soft rock to soft rock. The SPT result shown that compact/dense rock at tuffaceous sandstone layer with SPT value >50 at the intake area is at elevation of -2.159, the spillway area is at an elevation of +1.23 and a right-side wall/retaining wall area is at elevation 0.0. The value of the SPT is very important because the foundation stable must be in a rock layer with SPT value> 50 according to the Technical Specification. The initial design by consultant, elevation of concrete dam foundation of spillway, intake and retaining wall will build at elevation +06.00 and +08.00, regarding the sub-surface test (SPT and PMT) conducted earlier, it showed that sub surface condition at spillway, intake and retaining wall causes the need for strengthening the structure foundation so that construction failure does not occur due to sub-surface rock conditions that do not meet the requirements. Strengthening the foundation with the bore pile method is necessary because the foundation of the concrete building in the plan design is on alluvial sand with characteristic of loose sand (NSPT < 20). The results of bore pile foundation calculations with safety factor 2.5, the total piles needed on intake area is 156 piles consisting of 12 row and 13 columns with 19 m in length and space of each pile is 3 m. The total piles needed on Spillway 1 and 2 area is 196 piles consisting of 28 row and 14 columns with 17 m in length and space of each pile is 3 m. The total piles needed on this area is 64 piles consisting of 8 row and 8 columns with 15 m in length and space of each pile is 3 m.

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