

FINAL PROJECT

**BEARING CAPACITY ANALYSIS OF BORED PILE
FOUNDATION AND SETTLEMENT ON CLAY SOIL**

**Submitted to Universitas Islam Indonesia Yogyakarta to Fulfill the
Requirements for Obtaining the Degree of Bachelor of Civil Engineering**



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PREFACE

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NOTATION LIST

SF	= Safety Factor
Q_b	= End Bearing Capacity
N_b	= The statistical mean of the SPT numbers within an area approximately 8D above to 4D below the pile point
A_b	= Cross-sectional area of bored pile
Q_s	= Skin Friction
X_m	= 0,2 for bore pile
L_i	= length of soil layer (m)
p	= pile circumference (m)
N-SPT	= Number of statistical average stroke calculations
Q_u	= Ultimate bearing capacity of the pile
Q_{all}	= Allowable support capacity
A_{st}	= cross-sectional area of pile tip
L_i	= Length of soil layer
f	= Skin friction unit resistance
C_u	= Soil Cohesion
n	= Number of Piles
ΣP_{max}	= Maximum load on foundation pile plan
M_x	= X-direction moment
M_y	= Y-direction moment
$S_{e(1)}$	= Elastic shortening of the pile
$S_{e(2)}$	= Settlement of the pile due to the working load at the pile point
$S_{e(3)}$	= Settlement of the pile due to the working load along the pile shaft

Q_{wp}	= working load at the pile point
Q_{ws}	= working load along the pile shaft
A_p	= cross-sectional area of the pile point
L	= pile length
E_p	= modulus of elasticity of the pile material
ξ	= a constant in the range of 0.50–0.67, depending on the skin friction distribution
D	= width or diameter of pile
Q_{wp}	= working load at the pile point
q_p	= ultimate point resistance of the pile
C_p	= an empirical coefficient
L	= embedded length of pile
I_{ws}	= influence factor
E_s	= Modulus Elasticity
B	= Width
Z_0	= Depth
L	= Height
B_x	= width
B_y	= length
s	= distance pile to pile
a	= distance pile to edge
m	= number of pile x direction
n	= number of pile y direction
$\Delta S_{c(i)}$	= consolidation settlement of layer i
$\Delta e_{(i)}$	= change of void ratio caused by the increase in stress in layer i
$e_{(o)}$	= initial void ratio of layer i (before construction)
H_i	= thickness of layer i
F'_c	= concrete quality

TD	= Traffic Load
B1	= Width of highway
L	= Length of highway in 1 span
Q	= Evenly distributed load
p	= Pointed line load
DLA	= Dynamic load Allowance
W_{TD}	= traffic load
P_{TD}	= Traffic load staging reaction
TP	= Pedestrian Load
P_{TP}	= Load on abutment due to pedestrian
TB	= Brake Load
T_{TB}	= Brake load
Y_{TB}	= Arm's Length Work Foundation
EW	= Structure Wind Load
C_w	= Drag coefficient

ABSTRACT

The foundation is one of the most important elements in a building. The foundation serves to forward the load of the structure above it to the soil layer below. In the construction of the Tawang-Ngalang Bridge segment IV built on clay soil, a type of bored pile foundation with a diameter of 1 meter with a depth of 12 meters will be used. Researchers want to re-analyze the bored pile foundation in the field with dimension variations of 0,85 meters, 0,9 meters and 0,95 meters using several research methods. This aims to determine the bearing capacity and settlement of bored piles based on SNI 1725: 2016 Bridge Loading.

For this study, SNI 1725:2016 about bridge loading was used to determine loading on the bridge. While for calculation analysis, methods by Meyerhof (1956) and Reese and Wright (1977) would be used for determining the bearing capacity of single and group piles. Lastly, the settlement value would be determined using equation by Vesic (1977) and also verified by Finite Element Method of Plaxis 3D Software

The results of the structural loading analysis based on SNI 1725:2016 Bridge Loading obtained the value of $P = 35126,994 \text{ kN}$, $T_x = 11387,697 \text{ kN}$, $T_y = 2878,640 \text{ kN}$, $M_x = 31669,403 \text{ kN/m}$, $M_y = 16272,106 \text{ kN/m}$. Bearing capacity of single bored pile for existing dimension of Meyerhof (1956) method is 488,88 T and Reese and Wright (1977) method is 327,207 T. Elastic settlement on single pile for Meyerhoff (1956) method equation is 6,1 mm and Reese and Wright (1977) method is 4,02 mm. for plaxis 3D v20 results from Meyerhof (1956) method is 11,03 mm and from Reese and Wright (1977) method is 11,27 mm. for the capacity of a single pile at various dimensions (0,85 m, 0,9 m, and 0,95 m), the meyerhof method (1956) amounted to 379,136 T, 418,403 T, and 447,182 T and the Reese and Wright method (1977) amounted to 269,268 T, 290,350 T, and 306,198 T, respectively. Single pile settlement using Vesic (1977) method for Meyerhof (1956) method is 4,1 mm, 4,7 mm, and 5,3 mm and from Reese and Wright method is 3 mm, 3,3 mm, and 3,6 mm. for the results from plaxis 3D v20 for Meyerhof Equation is 5.54 mm, 13.63 mm, and 15.17 mm and for Reese and Wright equation is 10.5 mm, 10.46 mm, and 10.27 mm.

Keywords: Bearing capacity, Plaxis 3D, Meyerhof (1956) Method, Reese and Wright (1977) Method

CHAPTER I

PRELIMINARY

1.1 Background

A bridge is a construction that is useful for continuing the path through a lower obstacle. These obstacles are usually other roads (waterways or normal traffic roads). Bridges, which are part of the road, are indispensable in the land transportation network system that will support development in the area. Such as the bridge in Tawang-Ngalang Segment IV road and bridge construction project which will be used as the study case in this research. The purpose of this bridge is to complete the tourist route in Gunung Kidul Regency. As has been known, this tourist route, if it is completed, will have a very large impact on the regional economy it passes through in this case D.I. Yogyakarta, facilitate the flow of tourism during holidays, greater potential for regional development, and advancing tourism activities.

On a bridge construction, a foundation is the most important aspect of a construction project because it distributes the loads acting on the upper structure to the subgrade layer as its support. Generally, foundation is divided into two types, shallow and deep foundation. For this case, pile foundation, categorized as deep foundation, was used due to the load carried and the importance of the structure.

The selection of the type of pile foundation is based on the function of the superstructure (upper structure) to be carried by the foundation, the magnitude of the load and the weight of the superstructure, the condition of the soil where the building will be erected, and the cost of the foundation compared to the superstructure (Sardjono, 1998).

The foundation used in this project is the bored pile foundation. A bored pile foundation is a deep spherical foundation pile with the dimension 1 m in diameter and 12 m in depth. This type is also known as drill pile foundation which means it

is a type of deep tube foundation that functions to continue the load of the building structure on it from the ground surface to the hard soil layer below it. Drill pile foundations have the same function as pile foundations or other deep foundations. The interaction between the drill pile foundation and the soil is expected in the superstructure. Two methods can be used to determine the bearing capacity of the bored pile foundation, namely the static method and the dynamic method.

As for the static method, the method created by Meyerhof in 1956 and Reese and Wright in 1977 will be used for the calculation of single and group bearing capacities of the pile. While for the Finite Element Method, Plaxis 3D application will be used to obtain the settlement and consolidation value for each variation of group piles. At last, those calculation will be done in order to obtain the most effective and efficient diameter of bored pile foundation on the project of Tawang-Ngalang Segment IV Road and Bridge Construction Project.

1.2 Problem Formulation

The problem formulation is as follows.

1. How much is the value of the bearing capacity on a single pile with a diameter of 1 m and a pile length of 12 m using existing data with the method of Meyerhof (1956) and Reese and Wright (1977)?
2. How much is the value of the bearing capacity on a single pile using diameter variations, which is 0,85 m, 0,9 m, and 0,95 m in diameter with a length of 12 m on each pile using existing data with the method of Meyerhof (1956) and Reese and Wright (1977)?
3. How much is the value of bearing capacity on a group pile with a diameter of 100 cm and a pile length of 12 m using existing data with the method of Meyerhof (1956) and Reese and Wright (1977)?
4. How much is the value of the bearing capacity on a group pile using diameter variations, which is 0,85 m, 0,9 m, and 0,95 m in diameter with a length of 12 m on each pile using existing data with the method of Meyerhof (1956) and Wright (1977)?

5. How much is the settlement on existing condition and variations of group foundation with the foundation length of 12 m on Meyerhof (1956) model variation and Reese and Wright (1977) model variation?
6. How much is the settlement on existing condition and variations of group foundation with the foundation length of 12 m using Plaxis 3D on Meyerhof (1956) model variation and Reese and Wright (1977) model variation?

1.3 Purpose

The purpose of this plan is as follows.

1. Obtaining the value of the bearing capacity on a single pile with a diameter of 100 cm and a pile length of 12 m using existing data with the method of Meyerhof (1956) and Wright (1977).
2. Obtaining the value of the bearing capacity on a single pile using diameter variations, which is 0,85 m, 0,9 m, and 0,95 m in diameter with a length of 12 m on each pile using existing data with the method of Meyerhof (1956) and Wright (1977).
3. Obtaining the value of bearing capacity on a group pile with a diameter of 100 cm and a pile length of 12 m using existing data with the method of Meyerhof (1956) and Wright (1977).
4. Obtaining the value of the bearing capacity on a group pile using diameter variations, which is 0,85 m, 0,9 m, and 0,95 m in diameter with a length of 12 m on each pile using existing data with the method of Meyerhof (1956) and Wright (1977).
5. Obtaining the value of the settlement on existing condition and variations of group foundation with the foundation length of 12 m on Meyerhof (1956) model variation and Reese and Wright (1977) model variation.
6. Obtaining the value of the settlement on existing condition and variations of group foundation with the foundation length of 12 m using Plaxis 3D on Meyerhof (1956) model variation and Reese and Wright (1977) model variation.

1.4 Advantage

The advantages of this final project are as follows.

1. It can be used as a reference material for readers to calculate a foundation construction on clay or soils that have a low carrying capacity, and
2. Increase knowledge for readers about the carrying capacity of the bore pile foundation.

1.5 Limit

For the writing of the final project proposal to be structured, the following observation limits are made.

1. The research location is the Tawang-Ngalang Segment IV Road and Bridge Construction,
2. The lower structure used is bored pile foundation,
3. Bearing capacity analysis of the foundation used Meyerhof 1956 & Reese and Wright 1977 method as the static methods and Plaxis 3D v20 applications as the Finite Element Method,
4. Soil investigations was carried out by PT. Geomine Bara Studio as the consultant of the project,
5. The distance between piles is set to be 3D,
6. The N-SPT Data obtained from borehole (BH 01) used as the based data for calculation,
7. Pile group of all calculation is located on Abutment 1,
8. Analysis using Plaxis 3D is only a Settlement analysis, and
9. The safety factor used is 2,5.

CHAPTER II

LITERATURE REVIEW

2.1 General Review

The foundation is the lowest part of the building that transmits the load of the building to the soil or rock beneath it. There are two classifications of foundations, namely shallow foundations, and deep foundations. Shallow foundations are defined as foundations that support the load directly, such as footing foundations, longitudinal foundations, and raft foundations. Foundations can be defined as foundations that transmit building loads to hard soil or rocks that are located relatively far from the surface, for example, well foundations and pile foundations (Hardiyatmo, 2011).

2.2 Bored Pile Foundation

The selection of the type of foundation depends on the load that must be supported, the condition of the foundation soil, and the cost of making the foundation compared to the cost of the upper structure (Hardiyatmo, 2011). Installation of the bored pile foundation into the ground is done by drilling the ground first, which is then filled with reinforcement that has been assembled and cast concrete. If the soil contains water, then an iron pipe or what is commonly called a temporary casing is needed to hold the wall of the hole so that there is no landslide, and this pipe will be removed at the time of placing the concrete.

2.2.1 Analysis of Bearing Capacity of Bored Pile Foundation Using Meyerhof Method

Fadilah & Tunafiah (2018) researched to determine the bearing capacity of the foundation and pile settlement that occurs. The Foundation used in the construction project of 6 toll road sections within the city of Jakarta is the bore pile foundation, and the dimensions used a diameter of 1.2 meters. Data collection uses

secondary data, namely N-SPT data and PDA Test results in the field. The calculation of the carrying capacity is carried out by the conventional method using the Reese & Wright method from the results of the N-SPT Calculation of the pile foundation groups based on the value of efficiency using the Converse-Labarre method of 0.618. Single pile foundation settlement using a semi-empirical method and group settlement piles using the Vesic method. From the calculation results obtained that the ultimate bearing capacity according to the conventional method is smaller than the carrying capacity of the PDA test results.

Ramadhani (2019) conducted a study that aims to find out bearing capacity, dimensions, number of piles, and settlement of drill pile foundations using static methods. Structural analysis calculations are used by the auxiliary program ETABS 2016 software V.16.2.0 to obtain and retrieve style results working on building structures for use in design reinforcement. This study uses dimensions of 60 cm. Using variations the number of piles with the same diameter, namely type 1 as many as 5 piles, type 2 as many as 3 piles, and type 3 as many as 2 piles. The analyzed carrying capacity is single pile ultimate bearing capacity (Q_a) and pile group bearing capacity (Q_{pg}). Based on the calculation results obtained foundation types 1,2, and 3 Q_a value of 241,301 tons. For different Q_{pg} values, the foundation type 1 Q_{pg} is 965,2 tons, foundation 2 of 784,71 tons, and foundation type 3 of 452,44 tons. Foundation types 1,2, and 3 use 12 D19 reinforcement with pile foundation settlement of 2,3 cm in other words a decrease within the safe limit due to the value smaller than $S_{allowable}$ which is 6 cm. A drill pile foundation can be used because P_{max} resulting from the eccentricity force is smaller than the allowable capacity of the pile.

2.2.2 Analysis of Bearing Capacity of Bored Pile Foundation Using Reese & Wright Method

Tamimi (2020) conducted a study that aims to determine effective, safe, and economical drill pile foundations with a variety of dimensions. This study uses a dimension variation of 0.6 m, 0.7 m, and 0.8 m with a length of the mast of 14 m. Calculation of structural analysis used methods, static, theoretical methods, and loading using a support application, namely ETABS. Analysis result bearing

capacity of the Reese and Wright method of pile group respectively obtained at 2731,4232 kN, 3295,4614 kN, and 3890,5856 kN, the method Skempton obtained respectively 2196.6498 kN, 2644.3589 kN, and 3115.3824 kN. From the results obtained, overall it is safe against the loading total to be received.

2.2.3 Settlement Analysis of Bored Pile Foundations Using the Static Method

Idris (2019) conducted a study that aims to determine the burden accepted structure, bearing capacity, dimensions, and foundation settlement bore pile. Calculation of the structural analysis used the reference of SNI 1726 - 2012 and loading using the STAAD Pro V8i app. Based on the analysis and calculation of the results of the loading of the superstructure, obtained the largest load on column (B5-C5) with a magnitude of $P_u = 13944.2$ kN, $M_{uy} = 1394.973$ kN, and $M_{ux} = 278.139$ kN. Based on the calculation of the bearing capacity, the depth of the foundation is planned to a depth of 20 meters with two types of diameters, namely 90 cm and 80 cm. The calculation result of the drill pile with 90 cm, bearing capacity obtained by single pillar 359.75 tons and per group with an efficiency of 71.7394% obtained a carrying capacity of 1548.49 tons. On drill post 7 with 80 cm, the bearing capacity obtained is 313.50 tons of single pile and per group with an efficiency of 75.7766% obtained a carrying capacity of 950.23 tons. The biggest decrease is in column (B5-C5) of 1.382 cm.

2.2.4 Settlement Analysis of Bored Pile Foundations Using the Finite Element Method

Dhiya'ul Haq (2018) conducted a study that aims to find out the bearing capacity and settlement of drill pile foundations with variations in diameter. Calculation of structural analysis used methods, static, theoretical methods, and loading using a support application, namely PLAXIS v8.s. The analysis results in the cotton pile group bearing the Reese & Wright method and the Meyerhof method based on laboratory and field data with diameters 70, 80, and 90, show a value greater than the P value, which is 2,523,564 kN. Until the foundation bored pile 6 with this diameter, it is safe to use in the construction of Twin buildings UMY Building. The analysis of the settlement of the pile group foundation with the diameter obtained by the theoretical method and finite element method (PLAXIS

2D) shows the value of a decrease that is still safe from a significant decline allowed.

2.3 Comparison with Previous Research

The difference between previous research and the research that the author did can be seen in Table 2.1 below.

Table 2.1 Comparison of Past and Present Research

RESEARCHER	Fadhila & Tunafiah (2018)	Dhiya'ul Haq (2018)	Idris (2019)	Ramadhani (2019)	Tamimi (2020)	Athazalni (2024)
TITLE	<i>Analisa Daya Dukung Pondasi Bored Pile Berdasarkan Data N-Spt Menurut Rumus Reese&Wright Dan Penurunan</i>	<i>Analisis Kapasitas Dukung pondasi Tiang Bor Kelompok Dengan Variasi Dimensi</i>	<i>Perencanaan Pondasi Tiang Bor Pada Rusunami Sentraland Bekasi Jawa Barat</i>	<i>Studi Perencanaan Pondasi Tiang Bor (Bored Pile) Pada Gedung Trans Mart Malang.</i>	<i>Perencanaan Ulang Struktur Bawah Gedung Dengan Pondasi Bored Pile</i>	<i>Analysis Bearing Capacity of Bored Pile Foundation and Settlement on Clay Soil</i>
PURPOSE	To determine the bearing capacity of the foundation and pile settlement that occurs. The foundation used in the construction project of 6 toll road sections within the city of Jakarta is the bore pile foundation, and the dimensions used a diameter of 1.2 meters.	Knowing the Support Capacity and Foundation Lowering with Diameter Variation.	Knowing Power Support, Downgrade, and Repetition Bored Pile Foundation and Pilecap.	Knowing Power Support, Quantity reinforcement, and Foundation Drop Drill Pile and Pilecap.	Knowing the Bearing Capacity of Bored Pile Foundations with Variations in Dimensions.	Knowing the Bearing Capacity of Bored Pile Foundations with Variations in Dimensions and various quantities
METHOD	Reese & Wright Method, Converse-Labarre Method, Vesic Method.	Static (SPT & Lab) Method, Element Method, Numeric (SAP 2000 & PLAXIS v8.s) Method.	Static (SPT) Method, Meyerhof Method, Numeric (STAAD Pro V8i) Method.	Static (SPT) Method, Meyerhof method, Semi-Empirical Method	Skempton, Reese & Wright Method, Numerical Method (ETABS).	Meyerhof method, Reese and Wright method, Numeric (Plaxis 3D) method
RESULT	The calculation results obtained that the ultimate bearing capacity according to the conventional method is smaller than the carrying capacity of the PDA test results.	The magnitude of the bearing capacity is influenced by the dimensions. The larger the diameter, the greater the carrying capacity. For the decrease, the larger the diameter, the smaller the decrease.	Based on Power Calculation Support Use Foundation with Depth 20 and 2 diameters is 0.8 m and 0.9 m.	A bored Pile Foundation Can be used Because the Pmax Produced from the Eccentricity force is Smaller Than the Capacity Pile Permit.	The overall drill pile foundation is safe against the total load to be received	The calculation is to find a safe foundation diameter to build a bored pile foundation other than the existing size and calculate the result of the settlement.

(Source: Fadhila & Tunafiah (2018), Dhiya'ul Haq (2018), Ramadhani (2019), Idris (2019), Tamimi (2020), Athazalni (2024))

CHAPTER III

BASIC THEORY

3.1 Soil

Soil is a natural object found on the surface of the earth's skin, which is composed of mineral materials as a result of weathering rocks, and organic materials as a result of weathering of plant and animal remain, which is a medium or place of growth of plants with certain properties, which occurs as a result of the influence of a combination of climatic factors, parent material, living remains, the shape of the territory and the length of time of formation (Yuliprianto, 2010). Soil constituent materials are composed of four components: mineral solids, organic solids, water, and air. Mineral solids consist of primary rock and mineral lips, rock and mineral weathering, and secondary minerals. Organic solid matter consists of the remains and remains of corpses, especially plants, humic substances, and the living bodies of soil inhabitants, including the roots of living plants (Darusman, 2006). Water contains various solutes, so it is also called a soil solution.

Soil is extremely important in construction work. Soil is the foundation for supporting a building or the building's construction material, such as embankments or dams, and it can also be a source of external forces on the building. Because all buildings are constructed above or below ground, we require a foundation system to transfer the load from the building to the ground (Bowless, 1997).

3.1.1 Soil Classification

Soil categorization is the process of grouping or sub-grouping several soil types that exhibit comparable index qualities. Size, shape, and component materials are examples of these qualities. Soil classification is supposed to make soil use and utilization easier. Understanding soil behavior will help in the early stages of building design.

There are various types of soil categorization systems utilized in the building industry. The Universal System Classification Soils (USCS) classification system is a soil classification method that is frequently used in building design. Soil is classified by the USCS into two types: coarse-grained soil and fine-grained soil. Coarse-grained soils are gravel and sand soils that pass the No.200 sieve with 50% of their entire weight. Fine-grained soil is defined as dirt that passes sieve No.200 with more than 50% of its entire weight. Under this method, each soil is assigned a symbol consisting of two letters indicating its type and nature.

The first letter indicates the type, such as:

G = Gravel

S = Sand

M = Silt

C = Clay

O = organic sand

The second letter indicates the type, such as:

W = Well Graded

P = Poorly Graded

M = Contains Silt

C = Contains Clay

L = Low Plasticity

H = High Plasticity

The soil classification system based on the USCS can be seen in Table 3.1 below.

Table 3.1 USCS Soil Classification System

Major division			Group symbol	Typical name	Classification criteria			
Coarse-grained soils (More than 50% retained on No. 200 ASTM sieve)	Gravels 50% or more of coarse fraction retained on No. 4 ASTM sieve	Clean gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines.	Classification on the basis of percentage of fines. Less than 5% passing No. 200 ASTM sieve—GW, GP, SW, SP. More than 12% passing No. 200 ASTM sieve—GM, GC, SM, SC. 5% to 12% passing No. 200 ASTM sieve—Border-line classification requiring use of dual symbols.	$U = D_{60}/D_{10}$ greater than 4 $C_c = \frac{D_{60}^2}{D_{30} \times D_{20}}$ between 1 and 3.		
			GP	Poorly-graded gravels and gravel-sand mixtures, little or no fines.		Not meeting both criteria for GW.		
		Gravels with fines	GM	Silty gravels, gravel-sand-silt mixtures.		Atterberg limits plot below A-line or plasticity index less than 4.		
			GC	Clayey gravels, gravel-sand-clay mixtures.		Atterberg limits plot above A-line or plasticity index less than 4.		
	Sands More than 50% of coarse fraction passes No. 4 ASTM sieve	Clean sands	SW	Well-graded sands and gravelly sands, little or no fines.		U greater than 6 C_c between 1 and 3.		
			SP	Poorly-graded sands and gravelly sands, little or no fines.		Not meeting both criteria for SW.		
		Sands with fines	SM	Silty sands, and-silt mixtures.		Atterberg limits plot below A-line or plasticity index less than 4.		
			SC	Clayey sands, sand-clay mixtures.		Atterberg limits plot above A-line or plasticity index greater than 7.		
			Fine-grained soils (50% or more passes No. 200 ASTM Sieve)	Silt and Clays (Liquid limit 50% or less)		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.	Check Plasticity Chart
						CL	Inorganic clays or low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	
OL	Organic silts and organic silty clays of low plasticity.							
Silt and clays (Liquid limit greater than 50%)	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts.						
	CH	Inorganic clays of high plasticity, fat clays.						
	OH	Organic clays of medium to high plasticity.						
	Highly organic clays			P _t	Peat, muck and other highly organic soils.	Fibrous organic matter, will char, burn, or glow. Readily identified by colour, odour, spongy feel, and fibrous texture.		

Note: Boundary classification: Soils possessing characteristics of two groups are designated by combinations of group symbols — for example, GW-GC, well-graded, gravel-sand mixture with clay binder

3.2 Soil Investigation

Soil investigation is a technique for determining the properties and bearing capacity of the soil that will serve as a building's foundation. Soil testing is required before deciding on the type of foundation to use. The foundation type that will be used. The following are the goals of soil investigation.

1. Understanding the depth of hard soil at the site so that proper foundation placement may be determined,
2. determining the depth of the groundwater level,
3. understanding the soil's properties in each layer,
4. estimate the soil's carrying capability,
5. identify the soil sample that will be evaluated in a laboratory, and
6. decide the type of foundation to be employed.

Soil investigation typically consists of three stages: drilling, soil sampling, and laboratory testing of soil samples. Sondir or Cone Penetration Test (CPT) and

Standard Penetration Test (SPT) are two soil investigations conducted in the field (in-situ field tests) that are commonly used in Indonesia (SPT). The Standard Penetration Test (SPT) testing data was used in the study.

1. Standard Penetration Test (SPT)

A standard penetration test, also known as an SPT, is a method used in the field or on the job site to determine or obtain the bearing capacity of the soil directly on the project.

In addition, this test also aims to determine both the dynamic resistance of the soil and soil sampling with pounding techniques. This SPT test is a dynamic experiment conducted in a borehole by inserting a sample tube with an inner diameter of 35 mm to a depth of 305 mm using a 63,5 kg free-falling hammer from a height of 760 mm. The number of blows of the hammer to insert the sample tube as deep as 305 is expressed as the value of N. The implementation is carried out in three stages where the first stage is a temporary stand. The number of blows to insert in the second and third stages is summed up to obtain the value of N blows or SPT resistance expressed in crashes per 30 cm. second and third stages is summed up to obtain the N blow value or SPT resistance expressed in blows per 30 cm. The sequence of SPT testing can be seen in Figure 3.1 below.

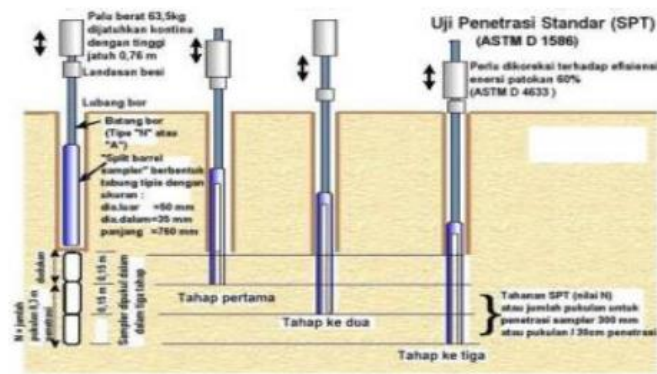


Figure 3.1 SPT Test

(Source: SNI 4153:2008)

3.3 Bore Pile Foundation

One type of foundation used to receive and transmit loads from above to the ground is the pile foundation. to receive and forward the load from the construction

above it to the soil at a certain depth. The soil has a specific depth. When the basic soil lacks the bearing capacity to support the load of the building above it, or when hard soil with sufficient bearing capacity is located at a great depth, piles are used. This type of pile foundation can also be used to support buildings that are subject to upward lifting forces, such as high-rise buildings that are subject to overturning forces caused by wind loads.

According to distribute the load, bore pile foundations are divided by 2, which is:

1. Point-bearing piles are piled with end resistance. These piles transmit the load through the end resistance to the hard hard soil.
2. Friction pile This type of pile is divided into 2 types based on the type of soil where it is placed.
 - a. Friction piles in soils with coarse grains and highly channelize easily transmit water. These piles transmit the load to the ground through skin shear. In the process of driving, these piles will cause reduce the soil pores and make the surrounding soil compact.
 - b. Friction piles on soils with very fine grains and difficult to channel water. These piles also transmit load to the ground via the skin, but the piling process does not compact the soil between the piles. between the piles to compact.

There are 2 methods commonly used in pile foundation installation. These methods are distinguished based on the tools used, namely drop the hammer and hydraulic jack. In installation using a drop hammer, the pile is hit with a hammer (drop hammer) so that it is stuck at a predetermined point on the ground. This tool is shaped like a hammer that is placed on the top of the pile to be driven. On the head of the pile is given a cap or cap which aims to prevent the pile from being damaged when hit with a hammer. Another tool is the hydraulic jack. This tool is placed in parallel with the pile to be staked. To press the pile, a pressure plate is placed at the top of the pile and then the pile is pressed into the ground. This system will cause the pile to be pressed continuously into the ground without causing noise and vibration.

According to their use, piles have several advantages and disadvantages. The advantages of using pile foundations are as follows.

1. The piles are produced in the factory, so it can be carried out quality inspection can be carried out strictly so that the piles produced have the maximum quality,
2. on small-scale piling work, does not require large costs a large cost,
3. the piling procedure is not affected by groundwater,
4. has sufficient pile availability because it is made in the factory,
5. the construction work is easy to supervise because the bearing capacity of the piles can be estimated by using a dial, and
6. piles can be driven to the desired depth.

The disadvantages of using pile foundations are as follows.

1. Soil surface bulging and soil disturbance due to piling may cause problems,
2. piling causes noise disturbance, vibration, and soil deformation which can damage surrounding buildings,
3. if the diameter of the pile is large, a large piling tool is also needed,
4. piling is done carefully to avoid damage to the pile head, on the pile head, and
5. if the length of the pile is less, it requires a connection with a special connector tool.

3.4 Bearing Capacity of Single Pile Foundation

The bearing capacity of a pile refers to its ability to support a load (Hardiyatmo, 2011). Loading on soil, such as foundation loads, causes it to distort and settle. When the load is increased, the decrease also increases. Under constant load, the foundation will experience a significant decrease at some point. This indicates that the bearing capacity has been depleted. Based on this, calculating the foundation's bearing capacity is critical so that the foundation load does not put too much strain on the soil below.

The calculation of the bearing capacity of the pile foundation can be divided into 2 based on the number of piles, namely the bearing capacity of a single pile and the bearing capacity of a group of piles.

3.4.1 Bearing Capacity of Single Pile Meyerhof Method (1956)

The Meyerhof technique can be used to calculate the bearing capacity of a single pile by considering the thread load on the pile's wall and tip. The Meyerhof method is used to calculate the bearing capacity of a single pile as follows:

1. End Bearing Capacity (Q_b)

The end bearing capacity is calculated using Equations 3.1.

$$Q_b = 20 \cdot N_b \cdot A_b \quad (3.1)$$

Description:

Q_p = End bearing capacity (T),

N_b = The statistical mean of the SPT numbers within an area approximately 8D above to 4D below the pile point,

A_b = Cross-sectional area of bored pile (m²).

2. Skin Friction (Q_s)

Skin friction of the pile can be calculated based on Equation 3.2 as follows.

$$Q_s = X_m \cdot N - \text{SPT} \cdot P \cdot L_i \quad (3.2)$$

Description:

$X_m = 0.2$ for bore pile,

L_i = length of soil layer (m),

p = pile circumference (m),

$N\text{-SPT}$ = Number of statistical average stroke calculations.

3. Ultimate Bearing Capacity (Q_u)

Ultimate bearing Capacity can be calculated based on Equation 3.3 as follows.

$$Q_u = Q_p + Q_s \quad (3.3)$$

Description:

Q_u = Ultimate bearing capacity of the pile (T),

Q_p = Bearing capacity of pile tip (T),

Q_s = pile cover capacity (T).

3. Allowable support capacity

The allowable bearing capacity can be calculated based on Equation 3.4 as follows as follows.

$$Q_{all} = \frac{Q_u}{SF} \quad (3.4)$$

Description:

Q_{all} = Allowable support capacity,

Q_u = ultimate pile capacity.

SF = safe factor (2 – 4)

3.4.2 Bearing Capacity of Single Pile Reese and Wright Method

1. End bearing capacity (Q_b)

The Reese and Wright method is one way for determining the bearing capacity of a single pile. The Reese and Wright Equation 3.5 is shown below.

$$Q_p = q_p \cdot A_p \quad (3.5)$$

Description:

Q_p = Pile tip support capacity (T),

A_p = Cross-sectional area of bored pile (m^2).

2. Skin Friction at Pile (Q_s)

Skin Friction at pile can be calculated based on Equation 3.6 as follows.

$$Q_s = f \cdot L_i \cdot A_{st} \quad (3.6)$$

$$f = \alpha \times C_u \quad (3.7)$$

Description:

A_{st} = cross-sectional area of pile tip (m),

L_i = Length of soil layer (m),

f = Skin friction unit resistance (kN/m²).

C_u = Soil Cohesion (kN/m²) = $\frac{2}{3} \times N - SPT \times 10$

3. Total Bearing Capacity of The Pile

Total bearing capacity of the pile can be calculated based on Equation 3.8 as follows.

$$Q_u = Q_p + Q_s \quad (3.8)$$

Description:

Q_u = Ultimate bearing capacity of the pile (T),

Q_p = Bearing capacity of pile tip (T),

Q_s = pile cover capacity (T).

4. Allowable support capacity

The allowable bearing capacity can be calculated based on Equation 3.9 as follows.

$$Q_{all} = \frac{Q_u}{SF} \quad (3.9)$$

Description:

Q_{all} = Allowable support capacity,

Q_u = ultimate pile capacity,

SF = safe factor (2 – 4).

3.5 Group Pile

The bearing capacity of a friction pile in clay soil will deteriorate as pile spacing increases. This is the total bearing capacity of a collection of friction piles. Piles in clay soils are frequently less than the result of a single pile in the group's

bearing capacity. As a result, the magnitude of the reduced total bearing capacity is determined by the size, shape, spacing, and length of the piles.

3.5.1 The number of piles

To determine the number of group piles, divide the value of the maximum load amount by the allowable bearing capacity of the foundation.

1. Number of Group Piles

To get the number of group piles can be seen in Equation 3.10 below.

$$n = \frac{\Sigma_{pmax}}{Q_{all}} \quad (3.10)$$

Description:

n = Number of pile groups,

Σ_{pmax} = Maximum load on foundation pile plan (Ton),

Q_{all} = Maximum bearing capacity of foundation pile plan (Ton).

Rounding up the results of determining the number of group foundation piles should be done to increase the foundation safety factor due to the loads that occur.

2. Distance Pile

The spacing between piles based on *Dirjen Bina Marga Dapartemen P.U.T.L.* is required by Equations 3.11 and 3.12 below.

$$s \geq 2,5D \quad (3.11)$$

$$a \geq 1,5D \quad (3.12)$$

Description:

s = distance pile to pile

a = distance pile to edge

D = diameter

3. Pile Efficiency

The approach to estimating pile efficiency suggested by the "Converse-Labarre Formula" is Equation 3.13 below.

$$E_g = 1 - \theta \left(\frac{(n-1)m + (m-1)n}{90 \times m \times n} \right) \quad (3.13)$$

Description:

E_g = pile efficiency

$\theta = \tan^{-1}\left(\frac{d}{s}\right)$

m = number of piles in x direction

n = number of piles in y direction

4. Create A Pile Arrangement Plan

The number of piles determines the arrangement of the group pile plan. An example of an arrangement of piles used in general can be seen in Figure 3.5 below.

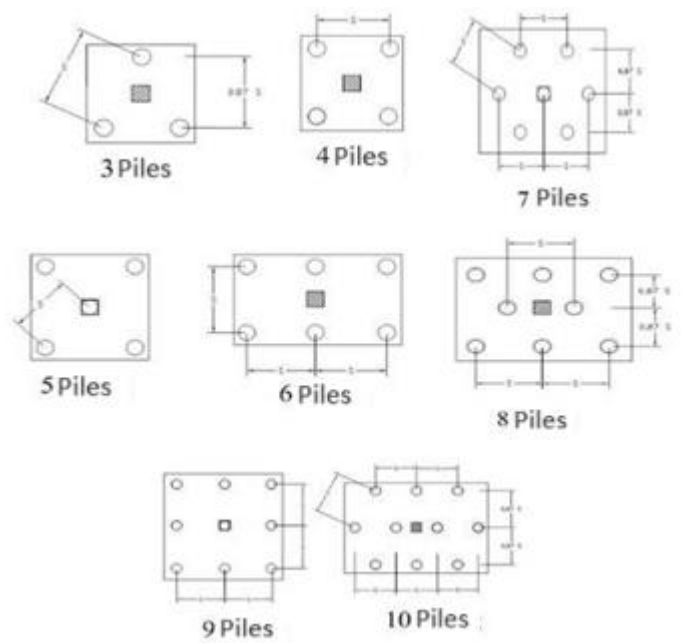


Figure 3.2 Common Pile Arrangements Used in the Field

3.5.2 Bearing capacity of group piles

The ultimate bearing capacity can be calculated using Equation 3.14 below.

$$Q_g = E_g \times n \times Q_{all} \quad (3.14)$$

Description:

Q_g = Bearing capacity of group pile

E_g = pile efficiency

n = total number of the piles

Q_{all} = allowable bearing capacity

3.5.3 Control of Pile Compressive Force against Axial and Momentary Loads

The pile's reaction to the force is depicted in Figure 3.6 below.

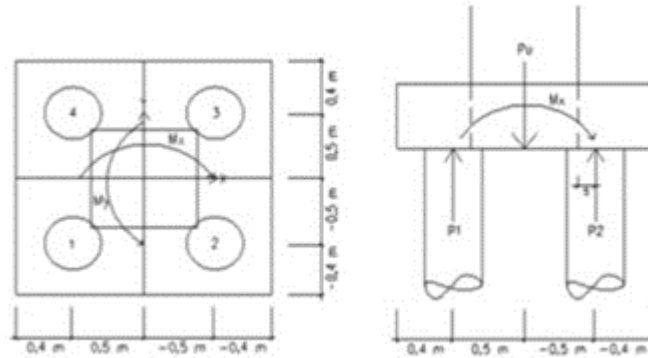


Figure 3.3 Vertical Force and Moment of the Pile Group in the Axis Direction

Following the determination of the number of piles and the design for group piles, the group foundation is regulated against the forces and moments that will occur in each foundation using Equations 3.15 and 3.16 below.

$$P_{max} = \frac{\Sigma p}{n} + \frac{M_x \cdot y}{\Sigma y^2} + \frac{m_y \cdot x}{\Sigma x^2} \quad (3.15)$$

$$P_{min} = \frac{\Sigma p}{n} - \frac{M_x \cdot y}{\Sigma y^2} - \frac{m_y \cdot x}{\Sigma x^2} \quad (3.16)$$

Description:

P_{max} = Max load that the pile foundation will receive (kN),

Σ_p = Total axial load (kN),

n = Number of piles uses.

M_x = X-direction moment (kNm),

M_y = Y-direction moment (kNm).

3.6 Pile Foundation Settlement

The pile decline can be classified into two types: pile decline and pile group settlement, with the amount of settlement affected by soil features and the spread of foundation pressure to the soil below.

3.6.1 Elastic Settlement

Elastic settlement of a pile foundation is the vertical displacement of the pile due to elastic deformation of the surrounding soil when it receives a load. This deformation is reversible, meaning that the pile will return to its original position if the load is removed.

Vesic (1977) separated the total settlement s_e of the pile head into three components and suggested expressions to determine them from elastic analysis. The three elastic components are:

1. Elastic shortening of the pile
2. Settlement of the pile due to the working load (Q_{wp}) at the pile point
3. Settlement of the pile due to the working load (Q_{ws}) along the pile shaft

The total elastic settlement of the pile head under the working load of Q_w is given by Equation 3.17 below.

$$S_e = S_{e(1)} + S_{e(2)} + S_{e(3)} \quad (3.17)$$

If there is no skin friction along the pile shaft, the pile load remains the same (i.e., Q_w) at all depths, and the elastic shortenizang becomes $Q_w L / A_p E_p$. Due to skin

friction, $S_{e(1)}$ will be different and depends on the way the skin friction varies along the pile shaft. It can be shown theoretically in Equation 3.18 below.

$$S_{e(1)} = \frac{(Q_{wp} + \xi \times Q_{ws})L}{A_p E_p} \quad (3.18)$$

Where:

- Q_{wp} = working load at the pile point,
- Q_{ws} = working load along the pile shaft,
- A_p = cross-sectional area of the pile point,
- L = pile length,
- E_p = modulus of elasticity of the pile material,
- ξ = a constant in the range of 0.50–0.67, depending on the skin friction distribution.

The settlement of a pile caused by the load carried at the pile point may be expressed in the Equation 3.19 below.

$$S_{e(2)} = \frac{Q_{wp} C_p}{D q_p} \quad (3.19)$$

Where:

- D = width or diameter of pile,
- Q_{wp} = working load at the pile point,
- q_p = ultimate point resistance of the pile,
- C_p = an empirical coefficient.

The settlement of a pile caused by the load carried by the pile shaft is given by a relation like Equation 3.20 below.

$$S_{e(3)} = \left(\frac{Q_{ws}}{p \times L} \right) \frac{D}{E_s} (1 - \mu_s^2) I_{ws} \quad (3.20)$$

Where:

- p = perimeter of the pile,
L = embedded length of pile,
 I_{ws} = influence factor.

Note that the term $Q_{ws}/\gamma pL$ is the average value of f along the pile shaft. The influence factor, I_{ws} , has a simple empirical relation (Vesic, 1977):

$$I_{ws} = 2 + 0,35 \sqrt{\frac{L}{D}} \quad (3.21)$$

Then, to see the value of modulus of elasticity of soil (E_s) and Poisson's ratio of soil (μ_s), can be seen in the Table 3.2 and Table 3.3.

Table 3.2 Poisson Ratio of Soil

Soil type	ν
Saturated clay	0,4 – 0,5
Unsaturated clay	0,1 – 0,3
Sandy loam	0,2 – 0,3
Silt	0,3 – 0,35
Dense sand	0,2 – 0,4
Coarse sand (Pore number, $e = 0.4-0.7$)	0,15
Fine sand (Pore number, $e = 0.4 - 0.7$)	0,25
Stone (depending on type)	0,1 – 0,4
Loose	0,1 – 0,3

Table 3.3 Modulus Elasticity Value for Soil

Soil type	E_s (kN/m ²)
Clay	
Very soft	300 – 3000
Soft	2000 – 4000
Medium	4500 – 9000

Continuation 3.3 Modulus Elasticity Value for Soil

Hard	7000 – 20000
Sandy	30000 – 42500
Sandy	
Silty	5000 – 20000
Not dense	10000 – 25000
Dense	50000 – 100000
Sand and Gravel	
Dense	80000 – 200000
Not dense	50000 – 140000
Silt	2000 – 20000
Loess	15000 – 600000
Shale	140000 - 1400000

3.6.2 Consolidation Settlement

Canonica (1991) and Wesley (2012) developed settlement calculations for pile foundations based on the Boussinesq theory. The stress in the soil due to the weight of the building can be assumed uniformly distributed at a depth of $2/3$ t (pile length) and spread with a spreading angle of 2V:1H in the settlement calculation of end bearing pile groups. The split of soil layers below the boundary line Z0 to Z3 is based on the value of foundation stresses of 20% or more, as shown in the figure below.

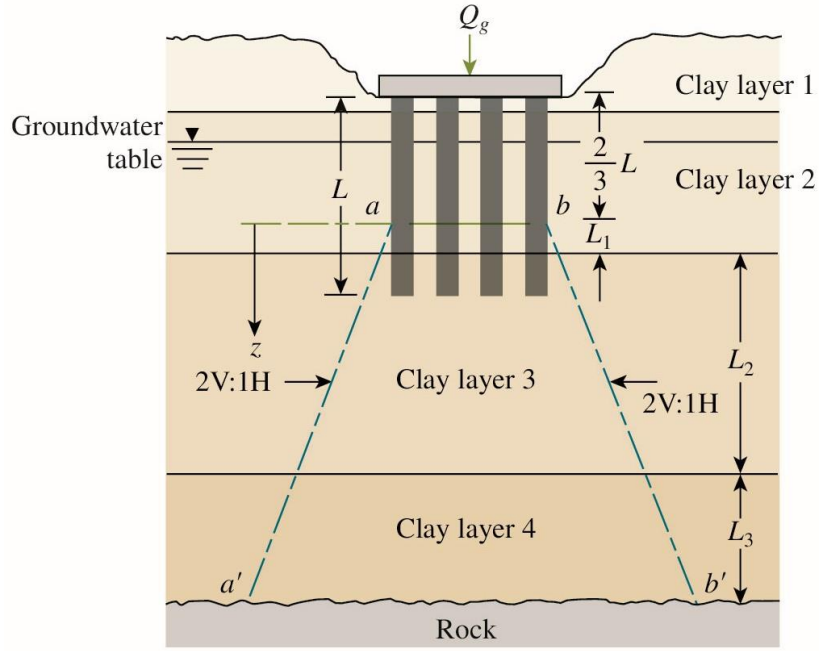


Figure 3.4 Canonica and Wesley Method

The analysis stage of the calculation of the decline in the Canonica (1991) and Wesley (2012) methods as follows.

1. Active Stress ($\Delta\sigma$)

As for calculating the active stress that occurs in the pile. The following equation is used to calculate the active stress that occurs in the pile the following Equation 3.22 below.

$$\Delta\sigma_{z0} = \frac{P}{(B+z_0)(L+z_0)} \quad (3.22)$$

Description:

P = Design Point Load (Kg),

B = Width (cm),

Z_0 = Depth (cm),

L = Height (cm).

2. Consolidation Settlement of Layer i ($\Delta S_{c(i)}$)

Calculate the consolidation settlement of each layer caused by the increased stress. The Equation 3.23 can be seen below.

$$\Delta S_{c(i)} = \left[\frac{\Delta e_{(i)}}{1 + e_o} \right] H_i \quad (3.23)$$

Where:

$\Delta S_{c(i)}$ = consolidation settlement of layer i

$\Delta e_{(i)}$ = change of void ratio caused by the increase in stress in layer i

$e_{(o)}$ = initial void ratio of layer i (before construction)

H_i = thickness of layer i

3.7 Loading on the Bridge According to SNI 1725-2016

Loading for bridge strength analysis serves as the foundation for calculating loads and forces for stress calculations. determining loads and forces for the calculation of stresses that occur in each bridge component. The use of this bridge is intended to achieve safe and economic strength based on local conditions, so that the bridge strength analysis process is effective. The bridge strength analysis process must be efficient. The analysis used as a reference in the calculation is SNI 1725:2016, which is about bridge loading.

Plan loads that are calculated based on the regulations described in SNI 1725:2016 consist of.

1. Permanent load
2. Traffic load
3. Environment load
4. Others load

3.7.1 Permanent Load

The dead load of a bridge is the sum of the weights of all structural and non-structural components. When applying normal and reduced load factors, each of these components must be considered an integral unit of action. In order to determine these components, the bridge planner must apply his expertise. Based on the dimensions shown in the drawings and the specific gravity of the materials used, the mass of each component of the building must be calculated. The weight of the building components is calculated by multiplying the mass by the acceleration of gravity (g). The gravity acceleration used in this standard is 9.81 m/sec^2 .

1. self-weight (MS)

The weight of the section and other structural elements it carries, including in this case the weight of materials and bridge parts that are structural elements, as well as nonstructural elements that are considered fixed, is referred to as self-weight. Table 3.4 shows the load factors used for self-weight.

Table 3.4 Load Factor

Material Type	Weight Factor (Γ_{ms})			
	Service Limit State (Γ_{ms}^s)		Ultimate Limit State (Γ_{ms}^u)	
	Material		Normal	Reduce
Fixed	Steel	1	1,10	0,90
	Aluminium	1	1,10	0,90
	Concreat Pre-Cast	1	1,20	0,85
	Concrete Cast In Place	1	1,30	0,75
	Wood	1	1,40	0,70

(Source: SNI 1725:2016)

2. Additional dead load (MA)

Additional dead load is the weight of all the materials that make up a load on the bridge which is a nonstructural element, and its magnitude can change during the life of the bridge.

3. Load caused by ground pressure (TA)

Soil parameters such as volume weight, cohesiveness, and shear angle should be used to compute the nominal earth pressure coefficient. When a load is applied, the soil behind a retaining wall is frequently subjected to an extra load.

Traffic loads influence a portion of the hypothesized active collapse area. This additional load has the same magnitude as 0.7m thick soil acting equally on the part of the soil crossed by the traffic load. uniformly on the portion of the ground through which the traffic load travels. This additional load is only used to compute lateral pressure.

4. Creep and shrinkage

The effects of creep and shrinkage must be considered when designing concrete bridges. These effects must be calculated using the bridge's dead load.

3.7.2 Traffic Load

1. Lane load “D” (TD)

The "D" lane load is made up of uniformly distributed loads and centered line loads. The "D" lane load acts across the entire width of the vehicle lane, exerting an influence on the bridge comparable to an actual vehicle load. The entire amount of "D" lane load acting is determined by the width of the vehicle lane itself.

- a. A uniformly distributed load has q (kPa), which depends on the total span length L of the load and is expressed by the formula.

Equation 3.24 and Equation 3.25 are as follows:

$$L \leq 30 \text{ m} \quad \rightarrow \quad q = 9,0 \text{ kPa} \quad (3.24)$$

$$L \geq 30 \text{ m} \quad \rightarrow \quad q = 9,0 \times \left(0,5 + \frac{15}{L}\right) \quad (3.25)$$

"L" is the sum of the lengths of each disconnected load of disconnected load. The lane load "D" is placed perpendicular to the direction of traffic.

- b. Line loads (BGT) can be installed anywhere along the bridge that is perpendicular to the traffic direction. BGT is placed in the middle of the span in this planning to obtain the greatest moment in the middle of the span. The intensity of BGT is $P = 49.0 \text{ kN/m}$.

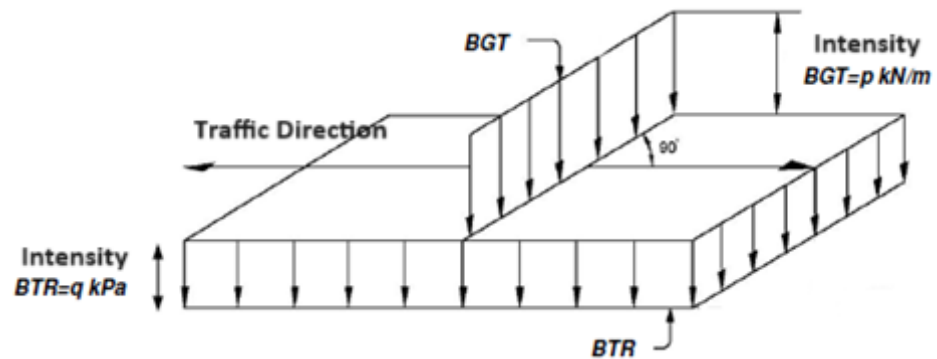


Figure 3.5 Lane Load "D"

(Source: SNI 1725:2016, Section 8.3)

Terms of load distribution "D" in the transverse direction of the bridge intensity are used 100% to obtain moments and shear in the longitudinal direction on the bridge girder.

2. Truckload (T)

Truckload "T" is made up of a semi-trailer truck vehicle with the axle arrangement and weight shown in Figure 3.3 divided into two equally distributed loads that form the contact area between the wheels and the floor surface. The distance between the two axles can be adjusted between 4.0 and 9.0 meters to achieve the greatest effect in the bridge's longitudinal direction.

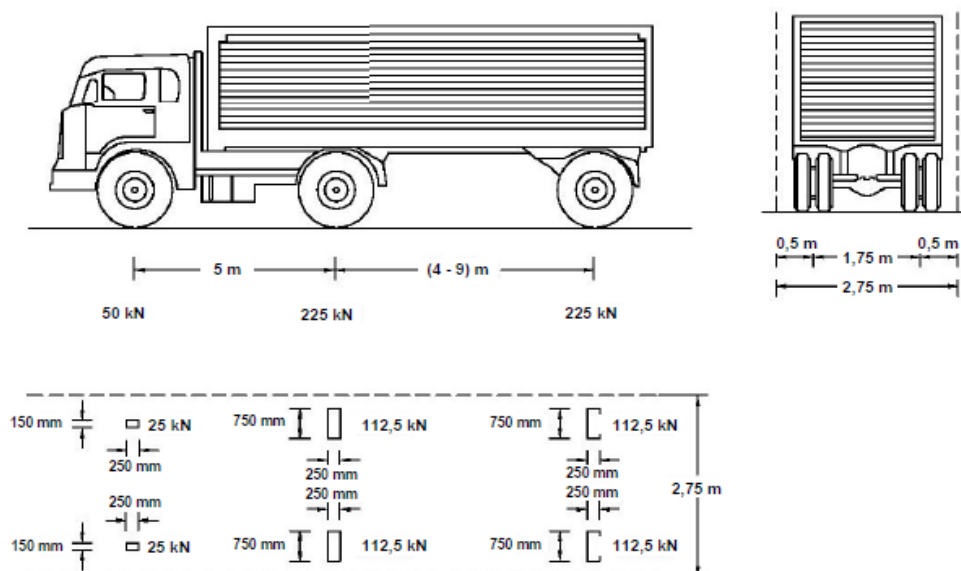


Figure 3.6 Truck Load "T"

(Source: SNI 1725:2016, Section 8.4)

3. Dynamic load factor

The interaction between the moving vehicle and the bridge results in the dynamic load factor (FBD). To simulate the shock from moving vehicles on bridge structures, dynamic load factors are applied to "D" and "T" truck loading. The FBD line load is depicted in Figure 3.4.

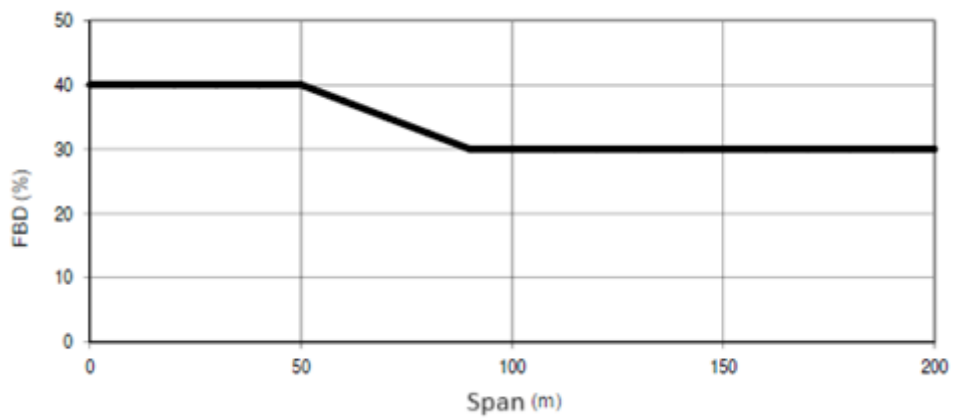


Figure 3.7 Dynamic Load Factors

(Source: SNI 1725:2016)

Table 3.5 Dynamic Load Factor for KEL Line Load

Equivalen Span L_E (M)	DLA (For Both Limit Conditions)
$L_E < 50$	0,4
$50 < L_E < 90$	$0,525 - 0,0025 L_E$
$L_E > 90$	0,3

Notes:

For simple spans L_E = actual span length. Find the continuous span based on the following Equation 3.26 below.

$$\text{For continuous spans } L_E = \sqrt{L_{\text{average}} - L_{\text{max}}} \quad (3.26)$$

Descriptions:

L_{average} = Average span length of continuous span,

$L_{\max}$ = Maximum span length of continuous span.

4. Brake forces

The brake force is considered to act horizontally in the direction of the bridge axis at 1.8 m from the vehicle floor surface. This effect must be considered as equal to with a brake force equal to:

- a. 25% from plan truck weight plus evenly divided lane load BTR
- b. 5% of design truck axle weight

5. Pedestrian loads (TP)

All elements of pavements wider than 6 m should be planned for nominal load of 5 kPa and shall be considered to act simultaneously with the vehicle load of each vehicle lane.

3.7.3 Environmental Loads

1. Wind loads

a. Wind on the structure (EWs)

The wind load should be assumed to be evenly distributed on the surfaces exposed to the wind. The area considered is the area of all components taken perpendicular to the wind direction, thus the following Equation 3.27 is obtained.

$$V_{DZ} = 2,5V_0\left(\frac{V_{10}}{V_B}\right) l\left(\frac{Z}{Z_0}\right) \quad (3.27)$$

Description:

V_{DZ} = Plan wind speed at plan elevation, Z (km/h),

V_{10} = Wind speed at an elevation of 10 m above ground or above planned water level 90 - 126 km/hour,

V_B = Plan wind speed 90 - 126 km/h,

Z = Elevation of the structure measured from the ground or from the water level where the wind load is calculated. water where the wind load is calculated,

V_0 = Wind friction speed (Table 3.6),

Z_0 = Friction length upstream of the bridge (Table 3.6).

Table 3.6 V_0 and Z_0 Values for Various Upstream Surface Condition Variations

Condition	Open Field	Sub-Urban	City
V_0 (mm)	13,2	17,6	19,3
Z_0 (mm)	70	1000	25000

Hence, the wind load on the structure can use Equation 3.28 as follows.

$$P_D = P_B \left(\frac{V_{DZ}}{V_B} \right)^2 \quad (3.28)$$

Description:

P_B = Basic wind pressure (Table 3.7)

Table 3.7 Base Wind Pressure

Upper Building Components	Wind Pressure (MPa)	Suction Wind (MPa)
Frame, Column, And Arches	0,0024	0,0012
Beam	0,0024	N/A
Flat Surfaces	0,0019	N/A

The P_D value must be greater than 4.4 kN/m.

For the lower structure, the base wind pressure is 0.0019 Mpa

b. Wind load on the vehicle

The design wind pressure for both the bridge structure and vehicles crossing the bridge must be determined. The bridge should be designed to withstand the force of wind pressure on the vehicle, which is assumed to be 1,46 N/mm continuous pressure.

2. Load Factors and Load Combinations

Based on SNI 1725:2016 there is a combination of loading in designing bridge structures. The following is the loading combination. Each loading combination must take a load factor based on Table 3.8.

1. *Kuat I*

A bridge combination that considers the forces that arise on the bridge under normal circumstances without considering wind loads. In this limit state, all nominal forces that occur are multiplied by the appropriate load factor.

2. *Kuat II*

A loading combination associated with the use of the bridge to carry special vehicle loads specified by the owner without considering wind loads.

3. *Kuat III*

A loading combination where the bridge is subjected to wind loads of 90 km/h to 126 km/h.

4. *Kuat IV*

A loading combination to account for the possibility of a large ratio of dead load to live load.

5. *Kuat V*

The loading combination relates to the normal operation of the bridge by considering wind loads at speeds of 90 km/h to 126 km/h.

6. *Ekstrim I*

A combination of live factor earthquake loading that considers the operation of live loads during an earthquake must be determined based on the importance of the bridge.

7. *Ekstrim II*

A loading combination that considers the combination of reduced live loads with loads arising from ship collisions, vehicle collisions, floods or other hydraulics loads, except for loading cases due to vehicle collisions (TC). Flood loading cases shall not be combined with loads due to vehicles and vessels.

8. *Layan I*

A loading combination related to bridge operations with all loads having nominal values and considering wind loads at speeds of 90 km/h to 126 km/h. This combination is also used to control deflection. This combination is also used to control deflections in steel culverts, tunnel lining plates, thermoplastic pipes and to control the crack width of reinforced concrete structures, and for tensile strain analysis of segmental concrete bridge cross sections. This loading combination should also be used for slope stability.

9. *Layan II*

A loading combination intended to prevent melting of steel structures and slippage of joints due to vehicle loads.

10. *Layan III*

A loading combination for calculating tensile stresses in the longitudinal direction of prestressed concrete bridges with the aim of controlling the magnitude of cracks and tensile main stresses in the body of segmental concrete bridges.

11. *Layan IV*

A loading combination for calculating tensile stresses in prestressed concrete columns with the aim of controlling the magnitude of cracking.

12. *Fatik*

A combination of fatigue and fracture loads with respect to fatigue life due to induced loads of infinite duration.

The load factor must be selected in such a way as to produce extreme conditions due to the working load. For each loading combination, the maximum and minimum extreme conditions must be investigated, where later the value will be used in the bridge design planning that will be carried out more fully can be seen in Table 3.8 below.

Table 3.8 Load Combinations and Load Factors

Limit vehicle	MS MA TA PR PL SH	TT TD TB TR TP	EU	EW _s	EW _L	BF	EUn	TG	ES	Use One of The Following		
										EQ	TC	TV
<i>Kuat I</i>	γ_p	1,8	1,00	-	-	1,00	0,50/1,20	γ_{EQ}	γ_{ES}	-	-	-
<i>Kuat II</i>	γ_p	1,4	1,00	-	-	1,00	0,50/1,20	γ_{EQ}	γ_{ES}	-	-	-
<i>Kuat III</i>	γ_p	-	1,00	1,4	-	1,00	0,50/1,20	γ_{EQ}	γ_{ES}	-	-	-
<i>Kuat IV</i>	γ_p	-	1,00	-	-	1,00	0,50/1,20	-	-	-	-	-
<i>Kuat V</i>	γ_p	-	1,00	0,40	1,00	1,00	0,50/1,20	γ_{EQ}	γ_{ES}	-	-	-
<i>Ekstrim I</i>	γ_p	γ_{EQ}	1,00	-	-	1,00	-	-	-	1,00	-	-
<i>Ekstrim II</i>	γ_p	0,50	1,00	-	-	1,00	-	-	-	-	1,00	1,00
<i>Layan I</i>	1,00	1,00	1,00	0,30	1,00	1,00	0,50/1,20	γ_{EQ}	γ_{ES}	-	-	-
<i>Layan II</i>	1,00	1,30	1,00	-	-	1,00	0,50/1,20	-	-	-	-	-
<i>Layan III</i>	1,00	0,80	1,00	-	-	1,00	0,50/1,20	γ_{EQ}	γ_{ES}	-	-	-
<i>Layan IV</i>	1,00	-	1,00	0,70	-	1,00	0,50/1,20	-	-	-	-	-
<i>Fatik</i>	-	0,75	-	-	-	-	-	-	-	-	-	-

(Source: SNI 1725:2016)

3.8 Plaxis 3D

PLAXIS 3D is a finite element software used in geotechnical applications in three-dimensional view. The program utilizes soil models to simulate soil behavior and is capable of modeling actual soil conditions both in plain strains and axisymmetrically. PLAXIS 3D also provides an easy-to-use graphical interface, allowing users to quickly model geometry and element meshes based on transverse and longitudinal cross sections.

For the simulation process in PLAXIS 3D, it consists of 3 stages which are explained below.

1. Data input

This involves the steps of creating and adjusting the model geometry to produce a finite element model that reflects the actual conditions. In addition, this step also includes inputting parameters such as soil data, pile modeling, and load modeling. To input the data can be seen in the Figure 3.8 and Figure 3.9 below.

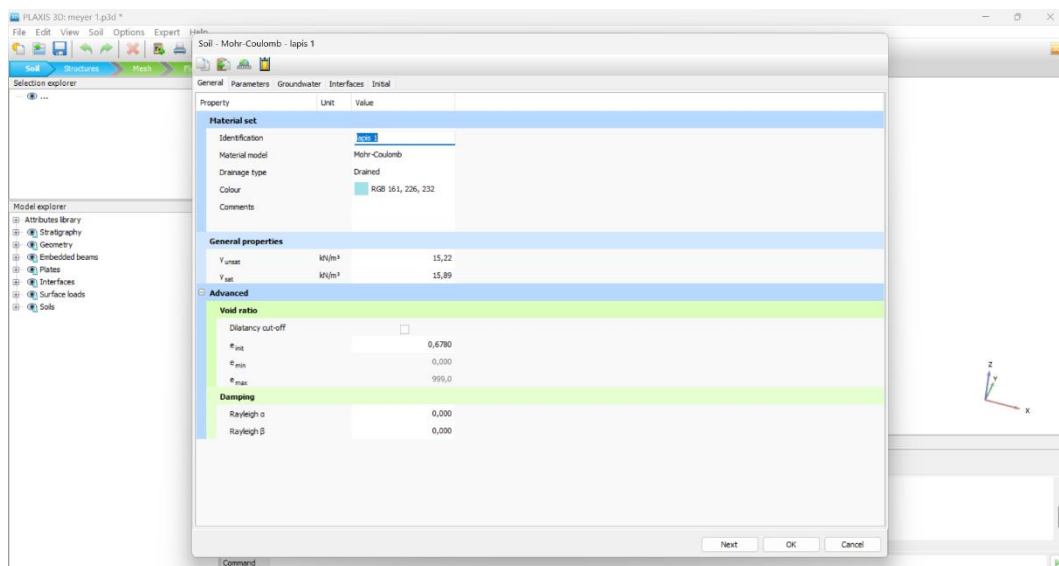


Figure 3.8 Input Parameters Soil

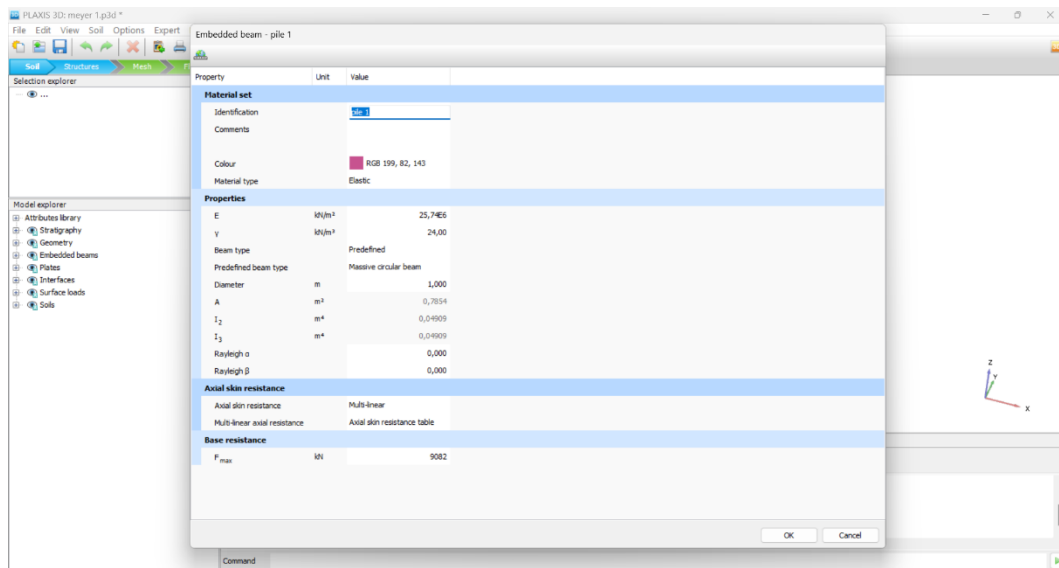


Figure 3.9 Input Parameter Pile

2. Calculate

This stage involves using the completed model and requires us to select appropriate calculation settings. This includes settings such as general calculation settings, calculation control parameters, load multiplier, stage construction, and Phi-c reduction to obtain the safety factor (SF). To input make the stage of construction can be seen in the Figure 3.10 below.

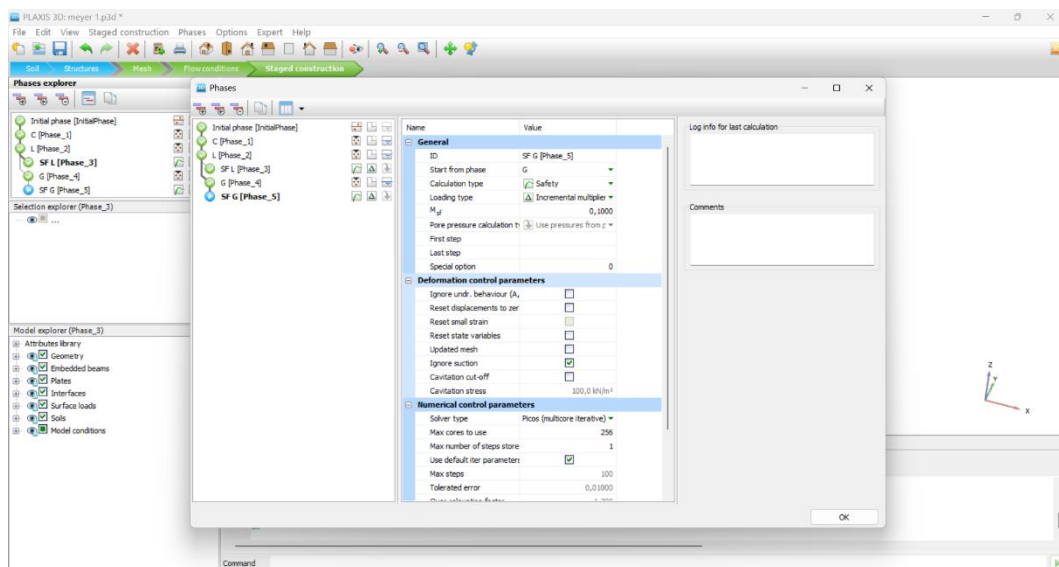


Figure 3.10 Stage Construction

3. Data output

This is the result of the calculation where the calculation continues until the equilibrium condition is reached. The output results of the program include deformation mesh, settlement, and factor of safety. To see the output data can be seen in the Figure 3.11 below.

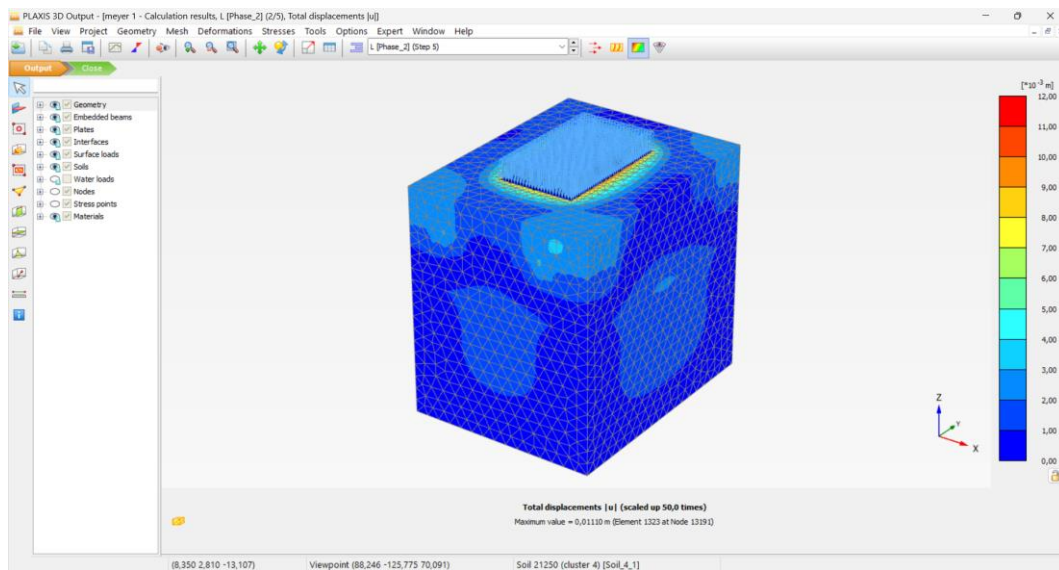


Figure 3.11 Output Result

CHAPTER IV

RESEARCH METHOD

4.1 Object and Research Location

The object that will be used in this research is a bored pile foundation in the Tawang-Ngalang segment IV road construction project in the area Nglegi, Kapanewon Patuk, Gunung Kidul. For details of the station location can be seen in Figure 4.1 below. While the object of research is the bearing capacity of the Bored Pile foundation and the settlement of the bored pile foundation of the abutment 1 structure of the Tawang-ngalang Road Bridge which can be seen in Figure 4.2.

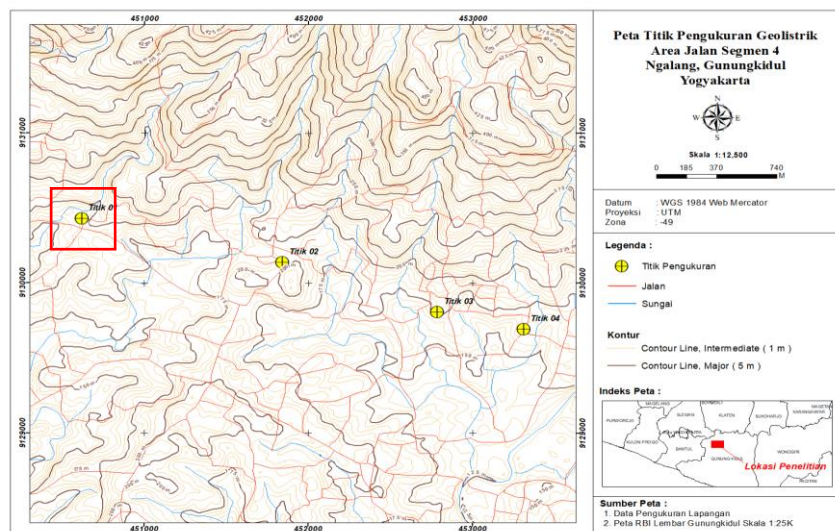


Figure 4.1 Location Map

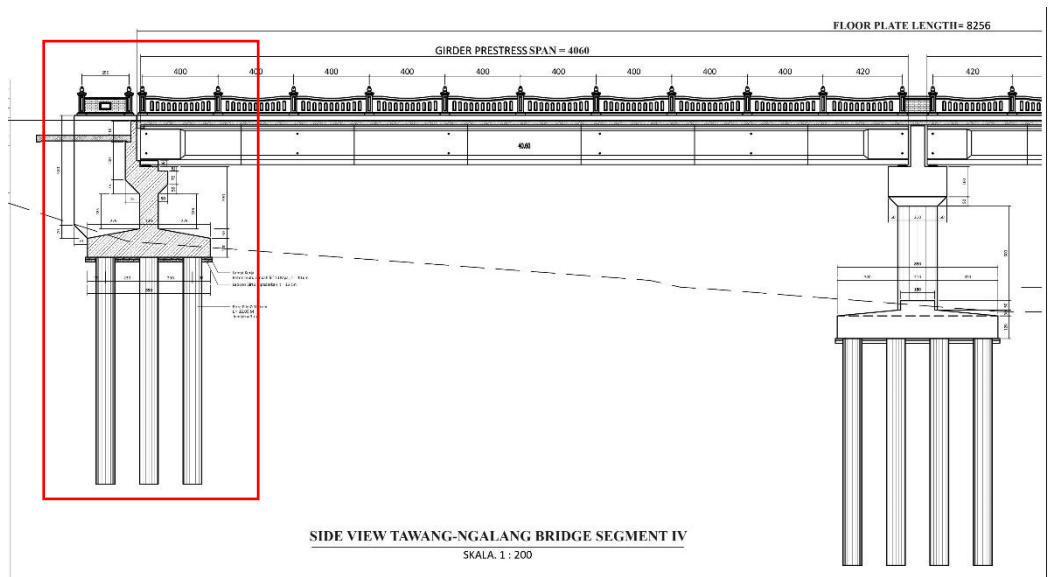


Figure 4.2 Longitudinal Section of Tawang-Ngalang Bridge Abutment 1

4.2 Research Method

A research method is a method of solving problems or developing science that employs scientific methods. More broadly, research methods are scientific methods for obtaining valid data with the goal of discovering, developing, and improving knowledge. to obtain valid data in order to discover, develop, and prove specific knowledge that can then be found, developed, and proven. and demonstrated, a specific knowledge that can then be used to understand, solve, and anticipate problems. used to understand, solve, and anticipate problems.

4.3 Data Collection

In this research, data were obtained from the tawang-ngalang bridge project. The data used to achieve the aims and objectives of the study are as follows:

1. Technical drawings Tawang-Ngalang bridge

The building structure data used is the bridge structure drawing, including the location map and details of the research location, then the bridge detail drawing in the form of Bored Pile foundation details and abutment 1 details.

2. Data on the results of soil investigations with the standard penetration test method

Soil data used is based on the results of the soil investigation of the Tawang-Ngalang Bridge and Bridge Construction Project segment IV. Soil investigation tests in the form of soil data from drilling with a drilling machine as many as 4 points with depths between 10-20 m, soil descriptions during drilling, the location of the groundwater table when drilling and Standard Penetration Test (SPT) tests are at a depth of 3,5 m. Soil investigation with a machine drill aims to determine the condition of the soil layers. From the results of the investigation, several types of soil layers were obtained at point BH.01, namely Red colored sandy silt with really soft consistency at a depth of $\pm$ 0,00 m to -3,00 m, Sandy clay with medium to high plasticity, medium consistency from a depth of -4,00 m to -7,00 m, Sandy material with loose material, fine to coarse size at a depth of -8,00 m, and Andesitic breccia and massive strength rock -9,00 m to -20,00 m.

4.4 Analysis Steps

4.4.1 Load Analysis

Loading analysis is used to determine how much load the foundation will receive. The author conducts a loading analysis by calculating the loads acting on the bridge structure based on the SNI 1725:2016 loading regulations and assisted by a computer program namely Microsoft Excel.

4.4.2 Bored Pile Foundation Analysis

This analysis is conducted to analyze the bearing capacity of drilled pile foundations with diameters of 0,8 m, 0,9 m, 0,95 m and 1 m with a depth of 12 m, as follows.

1. The steps to analyze the bearing capacity of a single pile using the Meyerhof and Reese & Wright methods are described as follows:
 - a. Find the ultimate end resistance value of the pile,
 - b. Find the value of skin friction of the pile,
 - c. Find the ultimate bearing capacity, and
 - d. Find the allowable bearing capacity.

2. Analysis of bearing capacity of group piles

Factors that affect the bearing capacity of the group are the efficiency factor and the number of piles.

4.4.3 Settlement of Piles Analysis

This analysis is carried out to determine the settlement that occurs in single poles and groups of poles.

1. The steps to analyze the elastic settlement of a single pile are as follows:
 - a. Calculate the settlement of floating poles or friction piles,
 - b. Calculate the settlement of the end bearing pile, and
 - c. Calculate the total settlement.
2. The steps to analyze the consolidation settlement of a pile group using the 2:1 spreading method are as follows:
 - a. Calculate the immediate settlement,
 - b. Calculate the consolidation settlement, and
 - c. Calculate the total settlement

4.4.4 Discussion

The discussion is a description of the results of the analysis carried out, namely the analysis of the bearing capacity of the Bored Pile foundation and the use of the Bored Pile foundation in the abutment 1 structure in the Tawang-Ngalang Bridge and road construction project segment IV.

4.4.5 Conclusion and Suggestions

A conclusion is a final judgment based on the information that has been previously outlined. Conclusions can contain facts, views, or arguments about a subject. Meanwhile, suggestions are solutions that have proven effective in solving the problems that arise. Based on the results of the analysis that has been carried out, we can conclude some conclusions and provide suggestions to achieve optimal results.

4.5 Literature Review

Any effort made by the researcher to find information relevant to the topic or problem that will or is being researched is referred to as a literature review. being

investigated. Scientific books, research reports, essays, theses and dissertations, and regulations can all provide this information. research reports, scientific essays, theses and dissertations, regulations, regulations, decrees, yearbooks, encyclopedias, and other printed and electronic sources. The review of literature is an activity that cannot be separated from research.

4.6 Flowchart

A research flow chart is a diagram that represents a process or system to document, plan, refine, and illustrate a workflow with many steps or stages. The figure below illustrates the flow chart of the Final Project research and the flow chart for using the Plaxis 3D software.

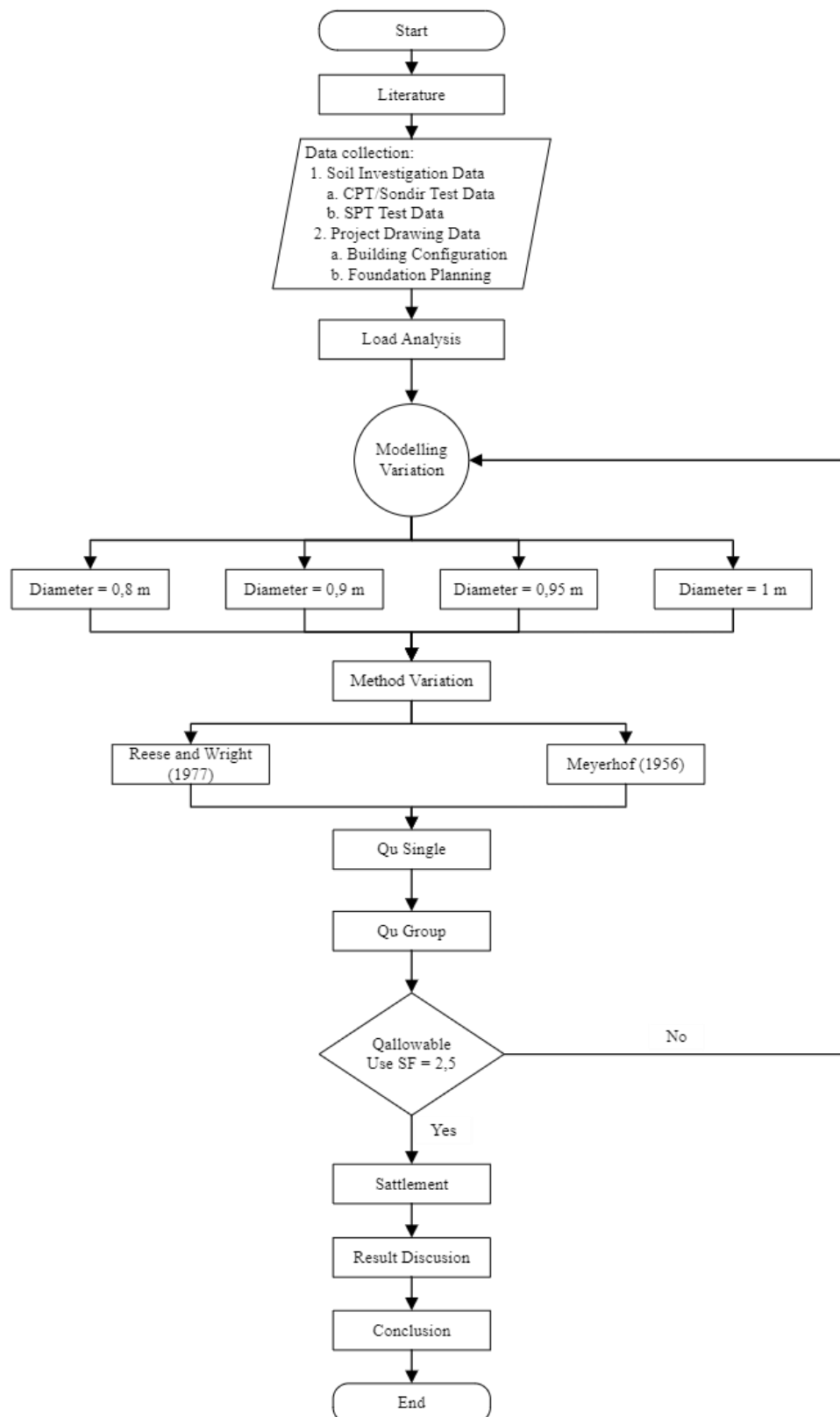


Figure 4.3 Final Project Research Flow Chart



Figure 4.4 Plaxis 3D Software Usage Flow Chart

CHAPTER V

RESULTS AND DISCUSSION

5.1 Bridge Data

The Tawang-Ngalang Bridge construction project is in Gedangsari District, Gunung Kidul Regency, DIY. The bridge is planned to have a length of 81,2 m and a bridge width of 11 m using a reinforced concrete structure that uses a foundation plan, namely a bored pile foundation. The map project can be seen in Figure 5.1 below.

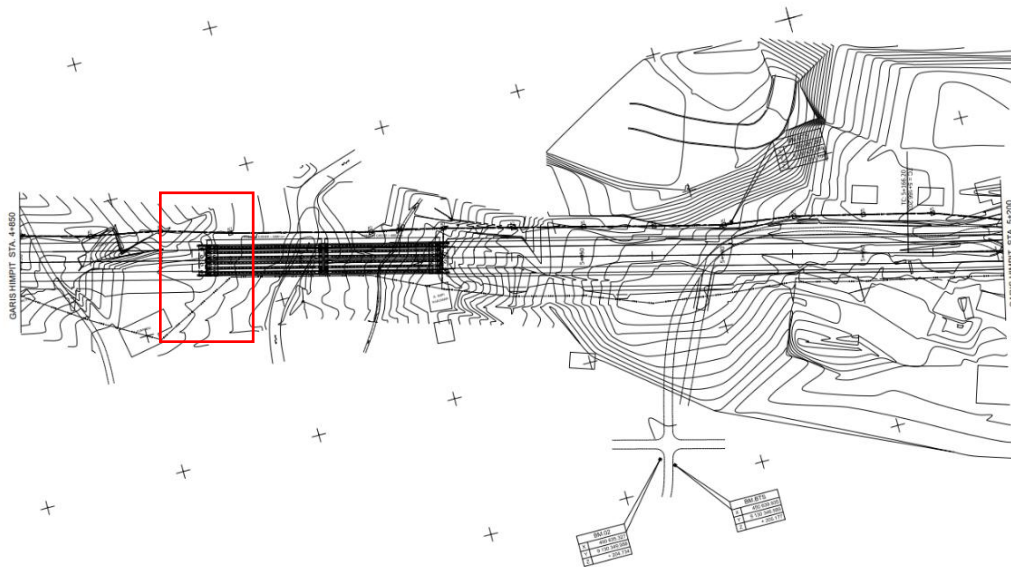


Figure 5.1 Tawang Ngalang Segment IV Bridge Plan

5.2 Bridge Load Analysis

The design of bridge structures is influenced by several reactions that occur on the upper structure of the bridge. Loading reactions on the bridge there are several factors that affect the load of the bridge structure. The effect of this load

must be adjusted to the loading combination according to Bridge Loading SNI 1725:2016.

5.2.1 Self-Weight (MS)

1. Upper Structure

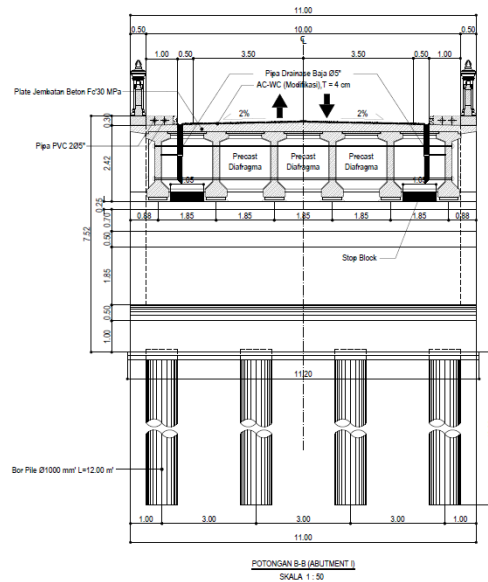


Figure 5.2 Upper Structure

Self-weight is the weight of materials and parts of the bridge that are part of the structural elements, plus non-structural elements that are carried and are fixed. The structure of the Tawang-Ngalang Bridge consists of a 40,6 m span, 11 m bridge width. This structural data can be seen in Table 5.1 below.

Table 5.1 Weight of Bridge Upper Structure

No	Weight Structure	width	Height	Length	Total	Weight volume	Weight
		m	m	m	n	kN/m ²	kN
1	Slab	8	0,2	40,6	1	25	1624
2	Deck Slab	1,25	0,07	40,6	5	25	444,063
3	Trotoar			40,6	2	25	2030
4	Middle Girder, A =		2,497	36,4	6	25	13633,62
5	Edge Girder, A =		1,939	4,2	6	25	1221,633
6	Middle Diaphragm, A =		0,750	0,2	20	25	74,95
7	Edge Diaphragm, A =		1,316	0,2	10	25	65,800

Continuation Table 5.1 Weight of Bridge Upper Structure

8	backrest 1, A=	4	0,5	22	25	1100
9	Backrest 2, A=	6	3,6	10	25	5400
Total Self-Weight of The Upper Structure					W_{MS}	44056,063
Load On Abutments Due to Self-Weight					P_{MS}	22028,03

2. Lower Structure

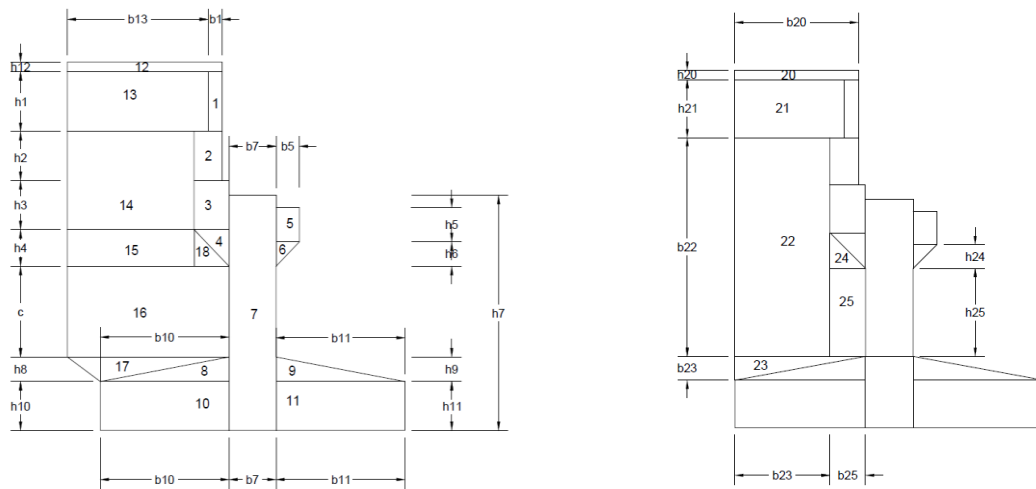


Figure 5.3 Lower Structure

Table 5.2 Weight of Bridge Lower Structure

No	Part Weight Parameters				Volume Weight	Weight
	b	h	Shape	Direct	kN/m ³	kN
1	0,3	1,22	1	-1	25	100,650
2	0,6	1	1	-1	25	165,000
3	0,75	1	1	-1	25	206,250
4	0,75	0,75	0,5	-1	25	77,344
5	0,7	0,5	1	1	25	96,250
6	0,5	0,5	0,5	1	25	34,375
7	1	4,8	1	1	25	1320,000
8	2,75	0,5	0,5	-1	25	189,063
9	2,75	0,5	0,5	1	25	189,063

Continuation Table 5.2 Weight of Bridge Lower Structure

10	2,75	1	1	-1	25	756,250
11	2,75	1	1	1	25	756,250
Wing Wall						
12	3,3	0,2	1	-1	25	181,500
13	3	1,22	1	-1	25	1006,500
14	2,7	2	1	-1	25	1485,000
15	2,7	0,75	1	-1	25	556,875
16	3,45	1,85	1	-1	25	1755,188
17	2,75	0,5	0,5	-1	25	189,063
18	0,75	0,75	0,5	-1	25	77,344
19	Stop Block		0,2	-1	25	2,200
Soil						
20	2,6	0,2	1	-1	25	143
21	1,22	2,3	1	-1	25	530,895
22	4,6	2	1	-1	25	1740,64
23	2,75	0,5	0,5	-1	25	130,075
24	0,75	0,75	0,5	-1	25	53,213
25	0,75	1,85	1	-1	25	262,515
						12004,5

So, the total weight of Self-weight is seen in the Table 5.3 below.

Table 5.3 Total Weight of Self-Weight Bridge

no	Description	P _{MS}
		kN
1	Upper Structure	12797,033
2	Lower Structure	12004,500
Total		24801,53295

5.2.2 Additional Dead Load (MA)

Additional dead load is the weight of non-structural elements on the bridge.
The calculation of the additional dead load can be seen in Table 5.4 below.

Table 5.4 Weight of Non-Structural Elements on The Bridge

No	Items	Height	Width	Length	Total	Weight Volume	Weight
		m	m	m	n	kN/m	kN
1	Lap. Aspal + Overley	0,1	8	40,6	1	22	714,56
2	Railings, Light, Dll		0,5	40,6	2	1	40,6
3	Instalasion ME		0,1	40,6	2	-	8,12
4	Rainwater	0,05	10	40,6	1	9,8	198,94
5	PJU		0,13		2	1,45	0,377
Total Additional Dead Weight on Abutment						W_{MA}	962,597
Load On Abutment Due to Additional Dead Load						P_{MA}	481,298

5.2.3 Soil Pressure (TA)

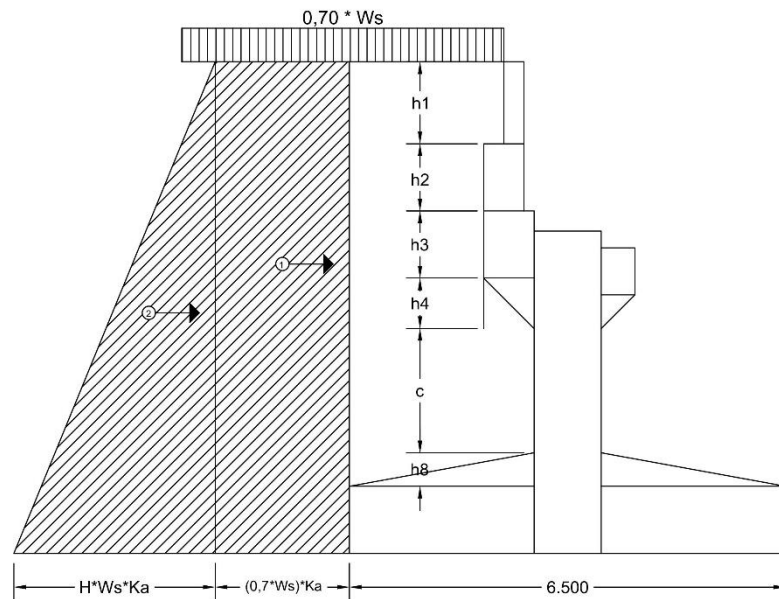


Figure 3.12 Soil Pressure (TA)

In the part of the ground behind the abutment wall that is loaded with traffic, an additional load equivalent to 0.60 m thick soil must be considered in the form of a uniform load equivalent to the vehicle load on that section. to calculate the soil pressure value can be done as follows:

$$\text{Soil weight, } w_s = 15,512 \text{ kN/m}^3$$

$$\text{Friction angel, } \phi = 35^\circ$$

$$\text{Cohesion, } c = 10,787 \text{ Kpa}$$

$$\text{Height of the abutment} = 4,8 \text{ m}$$

$$\text{Width of the abutment} = 11 \text{ m}$$

Distributed load due to the weight of 0.60 m high soil embankment which is equivalent to vehicle load:

$$0,60 \times w_s = 9,129 \text{ Kpa}$$

$$\begin{aligned} \phi' &= \tan^{-1}(k_\phi^R \times \tan \phi) \\ &= \tan^{-1}(0,7 \times \tan 35^\circ) = -0,211 \text{ rad} = 9,129 \end{aligned}$$

$$\begin{aligned} K_a &= \tan^2 \left(45^\circ - \frac{\phi'}{2} \right) \\ &= \tan^2 \left(45^\circ - \frac{9,129}{2} \right) = -12,135 \end{aligned}$$

Table 5.5 Soil Pressure Calculation

No	Force Due to Ground Pressure	T _{ta} (Kn)	Arm Due To O	y (M)	M _{ta} (Knm)
1	T _{TA} = (0,60 * w _s) * H * K _a * B _y	738,553	y = H / 2	2.4	1772,526
2	T _{TA} = 0,5 * H ² * w _s * K _a * B _y	2954,210	y = H / 3	1.6	4726,736
		3692,763			6499,262

5.2.4 Traffic Load (TD)

The value of the traffic load that occurs on the bridge is calculated as follows:

$$\text{Width of highway, } B_1 = 6,5 \text{ m}$$

$$\text{Length of highway in 1 span, } L = 40,6 \text{ m}$$

$$\begin{aligned} \text{Evenly distributed load, } q &= 8 \times \left(0,5 + \frac{15}{L} \right) \\ &= 8 \times \left(0,5 + \frac{15}{40,6} \right) \end{aligned}$$

$$= 6,596 \text{ kN/m}^2$$

$$\text{Pointed line load, } p = 49 \text{ kN/m}$$

$$\text{Dynamic load Allowance, DLA} = 0,4$$

Then the calculation of traffic load can be calculated as follows:

$$\begin{aligned} W_{TD} &= q \times L \times (5,5 + b) / 2 + p \times \text{DLA} \times (5,5 + b) / 2 \\ &= 6,596 \times 40,6 \times (5,5 + 6,5) / 2 + 2 \times 0,4 \times (5,5 + 6,5) / 2 \\ &= 1812 \text{ kN} \end{aligned}$$

$$\begin{aligned} \text{Traffic load staging reaction, } P_{TD} &= \frac{1}{2} \times W_{TD} \\ &= \frac{1}{2} \times 1812 \text{ kN} \\ &= 906 \text{ kN} \end{aligned}$$

5.2.5 Pedestrian Load (TP)

The value of the pedestrian load that occurs on the bridge is calculated as follows:

$$\begin{aligned} \text{Width of pavement, } b_2 &= 1 \text{ m} \\ \text{Length of pavement slab, } L &= 40,6 \text{ m} \\ \text{Total of pavement} &= 2 \\ \text{Area of sidewalk supported by abutment, } A &= b_2 \times L / 2 \times n \\ &= 1 \times \frac{40,6}{2} \times 2 \\ &= 40,6 \text{ m}^2 \\ \text{Uniform load on pedestrian, } q &= 5 - 0,33 \times (A - 10) \\ &= 5 - 0,33 \times (40,6 - 10) \\ &= 3,99 \text{ kPa} \\ \text{Load on abutment due to pedestrian, } P_{TP} &= A \times q \\ &= 40,6 \times 3,99 = 162 \text{ kN} \end{aligned}$$

5.2.6 Brake Load (TB)

The brake force is defined as a longitudinal force acting on the surface of the bridge floor. The amount of brake force applied to the bridge's longitudinal direction is determined by the overall length of the bridge span (L_t), as shown below.

1. 5 % of the designed truck load plus uniformly distributed lane load (UDL)

$$\text{Brake force} = 5\% \times (500 + (6,163 \times 8 \times 40,6))$$

$$= 125,08 \text{ kN}$$

2. 25 % of grand truck design weight

$$\text{Brake force} = 25\% \times 225$$

$$= 56,25 \text{ kN}$$

$$\text{Brake force used} = 125,080 \text{ kN}$$

Force arm towards the foundation,

$$Y_{TB} = \text{pile cap height} + \text{abutment height}$$

$$= 7,32 \text{ m}$$

Moment occurring due to brake force on foundation can be seen below.

$$M_{TB} = T_{TB} \times Y_{TB}$$

$$= 125,08 \times 7,32$$

$$= 915,92 \text{ kN}$$

5.2.7 Structure Wind Load (EW)

1. Wind load of the structure (E_{WT})

The structure's wind load would be determined by the wind load on the bridge's side plane as well as the wind load that would blow the vehicle. Wind loads on the bridge's side plane (T_{EW1}) are estimated below.

$$\text{Drag coefficient, } C_w = 1,5$$

$$A_b = \frac{L}{2} \times h_a$$

$$= \frac{40,6}{2} \times 4,31$$

$$= 87,493 \text{ m}^2$$

$$V_w = 30 \text{ m/s, because the location is more than 5km from the beach}$$

$$T_{EW1} = 0.0006 \times C_w \times (V_w^2) \times A_b$$

$$= 0.0006 \times 1.5 \times (30^2) \times 87,493$$

$$= 70,869 \text{ kN}$$

Wind load arms towards the foundation (Y_{EW1}) is 8,955 m. As a result, the wind load moment on the vehicle's side plane can be estimated as shown below.

$$M_{EW1} = T_{EW1} \times Y_{EW1}$$

$$= 70,869 \times 8,955$$

$$= 634,635 \text{ kNm}$$

Aside from blowing towards the side plane of the bridge, the wind also blows the vehicle driving through it (T_{EW2}). The computation of T_{EW2} is shown here, with a drag coefficient (C_w) of 1,2.

$$\begin{aligned} T_{EW2} &= 0.0006 \times C_w \times (V_w^2) \times \frac{L}{2} \\ &= 0.0006 \times 1.2 \times (30^2) \times \frac{40.6}{2} \\ &= 26,309 \text{ kN} \end{aligned}$$

With wind load arms (Y_{EW2}) of 7,2 meters. The following computation yields the instant of T_{EW2} .

$$\begin{aligned} M_{EW2} &= T_{EW2} \times Y_{EW2} \\ &= 26,309 \times 7,2 \\ &= 189,423 \text{ kNm} \end{aligned}$$

As a result, the total load and wind load moment operating on the bridge's side plane can be estimated as shown below.

$$\begin{aligned} T_{EWT} &= T_{EW1} + T_{EW2} \\ &= 70,869 + 26,309 \\ &= 97,178 \text{ kN} \end{aligned}$$

As for the moment of the wind load of the structure can be seen below.

$$\begin{aligned} M_{EWT} &= M_{EW1} + M_{EW2} \\ &= 634,635 + 189,423 \\ &= 824,058 \text{ kNm} \end{aligned}$$

2. Wind load of vehicle (EWL)

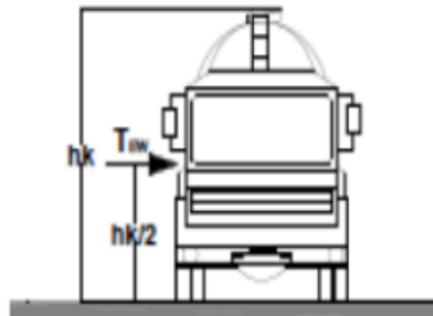


Figure 5.4 Wind Load Transfer to Vehicle

Firstly, the additional wind T_{EW} must be calculated below.

$$\begin{aligned} T_{EW} &= 0.0012 \times C_w \times (V_w^2) \\ &= 0.0012 \times 1.2 \times (30^2) \\ &= 1,296 \text{ kN} \end{aligned}$$

The vehicle load was determined by calculating the wind load transfer to the bridge floor. Where the vertical plane blown by the side wind of the vehicle height (h) is 2 m above the bridge floor, and the distance between vehicle wheels (x) is 1.75 m, the calculation of EWL is shown below.

$$\begin{aligned} P_{EW} &= 2 \times \left(\frac{1}{2} \times \frac{h}{x} \times T_{EW} \right) \times \frac{L}{2} \\ &= 2 \times \left(\frac{1}{2} \times \frac{2}{1.75} \times 1,296 \right) \times \frac{40.6}{2} \\ &= 30,067 \text{ kN} \end{aligned}$$

5.2.8 Earthquake Load (EQ)

1. Earthquake load (X-direction)

$$\text{Breast Wall Height, } L_b = 3,3 \text{ m}$$

$$\text{Breast Wall Cross-Section Size, } b=b_y = 11 \text{ m}$$

$$h=b/7 = 1 \text{ m}$$

$$\text{Breast wall cross-sectional inertia, } I_c = 0,917 \text{ m}^4$$

$$\text{Concrete quality, } f'_c = 30 \text{ Mpa}$$

$$\begin{aligned} \text{Elastic modulus of concrete, } E_c &= 4700 \times \sqrt{f'_c} \\ &= 4700 \times \sqrt{30} = 25742,960 \text{ Mpa} \\ &= 25742960,203 \text{ Kpa} \end{aligned}$$

$$\begin{aligned} \text{Stiffness value, } K_p &= 3 \times E_c \times I_c \times L_b^3 \\ &= 3 \times 25742960,203 \times 0,917 \times 3,3^3 \\ &= 1969923,493 \text{ kN/m} \end{aligned}$$

$$\text{Acceleration of gravity, } g = 9,81 \text{ m/s}^2$$

$$\text{Self-weight of Upper Structure, } P_{MS (upper)} = 12797,033 \text{ kN}$$

$$\text{Self-weight of the Lower Structure, } P_{MS (lower)} = 12004,500 \text{ kN}$$

$$\begin{aligned} \text{Total weight of the structure, } W_{TP} &= P_{MS (upper)} + \frac{P_{MS (lower)}}{2} \\ &= 12797,033 + \frac{12004,500}{2} \end{aligned}$$

$$= 18799,283 \text{ kN}$$

$$\begin{aligned} \text{Natural vibration time of the structure, } T &= 2 \times \pi \times \sqrt{\frac{W_{TP}}{g \times K_P}} \\ &= 2 \times \pi \times \sqrt{\frac{18799,283}{9,81 \times 1969923,493}} \\ &= 0,196 \text{ s} \end{aligned}$$

$$\text{Base shear coefficient, } C = 0,125$$

$$\begin{aligned} F &= 1,25 - 0,025 \times n \\ &= 1,25 - 0,025 \times 1 = 1,225 \end{aligned}$$

$$\begin{aligned} S &= 1 \times F \\ &= 1 \times 1,225 = 1,225 \end{aligned}$$

$$\begin{aligned} \text{Horizontal earthquake load coefficient, } K_h &= C \times S \\ &= 0,14 \times 1,225 = 0,1715 \end{aligned}$$

For bridges that carry > 2000 vehicles / day, bridges on major highways or arteries, and bridges where there are alternative routes, an importance factor is taken, $I = 1$

$$\begin{aligned} \text{Earthquake force, } T_{EQ} &= K_h \times I \times W_t \\ &= 0,1715 \times W_t \end{aligned}$$

Table 5.6 Earthquake Load In X-Direction

No	Weight W_t (kN)	T_{eq} (kN)	Length of Y (m)	M_{eq} (kNm)
Upper Structure				
Pms	12797,033	1959,546	7,520	14735,783
Pma	481,299	73,699	7,520	554,215
Abutment				
1	100,650	15,412	6,710	103,415
2	165,000	25,266	5,600	141,488
3	206,250	31,582	4,600	145,277
4	77,344	11,843	3,400	40,267
5	96,250	14,738	4,200	61,901
6	34,375	5,264	3,683	12,764
7	1320,000	202,125	2,425	490,153
8	189,063	28,950	1,167	33,776
9	189,063	28,950	1,167	33,776
10	756,250	115,801	0,500	57,900

Continuation Table 5.6 Earthquake Load In X-Direction

11	756,250	115,801	0,500	57,900
Wing Wall				
12	181,5	27,792	7,420	206,218
13	1006,5	154,120	6,710	1034,147
14	1485	227,391	5,100	1159,692
15	556,875	85,271	3,725	317,636
16	1755,1875	268,763	2,425	651,750
17	189,0625	28,950	1,333	38,599
18	77,34375	11,843	3,600	42,636
Soil				
20	143	21,897	7,420	162,475
21	530,8952	81,293	6,710	545,478
22	1740,64	266,536	4,160	1108,788
23	130,075	19,918	1,333	26,556
24	53,2125	8,148	3,600	29,333
25	262,515	40,198	2,245	90,244
20	143	21,897	7,420	162,475
		3871,097		21882,170

2. Earthquake load (Y-direction)

Breast wall cross-sectional inertia, $I_c = 0,917 \text{ m}^4$

Stiffness value, K_p

$$= 3 \times E_c \times I_c \times L_b^3$$

$$= 3 \times 25742960,203 \times 0,917 \times 3,3^3$$

$$= 1969923,493 \text{ kN/m}$$

Natural vibration time of the structure, T

$$= 2 \times \pi \times \sqrt{\frac{W_{TP}}{g \times K_p}}$$

$$= 2 \times \pi \times \sqrt{\frac{34321,781}{9,81 \times 1969923,493}}$$

$$= 0,265 \text{ s}$$

Total weight of the structure, W_{TP}

$$= P_{MS (upper)} + \frac{P_{MS (lower)}}{2}$$

$$= 12797,033 + \frac{12004,500}{2}$$

$$= 18799,283 \text{ kN}$$

Base shear coefficient, C

$$= 0,125$$

F

$$= 1,25 - 0,025 \times n$$

$$\begin{aligned}
&= 1,25 - 0,025 \times 1 = 1,225 \\
S &= 1 \times F \\
&= 1 \times 1,255 = 1,255 \\
\text{Horizontal earthquake load coefficient, } K_h &= C \times S \\
&= 0,14 \times 1,255 = 0,1715 \\
I &= 1 \\
\text{Earthquake force, } T_{EQ} &= K_h \times I \times W_t \\
&= 0,1715 \times W_t \\
\text{Self-weight of Upper Structure, } P_{MS \text{ (upper + lower)}} &= 24801,533 \text{ kN} \\
\text{Self-weight of the Lower Structure, } P_{MA} &= 481,298 \text{ kN} \\
\text{Dead Load, } W_t &= P_{MS} + P_{MA} \\
&= 24801,533 + 481,298 \\
&= 18799,283 \text{ kN} \\
\text{Earthquake load in the transverse direction of the bridge, } T_{EQ} &= k_h \times I \times W_t \\
&= 0,153 \times 1 \times \\
&18799,283 \\
&= 2878,640 \text{ kN} \\
\text{Moment on foundation due to earthquake load, } M_{EQ} &= T_{EQ} \times Y_{EQ} \\
&= 2878,640 \times 5,653 \\
&= 16272,106 \text{ kNm}
\end{aligned}$$

The following is a recapitulation of the results of the calculation of the load that occurs on the bridge which can be seen in Table 5.7 below.

Table 5.7 Recapitulation in Abutment 1

No	Weight	P (kN)	Tx (kN)	Ty (kN)	Mx (kNm)	My (kNm)
1	Self-Weight (Ms)	24801,533	-	-	-	-
2	Additional Dead Load (MA)	481,299	-	-	-	-
3	Soil Pressure (TA)	-	3815,855	-	6794,684	-
4	Traffic Load (TD)	906	-	-	-	-

Continuation Table 5.7 Recapitulation in Abutment 1

5	Pedestrian Load (TP)	-	125,080	-	915,586	-
6	Brake Load (TB)	162,002	-	-	-	-
7	Structure Wind Load (EWs)	-	-	97,178	-	824,058
8	Vehicle Wind Load (EWL)	30,067	-	-	-	-
9	Earthquake Load (EQ)	-	3871,097	2878,640	21882,170	16272,106

5.3 Bridge Loading Combinations

In this step, in accordance with sni 1725:2017, the analysis of the combination of loading on the construction of the Tawang-Ngalang bridge segment IV uses several loading combinations as in the Table 5.8 below.

Table 5.8 Bridge Load Combination

No	Load	Limit Condition					
		<i>Kuat I</i>	<i>Kuat II</i>	<i>Kuat III</i>	<i>Kuat IV</i>	<i>Kuat IV</i>	<i>Ekstrim I</i>
1	Self-Weight (Ms)	1,3	1,3	1,3	1,3	1,3	1,3
2	Additional Dead Load (MA)	2	2	2	2	2	2
3	Soil Pressure (TA)	1,4	1,4	1,4	1,4	1,4	1,4
4	Traffic Load (TD)	1,8	1,4	-	-	-	0,3
5	Pedestrian Load (TP)	1,8	1,4	-	-	-	0,3
6	Brake Load (TB)	1,8	1,4	-	-	-	0,3
7	Structure Wind Load (EWs)	-	-	1,4	-	0,4	-
8	Vehicle Wind Load (EWL)	-	-	-	-	1	-
9	Earthquake Load (EQ)	0	0	0	0	0	1

Table 5.9 Load Combination of *Kuat I*

Load Combination of <i>Kuat I</i>						
No	Weight	Limit Condition				
		P (kN)	Tx (kN)	Ty (kN)	Mx (kNm)	My (kNm)
1	Self-Weight (Ms)	32241,993	-	-	-	-

Continuation Table 5.9 Load Combination of Kuat I

2	Additional Dead Load (MA)	962,597	-	-	-	-
3	Soil Pressure (TA)	-	5342,197	-	9512,558	-
4	Traffic Load (TD)	1630,800	-	-	-	-
5	Pedestrian Load (TP)	-	225,144	-	1648,054	-
6	Brake Load (TB)	291,604	-	-	-	-
7	Structure Wind Load (EWs)	-	-	-	-	-
8	Vehicle Wind Load (EWL)	-	-	-	-	-
9	Earthquake Load (EQ)	-	-	-	-	-
		35126,994	5567,341	-	11160,612	-

Table 5.10 Load Combination of Kuat II

Load Combination of Kuat II						
No	Weight	Limit Condition				
		P (kN)	Tx (kN)	Ty (kN)	Mx (kNm)	My (kNm)
1	Self-Weight (Ms)	32241,993	-	-	-	-
2	Additional Dead Load (MA)	962,597	-	-	-	-
3	Soil Pressure (TA)	-	5342,197	-	9512,558	-
4	Traffic Load (TD)	1268,400	-	-	-	-
5	Pedestrian Load (TP)	-	175,112	-	1281,820	-
6	Brake Load (TB)	226,803	-	-	-	-
7	Structure Wind Load (EWs)	-	-	-	-	-
8	Vehicle Wind Load (EWL)	-	-	-	-	-
9	Earthquake Load (EQ)	-	-	-	-	-
		34699,793	5517,309	-	10794,377	-

Table 5.11 Load Combination of *Kuat III*

Load Combination of <i>Kuat III</i>						
No	Weight	Limit Condition				
		P (kN)	Tx (kN)	Ty (kN)	Mx (kNm)	My (kNm)
1	Self-Weight (Ms)	32241,993	-	-	-	-
2	Additional Dead Load (MA)	962,597	-	-	-	-
3	Soil Pressure (TA)	-	7479,076	-	13317,581	-
4	Traffic Load (TD)	-	-	-	-	-
5	Pedestrian Load (TP)	-	-	-	-	-
6	Brake Load (TB)	-	-	-	-	-
7	Structure Wind Load (EWs)	-	-	136,049	-	1153,681
8	Vehicle Wind Load (EWL)	-	-	-	-	-
9	Earthquake Load (EQ)	-	-	-	-	-
		33204,590	7479,076	136,049	13317,581	1153,681

Table 5.12 Load Combination of *Kuat IV*

Load Combination of <i>Kuat IV</i>						
No	Weight	Limit Condition				
		P (kN)	Tx (kN)	Ty (kN)	Mx (kNm)	My (kNm)
1	Self-Weight (Ms)	32241,993	-	-	-	-
2	Additional Dead Load (MA)	962,597	-	-	-	-
3	Soil Pressure (TA)	-	7479,076	-	9512,558	-
4	Traffic Load (TD)	-	-	-	-	-
5	Pedestrian Load (TP)	-	-	-	-	-
6	Brake Load (TB)	-	-	-	-	-
7	Structure Wind Load (EWs)	-	-	-	-	-

Continuation Table 5.12 Load Combination of *Kuat IV*

8	Vehicle Wind Load (EWL)	-	-	-	-	-
9	Earthquake Load (EQ)	-	-	-	-	-
		33204,590	7479,076	-	9512,558	-

Table 5.13 Load Combination of *Kuat V*

Load Combination of <i>Kuat V</i>						
No	Weight	Limit Condition				
		P (kN)	Tx (kN)	Ty (kN)	Mx (kNm)	My (kNm)
1	Self-Weight (Ms)	32241,993	-	-	-	-
2	Additional Dead Load (MA)	962,597	-	-	-	-
3	Soil Pressure (TA)	-	5342,197	-	9512,558	-
4	Traffic Load (TD)	-	-	-	-	-
5	Pedestrian Load (TP)	-	-	-	-	-
6	Brake Load (TB)	-	-	-	-	-
7	Structure Wind Load (EWs)	-	-	38,871	-	329,623
8	Vehicle Wind Load (EWL)	30,067	-	-	-	-
9	Earthquake Load (EQ)	-	-	-	-	-
		33234,657	5342,197	38,871	9512,558	329,623

Table 5.14 Load Combination of *Ekstrim I*

Load Combination of <i>Ekstrim I</i>						
No	Weight	Limit Condition				
		P (kN)	Tx (kN)	Ty (kN)	Mx (kNm)	My (kNm)
1	Self-Weight (Ms)	32241,993	-	-	-	-
2	Additional Dead Load (MA)	962,597	-	-	-	-

Continuation Table 5.14 Load Combination of *Ekstrim I*

3	Soil Pressure (TA)	-	7479,076	-	9512,558	-
4	Traffic Load (TD)	271,800	-	-	-	-
5	Pedestrian Load (TP)	-	37,524	-	274,676	-
6	Brake Load (TB)	48,601	-	-	-	-
7	Structure Wind Load (EWs)	-	-	-	-	-
8	Vehicle Wind Load (EWL)	-	-	-	-	-
9	Earthquake Load (EQ)	-	3871,097	2878,640	21882,170	16272,106
		33524,990	11387,697	2878,640	31669,403	16272,106

Table 5.15 Recapitulation Loading Combination

Recapitulation Loading Combination						
No	Weight	Limit Condition				
		P (kN)	Tx (kN)	Ty (kN)	Mx (kNm)	My (kNm)
1	<i>Kuat I</i>	35126,994	5567,341	-	11160,612	-
2	<i>Kuat II</i>	34699,793	5517,309	-	10794,377	-
3	<i>Kuat III</i>	33204,590	7479,076	136,049	13317,581	1153,681
4	<i>Kuat IV</i>	33204,590	-	-	-	-
5	<i>Kuat V</i>	33234,657	5342,197	38,871	9512,558	329,623
6	<i>Ekstrim I</i>	33524,990	11387,697	2878,640	31669,403	16272,106
Use		35126,994	11387,697	2878,640	31669,403	16272,106
Conversion kN to T		3512,699	1138,770	287,864	3166,940	1627,211

5.4 Roll Stability Control

5.4.1 Bridge Longitudinal Rolling Stability (X-Direction)

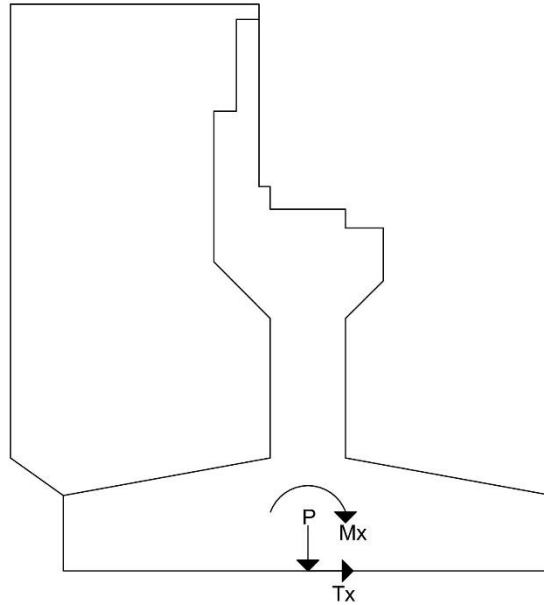


Figure 5.5 Bridge Longitudinal Rolling Stability X Direction

Location of rolling point A (end of foundation) to the center of the foundation,

$$\frac{B_x}{2} = \frac{6,5}{2} = 3,25 \text{ m}$$

The following are the formulas used to calculate the overturning resisting moment (M_p) and the safe number against overturning (SF):

$$M_p = P \times \left(\frac{B_x}{2}\right)$$
$$SF = \frac{M_p}{M_x} \geq 2,2$$

Description:

M_p = overturning resisting moment (knm),

P = load (kN),

B_x = the location of the point (A) of overturning towards the center of

the foundation (m), and

M_x = moment that causes overturning (kNm).

For example, the roll-retaining moment in *Kuat I*,

$$M_p = P \times \left(\frac{B_x}{2}\right)$$

$$\begin{aligned} M_p &= 35200,78 \times \left(\frac{6,5}{2}\right) \\ &= 114402,567 \end{aligned}$$

The moment that results in overturning,

$$M_x = 10747,022$$

Safe number against overturning,

$$\begin{aligned} SF &= \frac{M_p}{M_x} \geq 2,2 \\ &= \frac{114402,567}{10747,022} \geq 2,2 \\ &= 10,645 \geq 2,2 \end{aligned}$$

A recapitulation of the calculation of stability against rolling in the longitudinal direction of the bridge (X direction) is presented in Table 5.16 below.

Table 5.16 Bridge Longitudinal Rolling Stability (X-Direction)

No	Action/Load	P (kN)	M_x (kNm)	M_{px}	SF	Description
1	<i>Kuat I</i>	35126,994	11160,612	114162,7294	10,229	Safe
2	<i>Kuat II</i>	34699,793	10794,377	112774,3266	10,448	Safe
3	<i>Kuat III</i>	33204,590	13317,581	107914,917	8,103	Safe
4	<i>Kuat IV</i>	33204,590	-	107914,917	-	-
5	<i>Kuat V</i>	33234,657	9512,558	108012,6354	11,355	Safe
6	<i>Ekstrim I</i>	33524,990	31669,403	108956,219	3,440	Safe

5.4.2 Bridge Longitudinal Rolling Stability (Y-Direction)

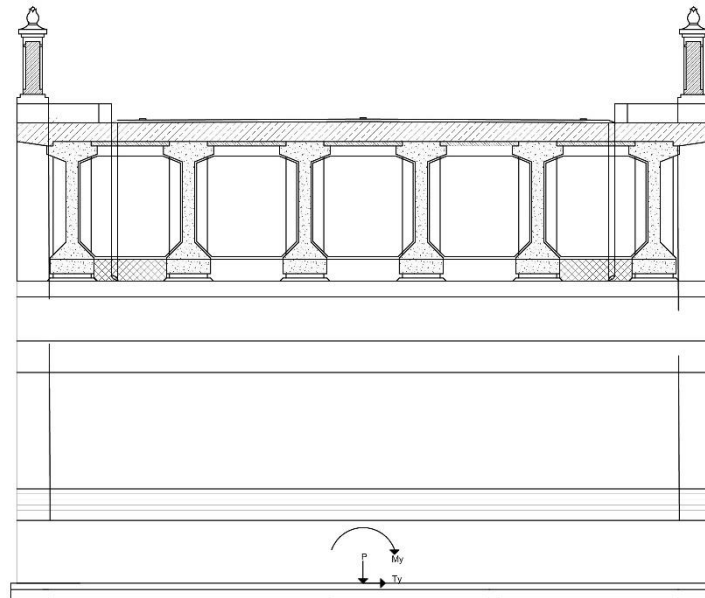


Figure 5.6 Bridge Longitudinal Rolling Stability Y Direction

Location of rolling point A (end of foundation) to the center of the foundation,

$$\frac{B_y}{2} = \frac{11}{2} = 5,5 \text{ m}$$

The following are the formulas used to calculate the overturning resisting moment (M_p) and the safe number against overturning (SF):

$$M_p = P \times \left(\frac{B_y}{2}\right)$$

$$SF = \frac{M_p}{M_y} \geq 2,2$$

Description:

M_p = overturning resisting moment (knm),

P = load (kN),

B_x = the location of the point (A) of overturning towards the center of the foundation (m), and

M_y = moment that causes overturning (kNm).

For example, the roll-retaining moment in *Kuat III*,

$$M_p = P \times \left(\frac{y}{2}\right)$$

$$\begin{aligned} M_p &= 33204,589 \times \left(\frac{5,5}{2}\right) \\ &= 182625,244 \end{aligned}$$

The moment that results in overturning,

$$M_y = 1153,681$$

Safe number against overturning,

$$\begin{aligned} SF &= \frac{M_p}{M_y} \geq 2,2 \\ &= \frac{182625,244}{1153,681} \geq 2,2 \\ &= 158,298 \geq 2,2 \end{aligned}$$

A recapitulation of the calculation of stability against rolling in the longitudinal direction of the bridge (Y direction) is presented in Table 5.17 below.

Table 5.17 Bridge Longitudinal Rolling Stability (Y-Direction)

No	Action/Load	P (kN)	M_y (kNm)	M_{py}	SF	Description
1	<i>Kuat I</i>	35126,9937	-	193198,465	-	-
2	<i>Kuat II</i>	34699,7928	-	190848,860		-
3	<i>Kuat III</i>	33204,5898	1153,681	182625,244	158,298	Safe
4	<i>Kuat IV</i>	33204,5898	-	182625,244	-	-

Continuation Table 5.18 Bridge Longitudinal Rolling Stability (Y-Direction)

5	<i>Kuat V</i>	33234,657	329,623	182790,614	554,544	Safe
6	<i>Ekstrim I</i>	33524,9905	16272,106	184387,448	11,332	Safe

5.5 Shear Stability Control

5.5.1 Bridge Longitudinal Shear Stability (X-Direction)

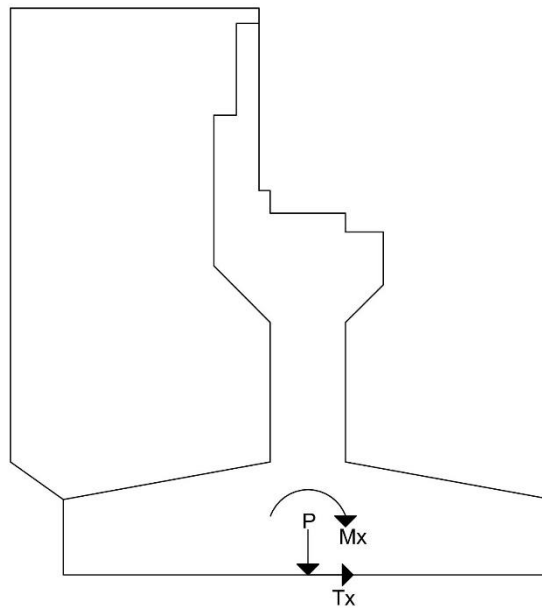


Figure 5.7 Bridge Longitudinal Shear Stability X Direction

Soil cohesion, c = 10,787 kPa

Soil friction angel, ϕ = 10,75°

Total load, P = 35200,789 kN

Shear resisting force in *Kuat I*,

$$\begin{aligned}
 H_p &= (c \times B_x \times B_y \times P \times \tan \phi) \\
 &= (10,787 \times 6,5 \times 11 \times 35200,9 \times \tan 10,72) \\
 &= 1890649,485 \text{ kN}
 \end{aligned}$$

The horizontal force acting,

$$H_x = 136,049 \text{ kN}$$

Safe numbers against shear,

$$\begin{aligned} SF &= \frac{H_P}{H_x} \geq 1,1 \\ &= \frac{1890649,485}{136,049} \geq 1,1 \\ &= 13108,716 \geq 1,1 \end{aligned}$$

The recapitulation of the calculation of shear stability in the longitudinal direction of the bridge (X direction) is presented in Table 5.18 below.

Table 5.19 Bridge Longitudinal Shear Stability Recapitulation

No	Action/Load	P (kN)	H _x (kNm)	M _{Px}	SF	Description
1	<i>Kuat I</i>	35126,994	5567,341	1886685,860	338,885	Safe
2	<i>Kuat II</i>	34699,793	5517,309	1863740,720	337,799	Safe
3	<i>Kuat III</i>	33204,590	7479,076	1783432,729	238,456	Safe
4	<i>Kuat IV</i>	33204,590	-	1783432,729	-	-
5	<i>Kuat V</i>	33234,657	5342,197	1785047,651	334,141	Safe
6	<i>Ekstrim I</i>	33524,990	11387,697	1800641,584	158,122	Safe

5.5.2 Bridge Longitudinal Shear Stability (Y-Direction)

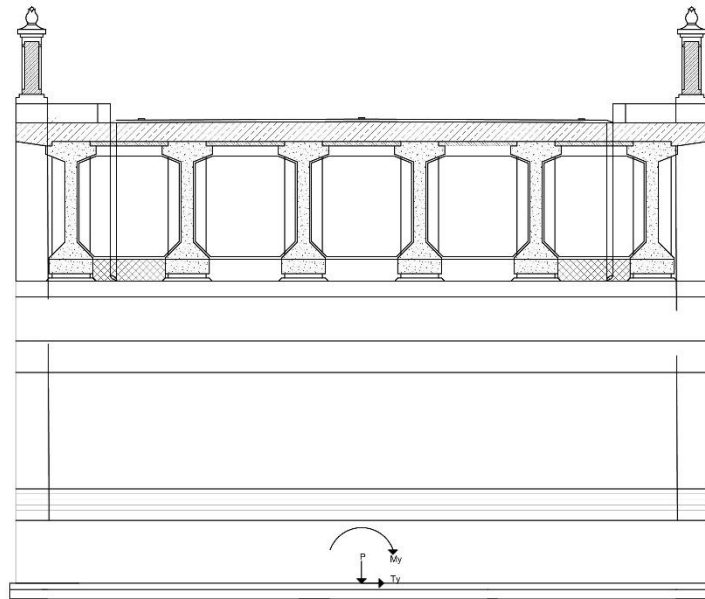


Figure 5.8 Bridge Longitudinal Shear Stability Y Direction

Soil cohesion, c = 10,787 kPa

Soil friction angel, ϕ = 10,75°

Total load, P = 35200,789 kN

Shear resisting force in *Kuat III*,

$$\begin{aligned}
 H_p &= (c \times B_x \times B_y \times P \times \tan \phi) \\
 &= (10,787 \times 6,5 \times 5,5 \times 35200,789 \times \tan 10,72) \\
 &= 1890649,485 \text{ kN}
 \end{aligned}$$

The horizontal force acting,

$$H_y = 5395,012 \text{ kN}$$

Safe numbers against shear,

$$\begin{aligned}
 SF &= \frac{H_p}{H_y} \geq 1,1 \\
 &= \frac{1890649,485}{5395,012} \geq 1,1
 \end{aligned}$$

$$= 350,444 \geq 1,1$$

The recapitulation of the calculation of shear stability in the longitudinal direction of the bridge (Y direction) is presented in Table 5.19 below.

Table 5.20 Bridge Longitudinal Shear Stability Recapitulation

No	Action/Load	P (kN)	H _y (kNm)	M _{Py}	SF	Description
1	<i>Kuat I</i>	35126,9937	-	1886685,860	-	-
2	<i>Kuat II</i>	34699,7928	-	1863740,720	-	-
3	<i>Kuat III</i>	33204,5898	136,049	1783432,729	13108,716	Safe
4	<i>Kuat IV</i>	33204,5898	-	1783432,729	-	-
5	<i>Kuat V</i>	33234,657	38,871	1785047,651	45922,052	Safe
6	<i>Ekstrim I</i>	33524,9905	2878,640	1800641,584	625,518	Safe

5.6 Bearing Capacity of Bored Pile Foundation

5.6.1 Analysis Based on Meyerhof method in Existing Conditions.

1. End bearing capacity

To calculate the average N, a depth of 4D below the end of the pile and 8D above the end of the pile was calculated as shown in Figure 5.9.

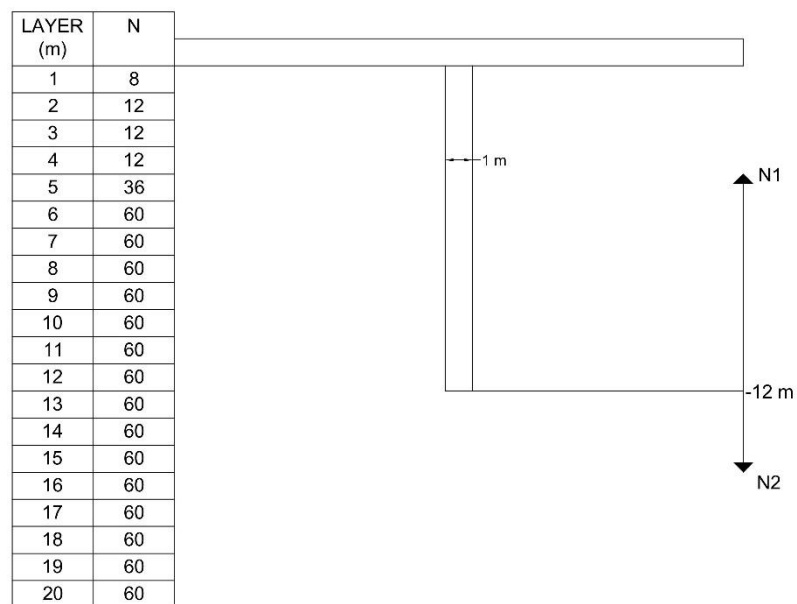


Figure 5.9 Location of N1 and N2 Meyerhof Method Diameter 1 m

$$N_b = 57,818$$

$$\begin{aligned} A_p &= 0,25 \times \pi \times D^2 \\ &= 0,25 \times \pi \times 1^2 \\ &= 0,785 \text{ m}^2 \end{aligned}$$

$$\begin{aligned} Q_p &= 20 \cdot N_b \cdot A_p \\ &= 20 \times 57,818 \times 0,785 \\ &= 908,2 \text{ Ton} \end{aligned}$$

2. Skin friction

a. For 1 - 3 m, the calculation is:

$$\begin{aligned} \text{N-SPT} &= \frac{8+12+12}{3} \\ &= 10,667 \\ P &= \pi \times D \\ &= 3,14 \times 1 \\ &= 3,14 \text{ m} \\ Q_s &= X_m \times N - \text{SPT} \times P \times Li \\ &= 0,2 \times 10,667 \times 3,14 \times 3 \\ &= 20,096 \text{ Ton} \end{aligned}$$

b. For 3 - 7 m, the calculation is:

$$\begin{aligned} \text{N-SPT} &= \frac{12+36+60+60}{4} \\ &= 42 \\ P &= \pi \times D \\ &= 3,14 \times 1 \\ &= 3,14 \text{ m} \\ Q_s &= X_m \times N - \text{SPT} \times P \times Li \\ &= 0,2 \times 42 \times 3,14 \times 4 \\ &= 105,504 \text{ Ton} \end{aligned}$$

c. For 8 m, the calculation is:

$$\begin{aligned} \text{N-SPT} &= 60 \\ P &= \pi \times D \end{aligned}$$

$$\begin{aligned}
&= 3,14 \times 1 \\
&= 3,14 \text{ m} \\
Q_s &= X_m \times N - SPT \times P \times Li \\
&= 0,2 \times 60 \times 3,14 \times 1 \\
&= 37,68 \text{ Ton}
\end{aligned}$$

d. For 9 - 12 m, the calculation is:

$$\begin{aligned}
N-SPT &= 60 \\
P &= \pi \times D \\
&= 3,14 \times 1 \\
&= 3,14 \text{ m} \\
Q_s &= X_m \times N - SPT \times P \times Li \\
&= 0,2 \times 60 \times 3,14 \times 4 \\
&= 150,72 \text{ Ton}
\end{aligned}$$

So, the skin friction is:

$$\begin{aligned}
Q_s &= 20,096 + 105,504 + 37,68 + 150,72 \\
&= 314 \text{ Ton}
\end{aligned}$$

3. Ultimate bearing capacity

$$\begin{aligned}
Q_u &= Q_b + Q_s \\
&= 908,2 + 314 \\
&= 1222,206 \text{ Ton}
\end{aligned}$$

4. Allowable bearing capacity

$$\begin{aligned}
Q_{all} &= \frac{Q_u}{SF} \\
&= \frac{1222,206}{2,5} \\
&= 488,88 \text{ Ton}
\end{aligned}$$

5.6.2 Analysis Based on Meyerhof Method in Dimension Variations 0,85 m.

1. End bearing capacity

To calculate the average N , a depth of $4D$ below the end of the pile and $8D$ above the end of the pile was calculated as shown in Figure 5.10.

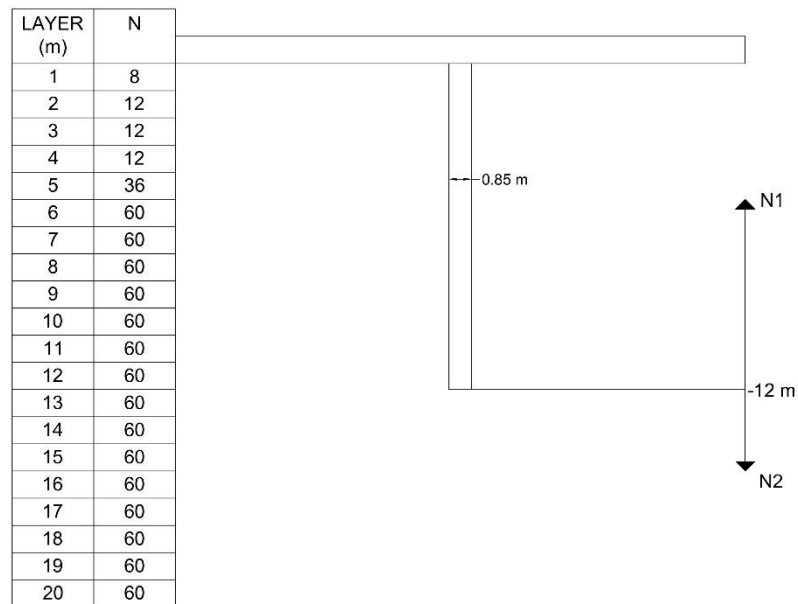


Figure 5.10 Location of N1 and N2 Meyerhof Method Diameter 0,85 m

$$N_b = 60$$

$$\begin{aligned}
 A_p &= 0,25 \times \pi \times D^2 \\
 &= 0,25 \times \pi \times 0,85^2 \\
 &= 0,567 \text{ m}^2
 \end{aligned}$$

$$\begin{aligned}
 Q_p &= 20 \cdot N_b \cdot A_p \\
 &= 20 \times 60 \times 0,567 \\
 &= 680,94 \text{ Ton}
 \end{aligned}$$

2. Skin friction:

a. For 1 - 3 m, the calculation is:

$$\begin{aligned}
 \text{N-SPT} &= \frac{8+12+12}{3} \\
 &= 10,667
 \end{aligned}$$

$$\begin{aligned}
 P &= \pi \times D \\
 &= 3,14 \times 0,85 \\
 &= 2,669 \text{ m}
 \end{aligned}$$

$$\begin{aligned}
 Q_s &= X_m \times N - \text{SPT} \times P \times Li \\
 &= 0,2 \times 10,667 \times 2,669 \times 3 \\
 &= 17,082 \text{ Ton}
 \end{aligned}$$

b. For 3 - 7 m, the calculation is:

$$N-SPT = \frac{12+36+60+60}{4}$$

$$= 42$$

$$P = \pi \times D$$

$$= 3,14 \times 0,85$$

$$= 2,669 \text{ m}$$

$$Q_s = X_m \times N - SPT \times P \times Li$$

$$= 0,2 \times 42 \times 2,669 \times 4$$

$$= 86,678 \text{ Ton}$$

c. For 8 m, the calculation is:

$$N-SPT = 60$$

$$P = \pi \times D$$

$$= 3,14 \times 0,85$$

$$= 2,669 \text{ m}$$

$$Q_s = X_m \times N - SPT \times P \times Li$$

$$= 0,2 \times 60 \times 2,669 \times 1$$

$$= 32,028 \text{ Ton}$$

d. For 9 - 12 m, the calculation is:

$$N-SPT = 60$$

$$P = \pi \times D$$

$$= 3,14 \times 0,85$$

$$= 2,669 \text{ m}$$

$$Q_s = X_m \times N - SPT \times P \times Li$$

$$= 0,2 \times 60 \times 2,669 \times 4$$

$$= 128,112 \text{ Ton}$$

So, the skin friction is:

$$Q_s = 17,082 + 89,678 + 32,028 + 128,112$$

$$= 266,9 \text{ Ton}$$

3. Ultimate bearing capacity

$$Q_u = Q_b + Q_s$$

$$= 680,94 + 266,9$$

$$= 947,84 \text{ Ton}$$

4. Allowable bearing capacity

$$Q_{all} = \frac{Q_u}{SF}$$

$$= \frac{947,84}{2,5}$$

$$= 379,136 \text{ Ton}$$

5.6.3 Analysis Based on Meyerhof Method in Dimension Variations 0,9 m.

1. End bearing capacity

To calculate the average N , a depth of $4D$ below the end of the pile and $8D$ above the end of the pile was calculated as shown in Figure 5.11.

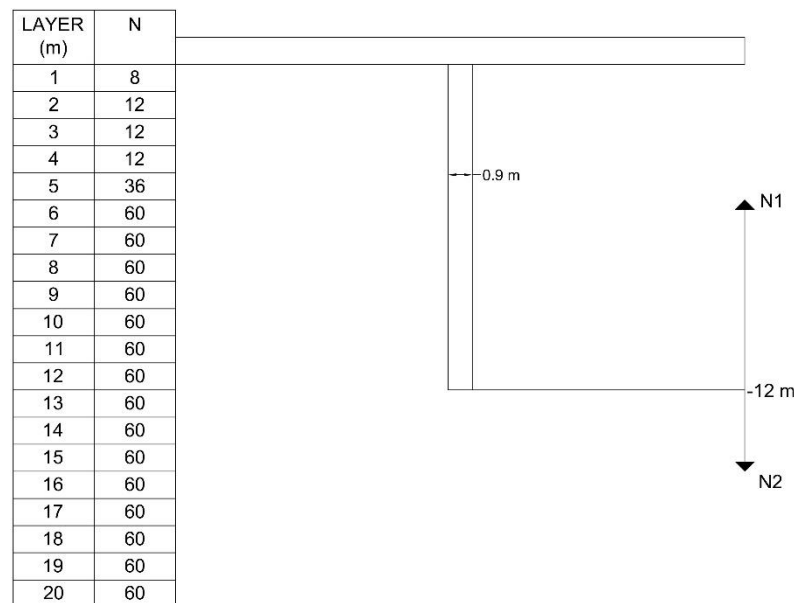


Figure 5.11 Location of N1 and N2 Meyerhof Method Diameter 0,9 m

$$N_b = 60$$

$$A_p = 0,25 \times \pi \times D^2$$

$$= 0,25 \times \pi \times 0,9^2$$

$$= 0,636 \text{ m}^2$$

$$Q_p = 20 \cdot N_b \cdot A_p$$

$$= 20 \times 60 \times 0,636$$

$$= 763,407 \text{ Ton}$$

2. Skin friction

- a. For 1 - 3 m, the calculation is:

$$N\text{-SPT} = \frac{8+12+12}{3}$$

$$= 10,667$$

$$P = \pi \times D$$

$$= 3,14 \times 0,9$$

$$= 2,826 \text{ m}$$

$$Q_s = X_m \times N - SPT \times P \times Li$$

$$= 0,2 \times 10,667 \times 2,826 \times 3$$

$$= 18,086 \text{ Ton}$$

- b. For 3 - 7 m, the calculation is:

$$N\text{-SPT} = \frac{12+36+60+60}{4}$$

$$= 42$$

$$P = \pi \times D$$

$$= 3,14 \times 0,9$$

$$= 2,669 \text{ m}$$

$$Q_s = X_m \times N - SPT \times P \times Li$$

$$= 0,2 \times 42 \times 2,826 \times 4$$

$$= 94,954 \text{ Ton}$$

- c. For 8 m, the calculation is:

$$N\text{-SPT} = 60$$

$$P = \pi \times D$$

$$= 3,14 \times 0,85$$

$$= 2,826 \text{ m}$$

$$Q_s = X_m \times N - SPT \times P \times Li$$

$$= 0,2 \times 60 \times 2,826 \times 1$$

$$= 33,192 \text{ Ton}$$

- d. For 9 - 12 m, the calculation is:

$$N\text{-SPT} = 60$$

$$\begin{aligned}
P &= \pi \times D \\
&= 3,14 \times 0,85 \\
&= 2,826 \text{ m} \\
Q_s &= X_m \times N - SPT \times P \times Li \\
&= 0,2 \times 60 \times 2,826 \times 4 \\
&= 135,648 \text{ Ton}
\end{aligned}$$

So, the skin friction is:

$$\begin{aligned}
Q_s &= 18,086 + 94,954 + 33,912 + 135,168 \\
&= 282,6 \text{ Ton}
\end{aligned}$$

5. Ultimate bearing capacity

$$\begin{aligned}
Q_u &= Q_b + Q_s \\
&= 763,407 + 282,6 \\
&= 1046 \text{ Ton}
\end{aligned}$$

6. Allowable bearing capacity

$$\begin{aligned}
Q_{all} &= \frac{Q_u}{SF} \\
&= \frac{1046}{2,5} \\
&= 418,403 \text{ Ton}
\end{aligned}$$

5.6.4 Analysis Based on Meyerhof Method in Dimension Variations 0,95 m.

1. End bearing capacity

To calculate the average N , a depth of $4D$ below the end of the pile and $8D$ above the end of the pile was calculated as shown in Figure 5.12.

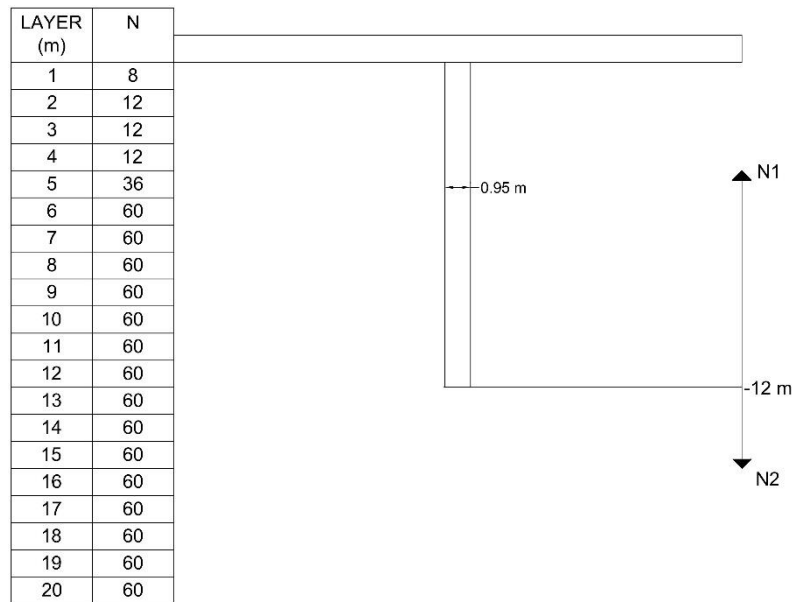


Figure 5.12 Location of N1 and N2 Meyerhof Method Diameter 0,95 m

$$N_b = 57,818$$

$$\begin{aligned}
 A_p &= 0,25 \times \pi \times D^2 \\
 &= 0,25 \times \pi \times 0,95^2 \\
 &= 0,709 \text{ m}^2
 \end{aligned}$$

$$\begin{aligned}
 Q_p &= 20 \cdot N_b \cdot A_p \\
 &= 20 \times 57,818 \times 0,708 \\
 &= 819,656 \text{ Ton}
 \end{aligned}$$

2. Skin friction

a. For 1 - 3 m, the calculation is:

$$\begin{aligned}
 \text{N-SPT} &= \frac{8+12+12}{3} \\
 &= 10,667
 \end{aligned}$$

$$\begin{aligned}
 P &= \pi \times D \\
 &= 3,14 \times 0,95 \\
 &= 2,983 \text{ m}
 \end{aligned}$$

$$\begin{aligned}
 Q_s &= X_m \times N - \text{SPT} \times P \times Li \\
 &= 0,2 \times 10,667 \times 2,983 \times 3 \\
 &= 19,091 \text{ Ton}
 \end{aligned}$$

b. For 3 - 7 m, the calculation is:

$$N-SPT = \frac{12+36+60+60}{4}$$

$$= 42$$

$$P = \pi \times D$$

$$= 3,14 \times 0,95$$

$$= 2,983 \text{ m}$$

$$Q_s = X_m \times N - SPT \times P \times Li$$

$$= 0,2 \times 42 \times 2,983 \times 4$$

$$= 94,954 \text{ Ton}$$

c. For 8 m, the calculation is:

$$N-SPT = 60$$

$$P = \pi \times D$$

$$= 3,14 \times 0,95$$

$$= 2,983 \text{ m}$$

$$Q_s = X_m \times N - SPT \times P \times Li$$

$$= 0,2 \times 60 \times 2,983 \times 1$$

$$= 35,796 \text{ Ton}$$

d. For 9 - 12 m, the calculation is:

$$N-SPT = 60$$

$$P = \pi \times D$$

$$= 3,14 \times 0,95$$

$$= 2,983 \text{ m}$$

$$Q_s = X_m \times N - SPT \times P \times Li$$

$$= 0,2 \times 60 \times 2,826 \times 4$$

$$= 143,184 \text{ Ton}$$

So, the skin friction is:

$$Q_s = 119,091 + 100,229 + 35,796 + 143,184$$

$$= 298,3 \text{ Ton}$$

3. Ultimate bearing capacity

$$Q_u = Q_b + Q_s$$

$$= 819,656 + 298,3$$

$$= 1117,956 \text{ Ton}$$

4. Allowable

$$Q_{all} = \frac{Q_u}{SF}$$

$$= \frac{1117,956}{2,5}$$

$$= 447,956 \text{ Ton}$$

5.6.5 Analysis Based on Reese and Wrigth method in Existing Conditions.

1. End bearing capacity

To calculate the average N, a depth of 4D below the end of the pile and 10D above the end of the pile was calculated as shown in Figure 5.13.

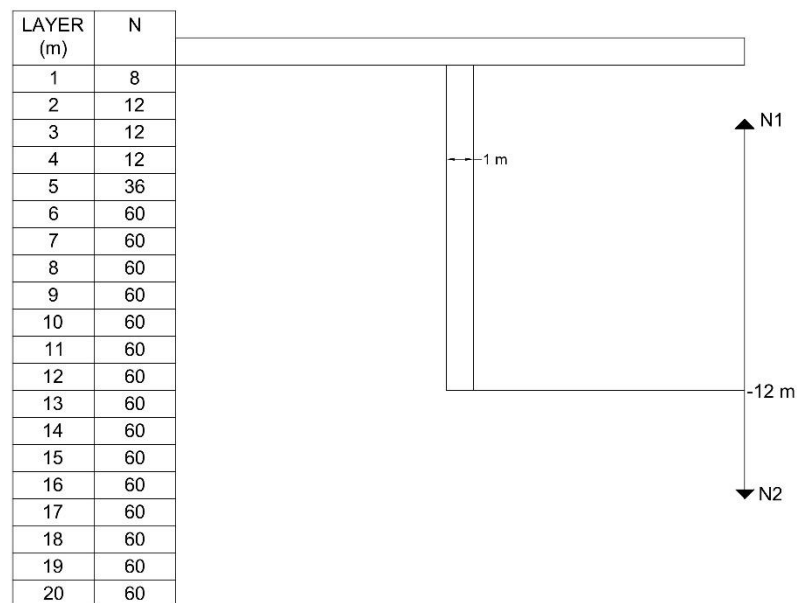


Figure 5.13 Location of N1 and N2 Reese and Wright Method Diameter 1 m

$$N-SPT = \frac{12+12+36+60+60+60+60+60+60+60+60+60+60+60}{14}$$

$$= 51,429$$

$$Cu = N - SPT \times \frac{2}{3} \times 10$$

$$= 51,429 \times \frac{2}{3} \times 10$$

$$= 342,86 \text{ kN/m}^2$$

$$\begin{aligned}
&= 34,286 \text{ T/m}^2 \\
q_p &= 9 \times cu \\
&= 9 \times 34,286 \\
&= 309 \text{ T/m} \\
A_p &= 0,25 \times \pi \times D^2 \\
&= 0,25 \times \pi \times 1^2 \\
&= 0,785 \text{ m}^2 \\
Q_b &= q_b \times A_p \\
&= 309 \times 0,785 \\
&= 242,351 \text{ Ton}
\end{aligned}$$

2. Skin friction

a. For 1 - 3 m, the calculation is:

$$\begin{aligned}
N\text{-SPT} &= \frac{8+12+12}{3} \\
&= 10,667 \\
cu &= \frac{2}{3} \times N - SPT \times 10 \\
&= \frac{2}{3} \times 10,667 \times 10 \\
&= 71,11 \text{ kN/m}^2 \\
&= 7,111 \text{ T/m}^2 \\
f &= \alpha \times cu \\
&= 0,55 \times 7,111 \\
&= 3,911 \text{ T/m}^2 \\
P &= \pi \times D \\
&= 3,14 \times 1 \\
&= 3,14 \text{ m} \\
Q_s &= f \times P \times Li \\
&= 3,991 \times 3,14 \times 3 \\
&= 36,843 \text{ Ton}
\end{aligned}$$

b. For 3 - 7 m, the calculation is:

$$N\text{-SPT} = \frac{12+36+60+60}{4}$$

$$\begin{aligned}
&= 42 \\
\text{cu} &= \frac{2}{3} \times N - SPT \times 10 \\
&= \frac{2}{3} \times 42 \times 10 \\
&= 280 \text{ kN/m}^2 \\
&= 28 \text{ T/m}^2 \\
f &= \alpha \times \text{cu} \\
&= 0,55 \times 28 \\
&= 15 \text{ T/m}^2 \\
P &= \pi \times D \\
&= 3,14 \times 1 \\
&= 3,14 \text{ m} \\
Q_s &= f \times P \times Li \\
&= 15 \times 3,14 \times 4 \\
&= 193,424 \text{ Ton}
\end{aligned}$$

c. For 8 m, the calculation is:

$$\begin{aligned}
\text{N-SPT} &= 60 \\
\text{cu} &= \frac{2}{3} \times N - SPT \times 10 \\
&= \frac{2}{3} \times 60 \times 10 \\
&= 400 \text{ kN/m}^2 \\
&= 40 \text{ T/m}^2 \\
f &= \alpha \times \text{cu} \\
&= 0,55 \times 40 \\
&= 22 \text{ T/m}^2 \\
P &= \pi \times D \\
&= 3,14 \times 1 \\
&= 3,14 \text{ m}^2 \\
Q_s &= f \times P \times Li \\
&= 22 \times 3,14 \times 1 \\
&= 69,626 \text{ Ton}
\end{aligned}$$

d. For 9 - 12 m, the calculation is:

$$N-SPT = 60$$

$$c_u = \frac{2}{3} \times N - SPT \times 10$$

$$= \frac{2}{3} \times 60 \times 10$$

$$= 400 \text{ kN/m}^2$$

$$= 40 \text{ T/m}^2$$

$$f = \alpha \times c_u$$

$$= 0,55 \times 40$$

$$= 22 \text{ T/m}^2$$

$$P = \pi \times D$$

$$= 3,14 \times 1$$

$$= 3,14 \text{ m}$$

$$Q_s = f \times P \times Li$$

$$= 22 \times 3,14 \times 4$$

$$= 276,32 \text{ Ton}$$

So, the skin friction is:

$$Q_s = 36,843 + 193,424 + 69,08 + 276,32$$

$$= 575,667 \text{ Ton}$$

5. Ultimate bearing capacity

$$Q_u = Q_b + Q_s$$

$$= 242,351 + 575,667$$

$$= 805,5 \text{ Ton}$$

6. Allowable

$$Q_{all} = \frac{Q_u}{SF}$$

$$= \frac{818}{2,5}$$

$$= 327,207 \text{ Ton}$$

5.6.6 Analysis Based on Reese and Wright Method in Dimension Variations 0,85 m.

1. End bearing capacity

To calculate the average N, a depth of 4D below the end of the pile and 10D above the end of the pile was calculated as shown in Figure 5.14.

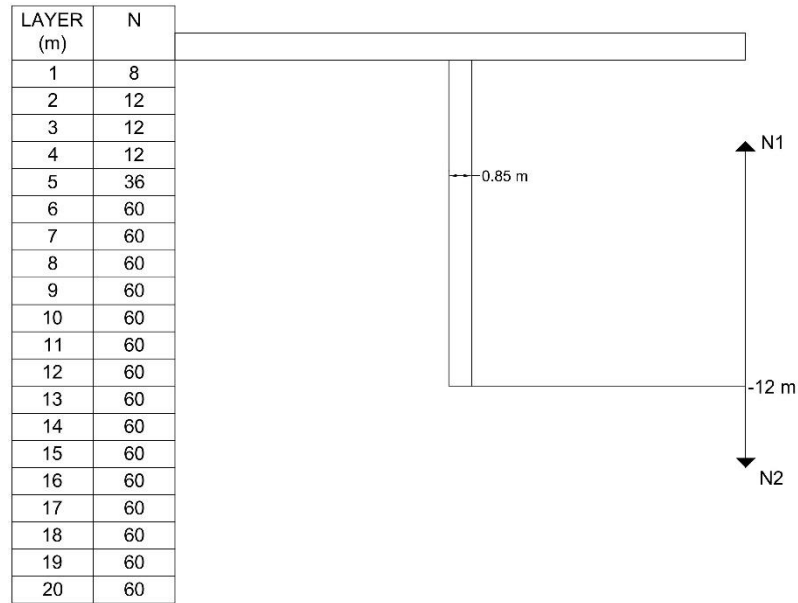


Figure 5.14 Location of N1 and N2 Reese and Wright Method Diameter 0,85 m

$$N-SPT = \frac{12+36+60+60+60+60+60+60+60+60+60+60+60}{12}$$

$$= 54,462$$

$$C_u = N - SPT \times \frac{2}{3} \times 10$$

$$= 54,462 \times \frac{2}{3} \times 10$$

$$= 360 \text{ kN/m}^2$$

$$= 36 \text{ T/m}^2$$

$$q_p = 9 \times cu$$

$$= 9 \times 36$$

$$= 324 \text{ T/m}$$

$$A_p = 0,25 \times \pi \times D^2$$

$$= 0,25 \times \pi \times 0,85^2$$

$$= 0,567 \text{ m}^2$$

$$Q_b = q_b \times A_p$$

$$= 324 \times 0,567$$

$$= 183,854 \text{ Ton}$$

2. Skin friction

a. For 1 - 3 m, the calculation is:

$$N\text{-SPT} = \frac{8+12+12}{3}$$

$$= 10,667$$

$$c_u = \frac{2}{3} \times N - SPT \times 10$$

$$= \frac{2}{3} \times 10,667 \times 10$$

$$= 71,11 \text{ kN/m}^2$$

$$= 7,111 \text{ T/m}^2$$

$$f = \alpha \times c_u$$

$$= 0,55 \times 7,111$$

$$= 3,911 \text{ T/m}^2$$

$$P = \pi \times D$$

$$= 3,14 \times 0,85$$

$$= 2,669 \text{ m}$$

$$Q_s = f \times P \times Li$$

$$= 3,911 \times 2,669 \times 3$$

$$= 31,316 \text{ Ton}$$

b. For 3 - 7 m, the calculation is:

$$N\text{-SPT} = \frac{12+36+60+60}{4}$$

$$= 42$$

$$c_u = \frac{2}{3} \times N - SPT \times 10$$

$$= \frac{2}{3} \times 42 \times 10$$

$$= 280 \text{ kN/m}^2$$

$$= 28 \text{ T/m}^2$$

$$f = \alpha \times c_u$$

$$= 0,55 \times 28$$

$$= 15,4 \text{ T/m}^2$$

$$\begin{aligned}
 P &= \pi \times D \\
 &= 3,14 \times 0,85 \\
 &= 2,669 \text{ m}
 \end{aligned}$$

$$\begin{aligned}
 Q_s &= f \times P \times Li \\
 &= 15,4 \times 2,669 \times 4 \\
 &= 164,41 \text{ Ton}
 \end{aligned}$$

c. For 8 m, the calculation is:

$$N\text{-SPT} = 60$$

$$\begin{aligned}
 Cu &= \frac{2}{3} \times N - SPT \times 10 \\
 &= \frac{2}{3} \times 60 \times 10 \\
 &= 400 \text{ kN/m}^2 \\
 &= 40 \text{ T/m}^2
 \end{aligned}$$

$$\begin{aligned}
 f &= \alpha \times cu \\
 &= 0,55 \times 40 \\
 &= 22 \text{ T/m}^2
 \end{aligned}$$

$$\begin{aligned}
 P &= \pi \times D \\
 &= 3,14 \times 0,85 \\
 &= 2,669 \text{ m}
 \end{aligned}$$

$$\begin{aligned}
 Q_s &= f \times P \times Li \\
 &= 22 \times 2,669 \times 1 \\
 &= 58,718 \text{ Ton}
 \end{aligned}$$

d. For 9 - 12 m, the calculation is:

$$N\text{-SPT} = 60$$

$$\begin{aligned}
 cu &= \frac{2}{3} \times N - SPT \times 10 \\
 &= \frac{2}{3} \times 60 \times 10 \\
 &= 400 \text{ kN/m}^2 \\
 &= 40 \text{ T/m}^2
 \end{aligned}$$

$$\begin{aligned}
 f &= \alpha \times cu \\
 &= 0,55 \times 40
 \end{aligned}$$

$$\begin{aligned}
&= 22 \text{ T/m}^2 \\
P &= \pi \times D \\
&= 3,14 \times 0,85 \\
&= 2,669 \text{ m} \\
Q_s &= f \times P \times Li \\
&= 22 \times 2,669 \times 4 \\
&= 234,872 \text{ Ton}
\end{aligned}$$

So, the skin friction is:

$$\begin{aligned}
Q_s &= 31,316 + 164,41 + 58,718 + 234,872 \\
&= 489,317 \text{ Ton}
\end{aligned}$$

a. Ultimate bearing capacity

$$\begin{aligned}
Q_u &= Q_b + Q_s \\
&= 183,854 + 489,317 \\
&= 673,171 \text{ Ton}
\end{aligned}$$

b. Allowable

$$\begin{aligned}
Q_{all} &= \frac{Q_u}{SF} \\
&= \frac{673,171}{2,5} \\
&= 269,268 \text{ Ton}
\end{aligned}$$

5.6.7 Analysis Based on Reese and Wright Method in Dimension Variations 0,9 m.

1. End bearing capacity

To calculate the average N , a depth of $4D$ below the end of the pile and $10D$ above the end of the pile was calculated as shown in Figure 5.15.

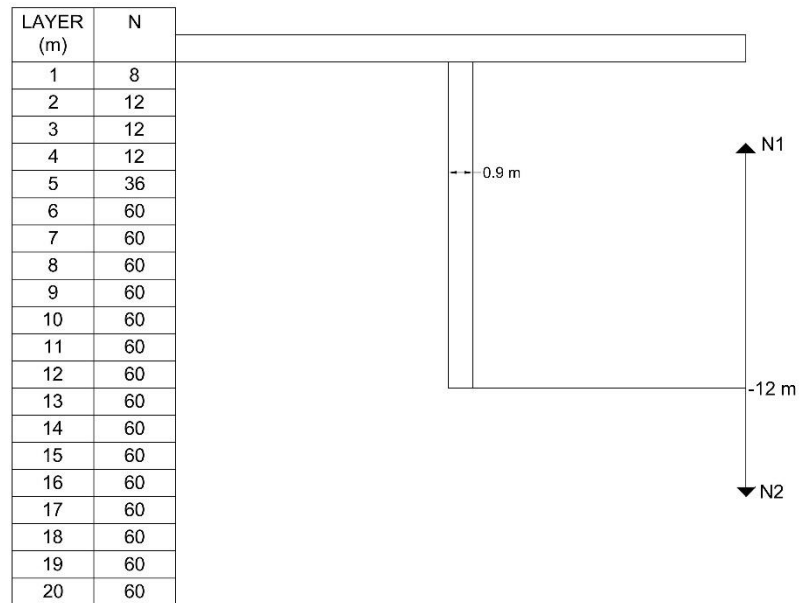


Figure 5.15 Location of N1 and N2 Reese and Wright Method Diameter 0,9 m

$$N-SPT = \frac{12+36+60+60+60+60+60+60+60+60+60+60+60}{13}$$

$$= 54,462$$

$$C_u = N - SPT \times \frac{2}{3} \times 10$$

$$= 54,462 \times \frac{2}{3} \times 10$$

$$= 363,308 \text{ kN/m}^2$$

$$= 36,3308 \text{ T/m}^2$$

$$q_p = 9 \times cu$$

$$= 9 \times 36,3308$$

$$= 327 \text{ T/m}^2$$

$$A_p = 0,25 \times \pi \times D^2$$

$$= 0,25 \times \pi \times 0,9^2$$

$$= 0,636 \text{ m}$$

$$Q_b = q_b \times A_p$$

$$= 327 \times 0,636$$

$$= 207,776 \text{ Ton}$$

2. Skin friction

e. For 1 - 3 m, the calculation is:

$$N-SPT = \frac{8+12+12}{3}$$

$$= 10,667$$

$$cu = \frac{2}{3} \times 10,667 \times 10$$

$$= \frac{2}{3} \times 10,667 \times 10$$

$$= 71,11 \text{ kN/m}^2$$

$$= 7,111 \text{ T/m}^2$$

$$f = \alpha \times cu$$

$$= 0,55 \times 7,111$$

$$= 3,911 \text{ T/m}^2$$

$$P = \pi \times D$$

$$= 3,14 \times 0,9$$

$$= 2,826 \text{ m}$$

$$Q_s = f \times P \times Li$$

$$= 3,911 \times 2,826 \times 3$$

$$= 33,158 \text{ Ton}$$

f. For 3 - 7 m, the calculation is:

$$N-SPT = \frac{12+36+60+60}{4}$$

$$= 42$$

$$cu = \frac{2}{3} \times N - SPT \times 10$$

$$= \frac{2}{3} \times 42 \times 10$$

$$= 280 \text{ kN/m}^2$$

$$= 28 \text{ T/m}^2$$

$$f = \alpha \times cu$$

$$= 0,55 \times 28$$

$$= 15,4 \text{ T/m}^2$$

$$P = \pi \times D$$

$$= 3,14 \times 0,9$$

$$= 2,826 \text{ m}$$

$$\begin{aligned}
Q_s &= f \times P \times Li \\
&= 15,4 \times 2,826 \times 4 \\
&= 174,082 \text{ Ton}
\end{aligned}$$

g. For 8 m, the calculation is:

$$N\text{-SPT} = 60$$

$$\begin{aligned}
cu &= \frac{2}{3} \times N - SPT \times 10 \\
&= \frac{2}{3} \times 60 \times 10 \\
&= 400 \text{ kN/m}^2 \\
&= 40 \text{ T/m}^2
\end{aligned}$$

$$\begin{aligned}
f &= \alpha \times cu \\
&= 0,55 \times 40 \\
&= 22 \text{ T/m}^2
\end{aligned}$$

$$\begin{aligned}
P &= \pi \times D \\
&= 3,14 \times 0,9 \\
&= 2,826 \text{ m}
\end{aligned}$$

$$\begin{aligned}
Q_s &= f \times P \times Li \\
&= 22 \times 2,826 \times 1 \\
&= 62,172 \text{ Ton}
\end{aligned}$$

h. For 9 - 12 m, the calculation is:

$$N\text{-SPT} = 60$$

$$\begin{aligned}
cu &= \frac{2}{3} \times N - SPT \times 10 \\
&= \frac{2}{3} \times 60 \times 10 \\
&= 400 \text{ kN/m}^2 \\
&= 40 \text{ T/m}^2
\end{aligned}$$

$$\begin{aligned}
f &= \alpha \times cu \\
&= 0,55 \times 40 \\
&= 22 \text{ T/m}^2
\end{aligned}$$

$$\begin{aligned}
P &= \pi \times D \\
&= 3,14 \times 0,9
\end{aligned}$$

$$\begin{aligned}
&= 2,826 \text{ m} \\
Q_s &= f \times P \times Li \\
&= 22 \times 2,826 \times 4 \\
&= 248,688 \text{ Ton}
\end{aligned}$$

So, the skin friction is:

$$\begin{aligned}
Q_s &= 33,158 + 174,082 + 62,172 + 248,688 \\
&= 518,1 \text{ Ton}
\end{aligned}$$

3. Ultimate bearing capacity

$$\begin{aligned}
Q_u &= Q_b + Q_s \\
&= 207,776 + 518,1 \\
&= 725,876 \text{ Ton}
\end{aligned}$$

4. Allowable

$$\begin{aligned}
Q_{all} &= \frac{Q_u}{SF} \\
&= \frac{725,876}{2,5} \\
&= 290,350 \text{ Ton}
\end{aligned}$$

5.6.8 Analysis Based on Reese and Wrigth Method in Dimension Variations 0,95 m.

1. End bearing capacity

To calculate the average N , a depth of $4D$ below the end of the pile and $10D$ above the end of the pile was calculated as shown in Figure 5.16.

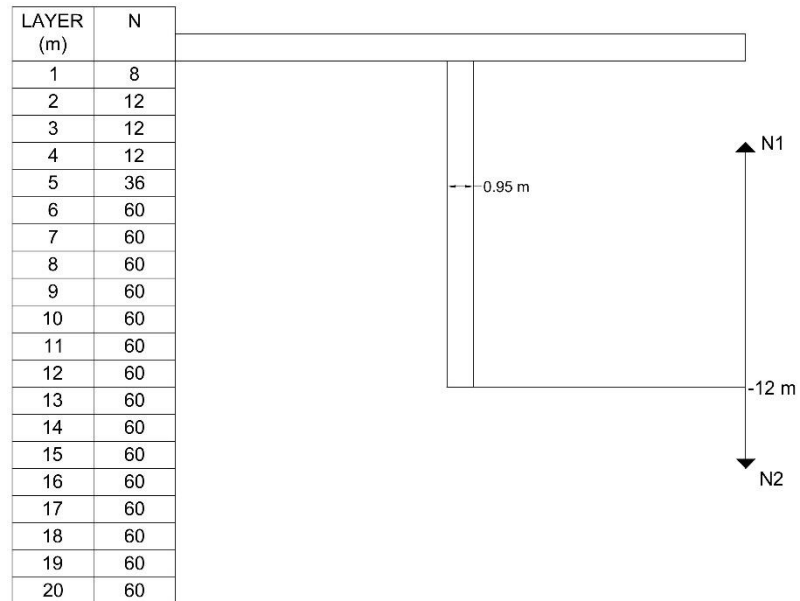


Figure 5.16 Location of N1 and N2 Reese and Wright Method Diameter 0,95 m

$$N-SPT = \frac{12+12+36+60+60+60+60+60+60+60+60+60+60+60}{14}$$

$$= 51,429$$

$$C_u = N - SPT \times \frac{2}{3} \times 10$$

$$= 48,8 \times \frac{2}{3} \times 10$$

$$= 342,86 \text{ kN/m}^2$$

$$= 34,286 \text{ T/m}^2$$

$$q_p = 9 \times cu$$

$$= 9 \times 32,533$$

$$= 293 \text{ T/m}^2$$

$$A_p = 0,25 \times \pi \times D^2$$

$$= 0,25 \times \pi \times 0,95^2$$

$$= 0,708 \text{ m}$$

$$Q_b = q_b \times A_p$$

$$= 34,286 \times 0,708$$

$$= 218,611 \text{ Ton}$$

2. Skin friction

a. For 1 - 3 m, the calculation is:

$$N-SPT = \frac{8+12+12}{3}$$

$$= 10,667$$

$$cu = \frac{2}{3} \times N - SPT \times 10$$

$$= \frac{2}{3} \times 10,667 \times 10$$

$$= 71,11 \text{ kN/m}^2$$

$$= 7,111 \text{ T/m}^2$$

$$f = \alpha \times cu$$

$$= 0,55 \times 7,111$$

$$= 3,911 \text{ T/m}^2$$

$$P = \pi \times D$$

$$= 3,14 \times 0,95$$

$$= 2,983 \text{ m}$$

$$Q_s = f \times P \times Li$$

$$= 3,911 \times 2,983 \times 3$$

$$= 35 \text{ Ton}$$

b. For 3 - 7 m, the calculation is:

$$N-SPT = \frac{12+36+60+60}{4}$$

$$= 42$$

$$cu = \frac{2}{3} \times N - SPT \times 10$$

$$= \frac{2}{3} \times 42 \times 10$$

$$= 280 \text{ kN/m}^2$$

$$= 28 \text{ T/m}^2$$

$$f = \alpha \times cu$$

$$= 0,55 \times 28$$

$$= 15,4 \text{ T/m}^2$$

$$P = \pi \times D$$

$$= 3,14 \times 0,95$$

$$= 2,893 \text{ m}^2$$

$$\begin{aligned}
Q_s &= f \times P \times Li \\
&= 15,4 \times 2,893 \times 4 \\
&= 183,753 \text{ T}
\end{aligned}$$

c. For 8 m, the calculation is:

$$N\text{-SPT} = 60$$

$$\begin{aligned}
cu &= \frac{2}{3} \times N - SPT \times 10 \\
&= \frac{2}{3} \times 60 \times 10 \\
&= 400 \text{ kN/m}^2 \\
&= 40 \text{ T/m}^2
\end{aligned}$$

$$\begin{aligned}
f &= \alpha \times cu \\
&= 0,55 \times 40 \\
&= 22 \text{ T/m}^2
\end{aligned}$$

$$\begin{aligned}
P &= \pi \times D \\
&= 3,14 \times 0,95 \\
&= 2,893 \text{ m}^2
\end{aligned}$$

$$\begin{aligned}
Q_s &= f \times P \times Li \\
&= 22 \times 2,893 \times 1 \\
&= 65,626 \text{ T}
\end{aligned}$$

d. For 9 - 12 m, the calculation is:

$$N\text{-SPT} = 60$$

$$\begin{aligned}
cu &= \frac{2}{3} \times N - SPT \times 10 \\
&= \frac{2}{3} \times 60 \times 10 \\
&= 400 \text{ kN/m}^2 \\
&= 40 \text{ T/m}^2
\end{aligned}$$

$$\begin{aligned}
f &= \alpha \times cu \\
&= 0,55 \times 40 \\
&= 22 \text{ T/m}^2
\end{aligned}$$

$$\begin{aligned}
P &= \pi \times D \\
&= 3,14 \times 0,95
\end{aligned}$$

$$\begin{aligned}
&= 2,893 \text{ m}^2 \\
Q_s &= f \times P \times Li \\
&= 22 \times 2,893 \times 4 \\
&= 262,504 \text{ T}
\end{aligned}$$

So, the skin friction is:

$$\begin{aligned}
Q_s &= 35 + 183,753 + 65,626 + 262,504 \\
&= 546,883 \text{ T}
\end{aligned}$$

3. Ultimate bearing capacity

$$\begin{aligned}
Q_u &= Q_b + Q_s \\
&= 218,611 + 546,883 \\
&= 765,495 \text{ T}
\end{aligned}$$

4. Allowable

$$\begin{aligned}
Q_{all} &= \frac{Q_u}{SF} \\
&= \frac{765,495}{2,5} \\
&= 306,198 \text{ T}
\end{aligned}$$

5.7 Bored Pile Foundation Requirement

5.7.1 Requirement and Control of Number Piles of Meyerhof Method in Existing Condition

1. Calculating number of piles.

$$\sum P_{max} = 35126,994 \text{ kN} = 3512,699 \text{ Ton}$$

$$Q_{all} = 488,88 \text{ Ton}$$

$$\begin{aligned}
n &= \frac{\sum P_{max}}{Q_{all}} \\
&= \frac{3512,699}{488,88} = 7,185 \approx 12 \text{ Pile}
\end{aligned}$$

2. Group Pile Arrangement Plan

Known based on the following figure 5.17 below.

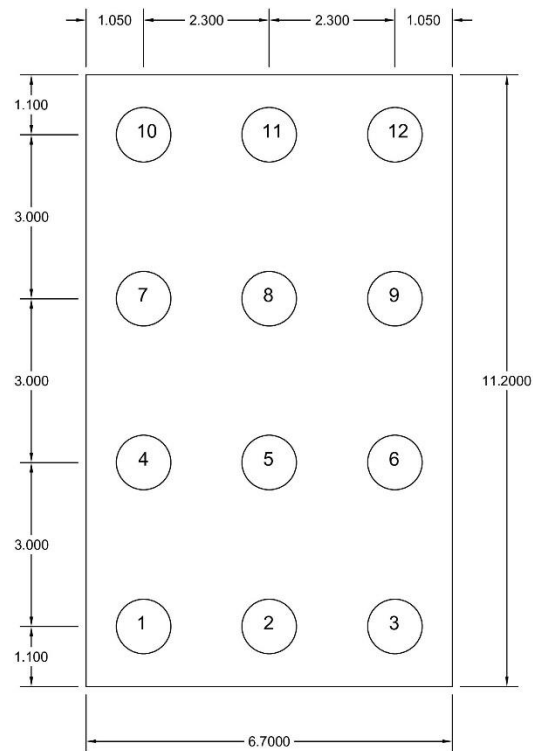


Figure 5.17 Foundation Arrangement Plan of Meyerhof Method with 1m Dimension

Known based on the following Figure 5.17.

Distance pile to pile, $s = 3$ Meter

Distance pile to edge, $a = 1,1$ Meter

Width, $B_x = 6,7$ Meter

Length, $B_y = 11,2$ Meter

3. Control of the forces that occur.

The computation and control of each foundation pile plan is shown in Table 5.20 Recapitulation.

Table 5.21 Control of Each Foundation Pile Plan Meyerhof Method With 1m Dimension

Pile	x_i m	y_i m	X^2 m^2	Y^2 m^2	$\sum My.x_i / \sum x^2$ T	$\sum Mx.y_i / \sum y^2$ T	P T	Check $P < Q_{all}$
1	-2,3	-4,5	5,290	20,250	-88,435	-105,565	98,725	Safe
2	0	-4,5	0,000	20,250	0,000	-105,565	187,160	Safe

**Continuation Table 5.20 Control of Each Foundation Pile Plan Meyerhof
Method With 1m Dimension**

3	2,3	-4,5	5,290	20,250	88,435	-105,565	275,596	Safe
4	-2,3	-1,5	5,290	2,250	-88,435	-35,188	169,101	Safe
5	0	-1,5	0,000	2,250	0,000	-35,188	257,537	Safe
6	2,3	-1,5	5,290	2,250	88,435	-35,188	345,972	Safe
7	-2,3	1,5	5,290	2,250	-88,435	35,188	239,478	Safe
8	0	1,5	0,000	2,250	0,000	35,188	327,913	Safe
9	2,3	1,5	5,290	2,250	88,435	35,188	416,349	Safe
10	-2,3	4,5	5,290	20,250	-88,435	105,565	309,854	Safe
11	0	4,5	0,000	20,250	0,000	105,565	398,290	Safe
12	2,3	4,5	5,290	20,250	88,435	105,565	486,725	Safe
			42,320	135,000			3512,699	

4. Pile Efficiency

$$m = 3$$

$$n = 4$$

$$s = 2,3 \text{ meter}$$

$$\theta = \arctg \left(\frac{D}{s} \right)$$

$$= \arctg \left(\frac{1}{3} \right)$$

$$= 0,322$$

$$\begin{aligned} E_g &= 1 - \theta \frac{(n-1) \times m + (m-1) \times n}{90 \times m \times n} \\ &= 1 - 0,32 \frac{(4-1) \times 3 + (3-1) \times 4}{90 \times 3 \times 4} \\ &= 0,995 \end{aligned}$$

5. Group Pile Support Capacity

$$n \text{ pile} = 12$$

$$\begin{aligned} Q_g &= E_g \times n \times Q_{all} \\ &= 0,995 \times 12 \times 488,88 \\ &= 5836,876 \text{ Ton} \end{aligned}$$

5.7.2 Requirement and Control of Number Piles of Meyerhof Method in Dimension Variations 0,85 m.

1. Calculating number of piles

$$\sum P_{\max} = 35126,994 \text{ kN} = 3512,699 \text{ Ton}$$

$$Q_{\text{all}} = 379,136 \text{ Ton}$$

$$n = \frac{\sum P_{\max}}{Q_{\text{all}}} = \frac{3512,699}{379,136} = 9,265 \approx 10 \text{ Pile}$$

2. Group Pile Arrangement Plan

Known based on the following figure 5.18 below.

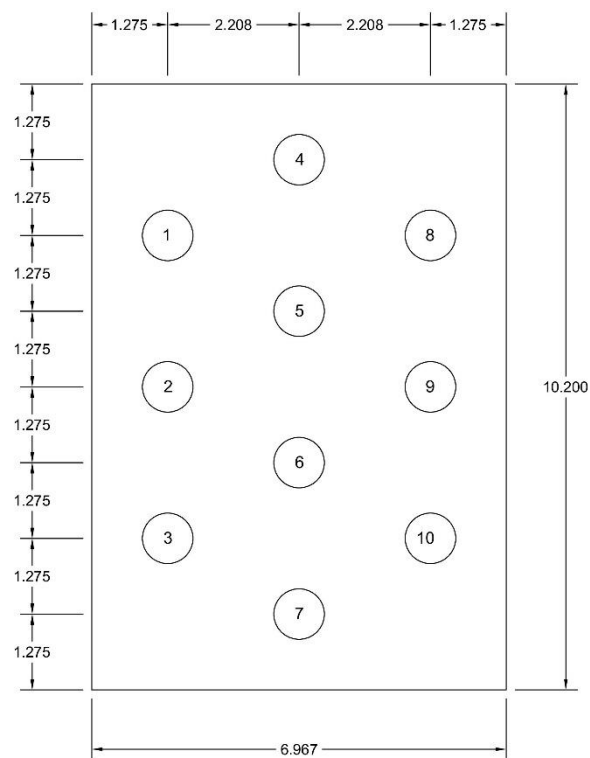


Figure 5.18 Foundation Arrangement Plan of Meyerhof Method with 0,85m Dimension

Known based on the following Figure 5.18.

Distance pile to pile, $s = 2,55$ Meter

Distance pile to edge, $a = 1,275$ Meter

Width, Bx = 6,7 Meter

Length, By = 10,2 Meter

3. Control of the forces that occur

The computation and control of each foundation pile plan is shown in Table 5.21 Recapitulation.

Table 5.22 Control of Each Foundation Pile Plan Meyerhof Method with 0,85m Dimension

Pile	xi	yi	X ²	Y ²	$\sum My.xi/\sum x^2$	$\sum Mx.yi/\sum y^2$	P	Check
	m	m	m ²	m ²	kN	kN	kN	P < Q _{all}
1	-2,208	2,55	4,877	6,503	-122,805	137,993	366,458	Safe
2	-2,208	0	4,877	0,000	-122,805	0,000	228,465	Safe
3	-2,208	-2,55	4,877	6,503	-122,805	-137,993	90,472	Safe
4	0,000	3,825	0,000	14,631	0,000	206,990	558,260	Not Safe
5	0,000	1,275	0,000	1,626	0,000	68,997	420,266	Not Safe
6	0,000	-1,275	0,000	1,626	0,000	-68,997	282,273	Safe
7	0,000	-3,825	0,000	14,631	0,000	-206,990	144,280	Safe
8	2,208	2,55	4,877	6,503	122,805	137,993	612,068	Not Safe
9	2,208	0	4,877	0,000	122,805	0,000	474,075	Not Safe
10	2,208	-2,55	4,877	6,503	122,805	-137,993	336,082	Safe
			29,262	58,523			3512,699	

4. Pile Efficiency

m = 3,333

n = 2,5

s = 2,55 meter

$\theta = \arctg\left(\frac{D}{s}\right)$
 $= \arctg\left(\frac{0,85}{2,55}\right)$
 $= 0,322$

$$\begin{aligned}
E_g &= 1 - \theta \frac{(n-1) \times m + (m-1) \times n}{90 \times m \times n} \\
&= 1 - 0,322 \frac{(2,5-1) \times 3,333 + (3,333-1) \times 2,5}{90 \times 3,333 \times 2,5} \\
&= 0,995
\end{aligned}$$

5. Group Pile Support Capacity

$$n_{\text{pile}} = 15$$

$$\begin{aligned}
Q_g &= E_g \times n \times Q_{\text{all}} \\
&= 0,995 \times 10 \times 379,136 \\
&= 3773,741 \text{ Ton}
\end{aligned}$$

5.7.3 Requirement and Control of Number Piles of Meyerhof Method in Dimension Variations 0,9 m.

1. Calculating number of piles

$$\sum P_{\text{max}} = 35126,994 \text{ kN} = 3512,699 \text{ Ton}$$

$$Q_{\text{all}} = 418,403 \text{ Ton}$$

$$\begin{aligned}
n &= \frac{\sum P_{\text{max}}}{Q_{\text{all}}} \\
&= \frac{3512,699}{418,403} = 8,395 \approx 9 \text{ Pile}
\end{aligned}$$

2. Group Pile Arrangement Plan

Known based on the following figure 5.19 below.

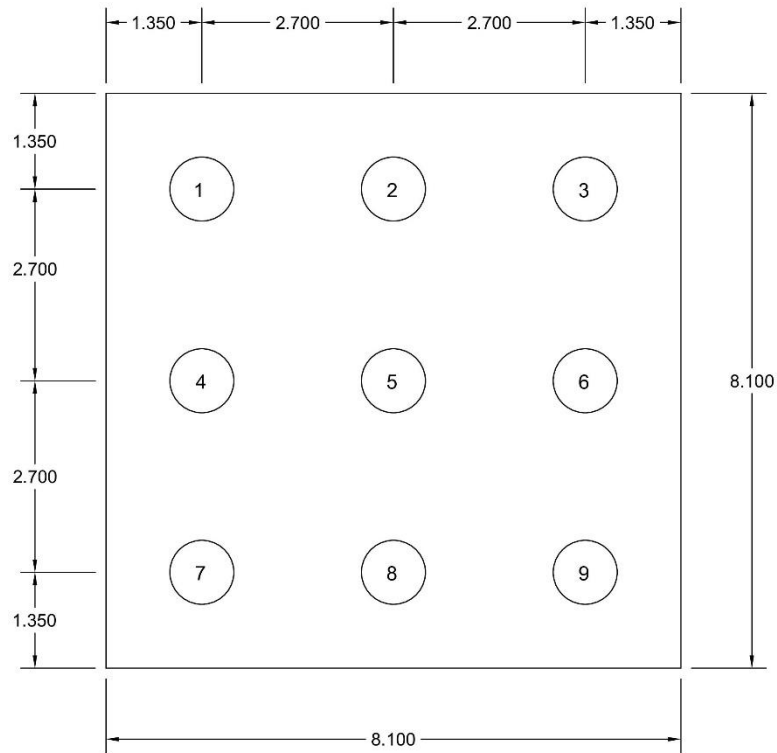


Figure 5.19 Foundation Arrangement Plan of Meyerhof Method with 0,9m Dimension

Known based on the following Figure 5.19.

Distance pile to pile, $s = 2,7$ Meter

Distance pile to edge, $a = 1,35$ Meter

Width, $B_x = 8,1$ Meter

Length, $B_y = 8,1$ Meter

3. Control of the forces that occur

The computation and control of each foundation pile plan is shown in Table 5.22 Recapitulation.

Table 5.23 Control of Each Foundation Pile Plan Meyerhof Method with 0,9m Dimension

Pile	xi	Yi	X ²	Y ²	$\sum My.xi / \sum x^2$	$\sum Mx.yi / \sum y^2$	P	Check
	m	M	m ²	m ²	kN	kN	kN	$P < Q_{all}$
1	-2,7	2,7	7,290	7,290	-100,445	100,445	390,300	Safe
2	-2,7	0	7,290	0,000	-100,445	0,000	289,855	Safe

**Continuation Table 5.22 Control of Each Foundation Pile Plan Meyerhof Method
with 0,9m Dimension**

3	-2,7	-2,7	7,290	7,290	-100,445	-100,445	189,410	Safe
4	0	2,7	0	7,290	0,000	100,445	490,745	Not Safe
5	0	0	0	0	0,000	0,000	390,300	Safe
6	0	-2,7	0	7,290	0,000	-100,445	289,855	Safe
7	2,7	2,7	7,290	7,290	100,445	100,445	591,190	Not Safe
8	2,7	0	7,290	0,000	100,445	0,000	490,745	Not Safe
9	2,7	-2,7	7,290	7,290	100,445	-100,445	390,300	Safe
			43,740	43,740			3512,699	

4. Pile Efficiency

$$m = 3$$

$$n = 3$$

$$s = 2,7 \text{ meter}$$

$$\theta = \arctg\left(\frac{D}{s}\right)$$

$$= \arctg\left(\frac{0,9}{2,7}\right)$$

$$= 0,322$$

$$Eg = 1 - \theta \frac{(n-1) \times m + (m-1) \times n}{90 \times m \times n}$$

$$= 1 - 0,322 \frac{(3-1) \times 3 + (3-1) \times 3}{90 \times 3 \times 3}$$

$$= 0,995$$

5. Group Pile Support Capacity

$$n_{\text{pile}} = 9$$

$$Q_g = Eg \times n \times Q_{\text{all}}$$

$$= 0,995 \times 9 \times 418,403$$

$$= 3747,676 \text{ Ton}$$

5.7.4 Requirement and Control of Number Piles of Meyerhof Method in Dimension Variations 0,95 m.

1. Calculating number of piles

$$\sum P_{\max} = 35126,994 \text{ kN} = 3512,699 \text{ Ton}$$

$$Q_{\text{all}} = 447,182 \text{ Ton}$$

$$n = \frac{\sum P_{\max}}{Q_{\text{all}}} = \frac{3512,699}{447,182} = 7,885 \approx 8 \text{ Pile}$$

2. Group Pile Arrangement Plan

Known based on the following figure 5.20 below.

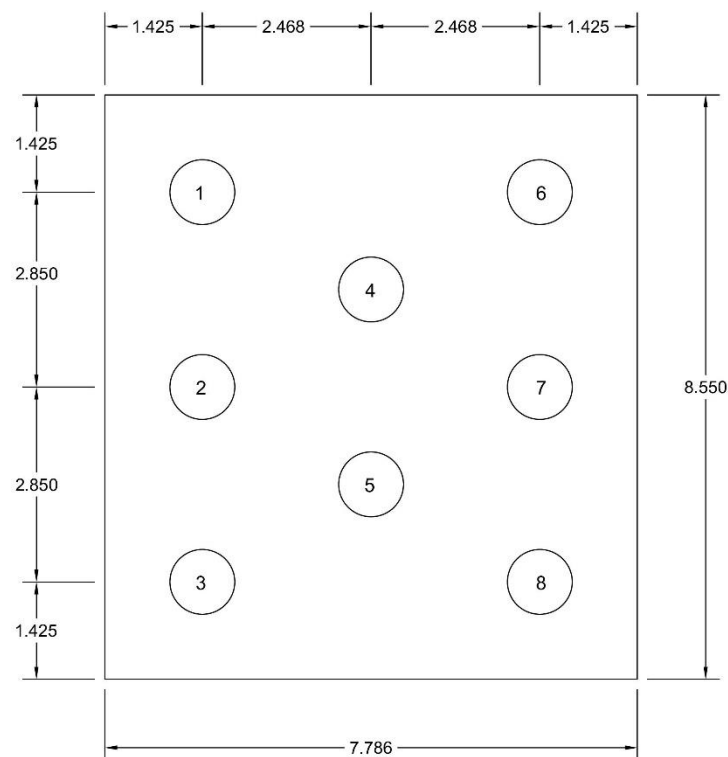


Figure 5.20 Foundation Arrangement Plan of Meyerhof Method with 0,95m Dimension

Known based on the following Figure 5.20.

Distance pile to pile, $s = 2,85$ Meter

Distance pile to edge, $a = 1,425$ Meter

Width, Bx = 7,786 Meter

Length, By = 8,55 Meter

3. Control of the forces that occur

The computation and control of each foundation pile plan is shown in Table 5.23 Recapitulation.

Table 5.24 Control of Each Foundation Pile Plan Meyerhof Method with 0,95m Dimension

Pile	xi	Yi	X ²	Y ²	$\sum My.xi/\sum x^2$	$\sum Mx.yi/\sum y^2$	P	Check
	m	M	m ²	m ²	kN	kN	kN	P < Q _{all}
1	-2,468	2,850	6,092	8,123	-109,878	246,935	576,144	Not Safe
2	-2,468	0,000	6,092	0,000	-109,878	0,000	329,209	Safe
3	-2,468	-2,85	6,092	8,123	-109,878	-246,935	82,274	Safe
4	0	1,425	0,000	2,031	0,000	123,467	562,555	Not Safe
5	0	-1,425	0,000	2,031	0,000	-123,467	315,620	Safe
6	2,468	2,850	6,092	8,123	109,878	246,935	795,901	Not Safe
7	2,468	0,000	6,092	0,000	109,878	0,000	548,966	Not Safe
8	2,468	-2,85	6,092	8,123	109,878	-246,935	302,031	Safe
			36,552	36,551			3512,699	

4. Pile Efficiency

m = 2,666

n = 2,666

s = 2,85 meter

$\theta = \arctg\left(\frac{D}{s}\right)$

$= \arctg\left(\frac{0,95}{2,85}\right)$

= 0,322

$Eg = 1 - \theta \frac{(n-1) \times m + (m-1) \times n}{90 \times m \times n}$

$= 1 - 0,322 \frac{(2,666-1) \times 2,666 + (2,666-1) \times 2,666}{90 \times 2,666 \times 2,666}$

= 0,995

5. Group Pile Support Capacity

$$\begin{aligned}
 n_{\text{pile}} &= 9 \\
 Q_g &= E_g \times n \times Q_{\text{all}} \\
 &= 0,995 \times 9 \times 447,182 \\
 &= 3561,474 \text{ Ton}
 \end{aligned}$$

5.7.5 Requirement and Control of Number Piles of Reese and Wright Method in Existing Dimension

1. Calculating number of piles

$$\sum P_{\text{max}} = 35126,994 \text{ kN} = 3512,699 \text{ Ton}$$

$$Q_{\text{all}} = 327,207 \text{ Ton}$$

$$\begin{aligned}
 n &= \frac{\sum P_{\text{max}}}{Q_{\text{all}}} \\
 &= \frac{3512,699}{327,207} = 10,735 \approx 11 \text{ Pile}
 \end{aligned}$$

2. Group Pile Arrangement Plan

Known based on the following figure 5.21 below.

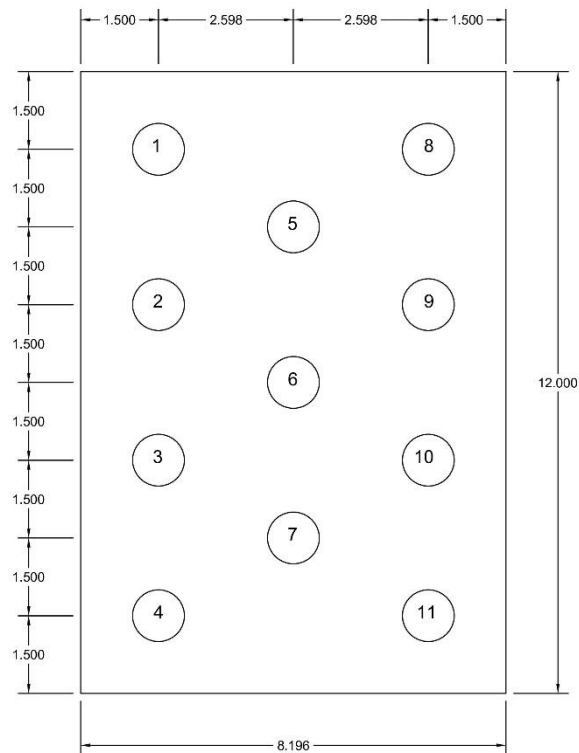


Figure 5.21 Foundation Arrangement Plan of Reese and Wright Method With 1m Dimension

Known based on the following Figure 5.21.

Distance pile to pile, $s = 3$ Meter

Distance pile to edge, $a = 1,5$ Meter

Width, $B_x = 8,196$ Meter

Length, $B_y = 12$ Meter

3. Control of the forces that occur

The computation and control of each foundation pile plan is shown in Table 5.24 Recapitulation.

Table 5.25 Control of Each Foundation Pile Plan Reese and Wright Method With 1m Dimension

Pile	xi	Yi	X ²	Y ²	$\sum My.xi/\sum x^2$	$\sum Mx.yi/\sum y^2$	P	Check
	m	M	m ²	m ²	kN	kN	kN	P < Q _{all}
1	-2,598	4,5	6,750	20,250	-78,288	131,956	373,004	Not Safe
2	-2,598	1,5	6,750	2,250	-78,288	43,985	285,033	Safe
3	-2,598	-1,5	6,750	2,250	-78,288	-43,985	197,063	Safe
4	-2,598	-4,5	6,750	20,250	-78,288	-131,956	109,092	Safe
5	0,000	3	0,000	9,000	0,000	87,971	407,307	Not Safe
6	0,000	0	0,000	0,000	0,000	0,000	319,336	Safe
7	0,000	-3	0,000	9,000	0,000	-87,971	231,366	Safe
8	2,598	4,5	6,750	20,250	78,288	131,956	529,581	Not Safe
9	2,598	1,5	6,750	2,250	78,288	43,985	441,610	Not Safe
10	2,598	-1,5	6,750	2,250	78,288	-43,985	353,640	Not Safe
11	2,598	-4,5	6,750	20,250	78,288	-131,956	265,669	Safe
			54,001	108,000			3512,699	

4. Pile Efficiency

$m = 3,666$

$n = 2,75$

$s = 3$ meter

$\theta = \arctg\left(\frac{D}{s}\right)$

$$\begin{aligned}
&= \operatorname{arctg} \left(\frac{1}{3} \right) \\
&= 0,322 \\
\text{Eg} \quad &= 1 - \theta \frac{(n-1) \times m + (m-1) \times n}{90 \times m \times n} \\
&= 1 - 0,322 \frac{(2,75-1) \times 3,666 + (3,666-1) \times 2,75}{90 \times 3,666 \times 2,75} \\
&= 0,995
\end{aligned}$$

5. Group Pile Support Capacity

$$\begin{aligned}
n \text{ pile} &= 11 \\
Q_g &= E_g \times n \times Q_{\text{all}} \\
&= 0,995 \times 11 \times 327,207 \\
&= 3581,734 \text{ Ton}
\end{aligned}$$

5.7.6 Requirement and Control of Number Piles of Reese and Wright Method in 85 cm Dimension Variation Condition

1. Calculating number of piles

$$\begin{aligned}
\sum P_{\text{max}} &= 35126,994 \text{ kN} = 3512,699 \text{ Ton} \\
Q_{\text{all}} &= 269,268 \text{ Ton} \\
n &= \frac{\sum P_{\text{max}}}{Q_{\text{all}}} \\
&= \frac{3512,699}{269,268} = 13,045 \approx 14 \text{ pile}
\end{aligned}$$

2. Group Pile Arrangement Plan

Known based on the following figure 5.22.

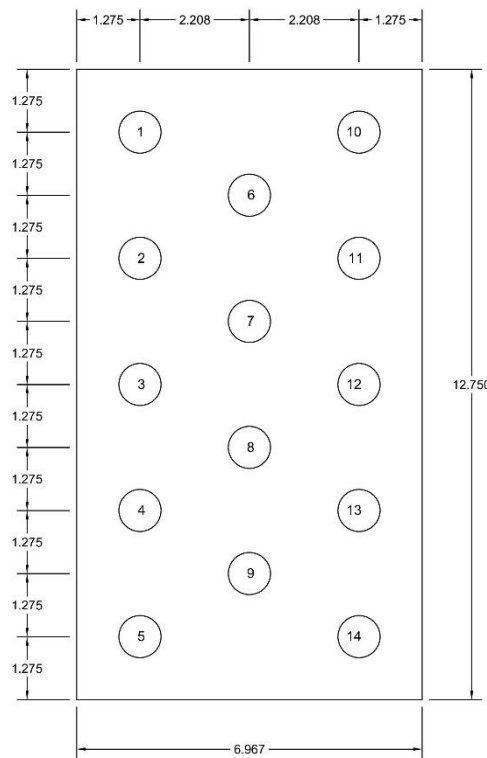


Figure 5.22 Foundation Arrangement Plan of Reese and Wright Method With 0,85m Dimension

Known based on the following Figure 5.22.

Distance pile to pile, $s = 2,55$ Meter

Distance pile to edge, $a = 1,275$ Meter

Width, $B_x = 6,7$ Meter

Length, $B_y = 12,75$ Meter

3. Control of the forces that occur

The computation and control of each foundation pile plan is shown in Table 5.25 Recapitulation.

Table 5.26 Control of Each Foundation Pile Plan Meyerhof Reese and Wright With 0,85m Dimension

Pile	x_i	Y_i	X^2	Y^2	$\sum My.x_i / \sum x^2$	$\sum Mx.y_i / \sum y^2$	P	Check
	m	M	m ²	m ²	kN	kN	kN	$P < Q_{all}$
1	-2,208	5,0294	4,877	25,295	-73,683	99,735	276,959	Not Safe
2	-2,208	2,55	4,877	6,503	-73,683	50,567	227,792	Safe

**Continuation Table 5.25 Control of Each Foundation Pile Plan Meyerhof Reese and
Wright With 0,85m Dimension**

3	-2,208	0	4,877	0,000	-73,683	0,000	177,224	Safe
4	-2,208	-2,55	4,877	6,503	-73,683	-50,567	126,657	Safe
5	-2,208	-5,0294	4,877	25,295	-73,683	-99,735	77,490	Safe
6	0	3,825	0,000	14,631	0,000	75,851	326,758	Not Safe
7	0	1,275	0,000	1,626	0,000	25,284	276,191	Not Safe
8	0	-1,275	0,000	1,626	0,000	-25,284	225,623	Safe
9	0	-3,825	0,000	14,631	0,000	-75,851	175,056	Safe
10	2,208	5,0294	4,877	25,295	73,683	99,735	424,324	Not Safe
11	2,208	2,55	4,877	6,503	73,683	50,567	375,157	Not Safe
12	2,208	0	4,877	0,000	73,683	0,000	324,590	Not Safe
13	2,208	-2,55	4,877	6,503	73,683	-50,567	274,023	Not Safe
14	2,208	-5,0294	4,877	25,295	73,683	-99,735	224,855	Safe
			48,770	159,702			3512,699	

4. Pile Efficiency

$$m = 4,666$$

$$n = 2,8$$

$$s = 2,55 \text{ meter}$$

$$\begin{aligned}\theta &= \arctg\left(\frac{D}{s}\right) \\ &= \arctg\left(\frac{0,85}{2,55}\right) \\ &= 0,322\end{aligned}$$

$$\begin{aligned}Eg &= 1 - \theta \frac{(n-1) \times m + (m-1) \times n}{90 \times m \times n} \\ &= 1 - 0,381 \frac{(2,8-1) \times 4,666 + (4,666-1) \times 2,8}{90 \times 4,666 \times 2,8} \\ &= 0,995\end{aligned}$$

5. Group Pile Support Capacity

$$n \text{ pile} = 14$$

$$\begin{aligned}Qg &= Eg \times n \times Q_{all} \\ &= 0,995 \times 14 \times 269,268 \\ &= 3750,503 \text{ Ton}\end{aligned}$$

5.7.7 Requirement and Control of Number Piles of Reese and Wright Method in 90 cm Dimension Variation Condition

1. Calculating number of piles

$$\sum P_{\max} = 35126,994 \text{ kN} = 3512,699 \text{ Ton}$$

$$Q_{\text{all}} = 290,350 \text{ Ton}$$

$$n = \frac{\sum P_{\max}}{Q_{\text{all}}} = \frac{3512,699}{290,350} = 12,098 \approx 13 \text{ pile}$$

2. Group Pile Arrangement Plan

Known based on the following figure 5.23 below.

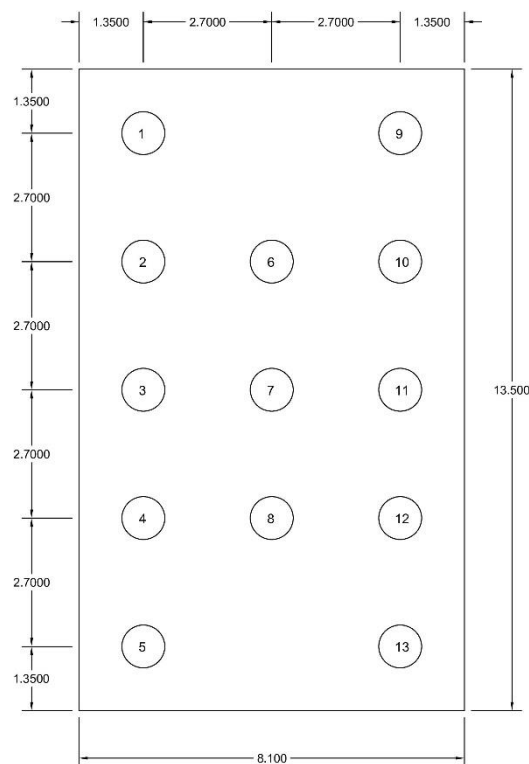


Figure 5.23 Foundation Arrangement Plan of Reese and Wright Method With 0,9m Dimension

Known based on the following Figure 5.23.

Distance pile to pile, $s = 2,7$ Meter

Distance pile to edge, $a = 1,35$ Meter

Width, Bx = 8,1 Meter

Length, By = 13,5 Meter

3. Control of the forces that occur

The computation and control of each foundation pile plan is shown in Table 5.26 Recapitulation.

Table 5.27 Control of Each Foundation Pile Plan of Reese and Wright Method With 0,9m Dimension

Pile	xi	Yi	X ²	Y ²	$\Sigma My.xi/\Sigma x^2$	$\Sigma Mx.yi/\Sigma y^2$	P	Check
	m	M	m ²	m ²	kN	kN	kN	P < Q _{all}
1	-2,700	5,4	7,290	29,160	-60,267	54,788	264,729	Safe
2	-2,700	2,7	7,290	7,290	-60,267	27,394	237,335	Safe
3	-2,700	0	7,290	0,000	-60,267	0,000	209,941	Safe
4	-2,700	-2,7	7,290	7,290	-60,267	-27,394	182,546	Safe
5	-2,700	-5,4	7,290	29,160	-60,267	-54,788	155,152	Safe
6	0,000	2,7	0,000	7,290	0,000	27,394	297,602	Not Safe
7	0,000	0	0,000	0,000	0,000	0,000	270,208	Safe
8	0,000	-2,7	0,000	7,290	0,000	-27,394	242,814	Safe
9	2,700	5,4	7,290	29,160	60,267	54,788	385,263	Not Safe
10	2,700	2,7	7,290	7,290	60,267	27,394	357,869	Not Safe
11	2,700	0	7,290	0,000	60,267	0,000	330,475	Not Safe
12	2,700	-2,7	7,290	7,290	60,267	-27,394	303,081	Not Safe
13	2,700	-5,4	7,290	29,160	60,267	-54,788	275,686	Safe
			72,900	160,380			3512,699	

4. Pile Efficiency

m = 4,333

n = 2,6

s = 2,7 meter

$$\begin{aligned}\theta &= \arctg \left(\frac{D}{s} \right) \\ &= \arctg \left(\frac{0,9}{2,7} \right) \\ &= 0,322\end{aligned}$$

$$\begin{aligned}Eg &= 1 - \theta \frac{(n-1) \times m + (m-1) \times n}{90 \times m \times n} \\ &= 1 - 0,322 \frac{(2,6-1) \times 4,333 + (4,333-1) \times 2,6}{90 \times 4,333 \times 2,6} \\ &= 0,995\end{aligned}$$

5. Group Pile Support Capacity

$$n \text{ pile} = 13$$

$$\begin{aligned}Qg &= Eg \times n \times Q_{all} \\ &= 0,995 \times 13 \times 290,350 \\ &= 3755,872 \text{ T}\end{aligned}$$

5.7.8 Requirement and Control of Number Piles of Reese and Wright Method in 95 cm Dimension Variation Condition

1. Calculating number of piles

$$\sum P_{max} = 35126,994 \text{ kN} = 3512,699 \text{ Ton}$$

$$Q_{all} = 306,198 \text{ Ton}$$

$$\begin{aligned}n &= \frac{\sum P_{max}}{Q_{all}} \\ &= \frac{3512,699}{306,198} = 11,480 \approx 12 \text{ pile}\end{aligned}$$

2. Group Pile Arrangement Plan

Known based on the following Figure 5.24 below.

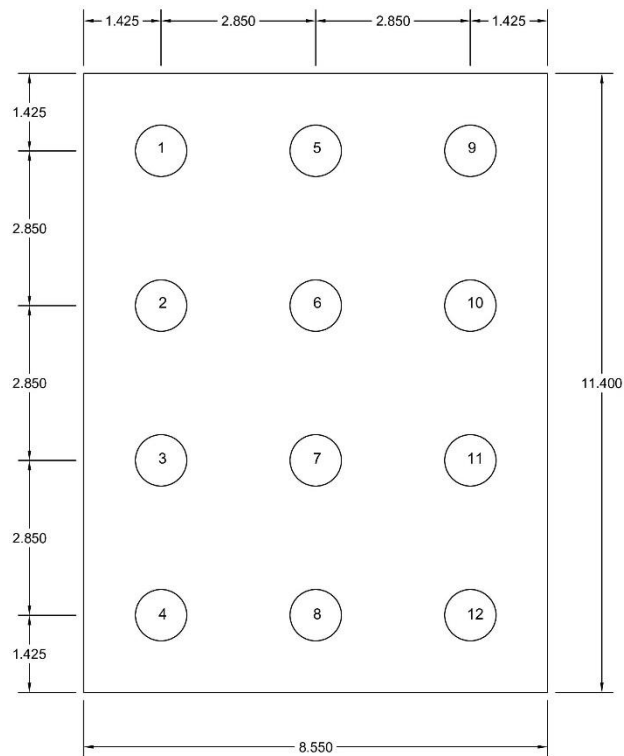


Figure 5.24 Foundation Arrangement Plan of Reese and Wright Method With 0,95m Dimension

Known based on the following Figure 5.24.

Distance pile to pile, $s = 2,85$ Meter

Distance pile to edge, $a = 1,425$ Meter

Width, $B_x = 8,55$ Meter

Length, $B_y = 11,4$ Meter

3. Control of the forces that occur

The computation and control of each foundation pile plan is shown in Table 5.27 Recapitulation.

Table 5.28 Control of Each Foundation Pile Plan Reese and Wright Method With 0,95m Dimension

Pile	x_i	Y_i	X^2	Y^2	$\frac{\sum My.x_i}{\sum x^2}$	$\frac{\sum Mx.y_i}{\sum y^2}$	P	Check
	m	M	m ²	m ²	kN	kN	kN	$P < Q_{all}$
1	-2,85	4,275	8,123	18,276	-71,369	57,095	278,451	Safe

Continuation Table 5.27 Control of Each Foundation Pile Plan Reese and Wright
Method With 0,95m Dimension

2	-2,85	1,425	8,123	2,031	-71,369	19,032	240,388	Safe
3	-2,85	-1,425	8,123	2,031	-71,369	-19,032	202,324	Safe
4	-2,85	-4,275	8,123	18,276	-71,369	-57,095	164,261	Safe
5	0	4,275	0,000	18,276	0,000	57,095	349,820	Not Safe
6	0	1,425	0,000	2,031	0,000	19,032	311,757	Not Safe
7	0	-1,425	0,000	2,031	0,000	-19,032	273,693	Safe
8	0	-4,275	0,000	18,276	0,000	-57,095	235,630	Safe
9	2,85	4,275	8,123	18,276	71,369	57,095	421,189	Not Safe
10	2,85	1,425	8,123	2,031	71,369	19,032	383,126	Not Safe
11	2,85	-1,425	8,123	2,031	71,369	-19,032	345,062	Not Safe
12	2,85	-4,275	8,123	18,276	71,369	-57,095	306,999	Not Safe
			64,980	121,838			3512,699	

4. Pile Efficiency

$$m = 3$$

$$n = 4$$

$$s = 2,85 \text{ meter}$$

$$\theta = \arctg\left(\frac{D}{s}\right)$$

$$= \arctg\left(\frac{0,95}{2,85}\right)$$

$$= 0,332$$

$$Eg = 1 - \theta \frac{(n-1) \times m + (m-1) \times n}{90 \times m \times n}$$

$$= 1 - 0,332 \frac{(4-1) \times 3 + (3-1) \times 4}{90 \times 3 \times 4}$$

$$= 0,995$$

5. Group Pile Support Capacity

$$n_{\text{pile}} = 12$$

$$\begin{aligned} Q_g &= E_g \times n \times Q_{\text{all}} \\ &= 0,995 \times 12 \times 306,198 \\ &= 3655,765 \text{ T} \end{aligned}$$

5.8 Analysis of Bored Pile Elastic Settlement

5.8.1 Elastic Settlement for Meyerhof Method in Existing Dimension

a. Settlement in single pile

1) Elastic shortening of the pile can be calculated:

$$\begin{aligned} Se_1 &= \frac{(Q_{wp} + \xi \times Q_{ws})L}{A_p E_p} \\ &= \frac{(908,2 + 0,5 \times 314) \times 12}{0,785 \times 2574296,02} \\ &= 0,0039 \text{ m} \end{aligned}$$

2) Settlement of the pile due to the working load at the pile point can be calculated:

$$\begin{aligned} Se_2 &= \frac{C_p \times Q_{wp}}{D \times q_p} \\ &= \frac{C_p \times Q_{wp}}{D \times (90 \times C_u)} \\ &= \frac{C_p \times Q_{wp}}{D \times \left(90 \times \left(\frac{2}{3} \times N_{spt} \times 10\right)\right)} \\ &= \frac{0,05 \times 908,2}{1 \times \left(90 \times \left(\frac{2}{3} \times 60 \times 10\right)\right)} \\ &= 0,00126 \text{ m} \end{aligned}$$

3) For calculating influence factor, I_{ws} , can be calculated:

$$\begin{aligned} I_{ws} &= 2 + 0,35 \sqrt{\frac{L}{D}} \\ &= 2 + 0,35 \sqrt{\frac{12}{1}} \\ &= 3,212 \end{aligned}$$

4) Settlement of the pile due to the working load along the pile shaft can be calculated:

$$\begin{aligned}
Se_3 &= \left(\frac{Q_{ws}}{p \times L} \right) \frac{D}{E_s} (1 - \mu_s^2) I_{ws} \\
&= \left(\frac{942}{3,14 \times 12} \right) \times \frac{1}{23618} \times (1 - 0,4^2) \times 3,212 \\
&= 0,00095 \text{ m}
\end{aligned}$$

5) So, the total settlement is:

$$\begin{aligned}
S_e &= Se_1 + Se_2 + Se_3 \\
&= 0,0039 + 0,00126 + 0,00095 \\
&= 0,0061 \text{ m} \approx 6,1 \text{ mm}
\end{aligned}$$

b. Settlement in group pile

To determine settlement in group pile can be calculated:

$$\begin{aligned}
S_g &= S_e \times \sqrt{\frac{B_g}{D}} \\
&= 0,0061 \times \sqrt{\frac{7}{1}} \\
&= 0,0145 \text{ m} \approx 14,5 \text{ mm}
\end{aligned}$$

5.8.2 Elastic Settlement for Meyerhof Method in Dimension Variations 0,85 m.

a. Settlement in single pile

1) Elastic shortening of the pile can be calculated:

$$\begin{aligned}
Se_1 &= \frac{(Q_{wp} + \xi \times Q_{ws})L}{A_p E_p} \\
&= \frac{(680,940 + 0,5 \times 266,9) \times 12}{0,567 \times 2574296,02} \\
&= 0,00215 \text{ m}
\end{aligned}$$

2) Settlement of the pile due to the working load at the pile point can be calculated:

$$\begin{aligned}
Se_2 &= \frac{c_p \times Q_{wp}}{D \times q_p} \\
&= \frac{c_p \times Q_{wp}}{D \times (90 \times C_u)} \\
&= \frac{c_p \times Q_{wp}}{D \times \left(90 \times \left(\frac{2}{3} \times N_{spt} \times 10 \right) \right)} \\
&= \frac{0,05 \times 680,940}{0,85 \times \left(90 \times \left(\frac{2}{3} \times 60 \times 10 \right) \right)} \\
&= 0,00111 \text{ m}
\end{aligned}$$

3) For calculating influence factor, I_{ws} , can be calculated:

$$\begin{aligned} I_{ws} &= 2 + 0,35 \sqrt{\frac{L}{D}} \\ &= 2 + 0,35 \sqrt{\frac{12}{0,85}} \\ &= 3,315 \end{aligned}$$

4) Settlement of the pile due to the working load along the pile shaft can be calculated:

$$\begin{aligned} Se_3 &= \left(\frac{Q_{ws}}{p \times L} \right) \frac{D}{E_s} (1 - \mu_s^2) I_{ws} \\ &= \left(\frac{266,9}{2,669 \times 12} \right) \times \frac{0,85}{23618} \times (1 - 0,4^2) \times 3,315 \\ &= 0,00084 \text{ m} \end{aligned}$$

5) So, the total settlement is:

$$\begin{aligned} S_e &= Se_1 + Se_2 + Se_3 \\ &= 0,00215 + 0,00111 + 0,00084 \\ &= 0,0041 \text{ m} \approx 4,1 \text{ mm} \end{aligned}$$

b. Settlement in group pile

To determine settlement in group pile can be calculated:

$$\begin{aligned} S_g &= S_e \times \sqrt{\frac{B_g}{D}} \\ &= 0,0041 \times \sqrt{\frac{5,267}{0,85}} \\ &= 0,0102 \text{ m} \approx 10,2 \text{ mm} \end{aligned}$$

5.8.3 Elastic Settlement for Meyerhof Method in Dimension Variations 0,9 m.

a. Settlement in single pile

1) Elastic shortening of the pile can be calculated:

$$\begin{aligned} Se_1 &= \frac{(Q_{wp} + \xi \times Q_{ws})L}{A_p E_p} \\ &= \frac{1045,62 \times 12}{0,636 \times 2574296,02} \\ &= 0,00268 \text{ m} \end{aligned}$$

2) Settlement of the pile due to the working load at the pile point can be calculated:

$$\begin{aligned}
Se_2 &= \frac{C_p \times Q_{wp}}{D \times q_p} \\
&= \frac{C_p \times Q_{wp}}{D \times (90 \times C_u)} \\
&= \frac{C_p \times Q_{wp}}{D \times \left(90 \times \left(\frac{2}{3} \times N_{spt} \times 10\right)\right)} \\
&= \frac{0,05 \times 763,407}{0,9 \times \left(90 \times \left(\frac{2}{3} \times 60 \times 10\right)\right)} \\
&= 0,00118 \text{ m}
\end{aligned}$$

3) For calculating influence factor, I_{ws} , can be calculated:

$$\begin{aligned}
I_{ws} &= 2 + 0,35 \sqrt{\frac{L}{D}} \\
&= 2 + 0,35 \sqrt{\frac{12}{0,9}} \\
&= 3,278
\end{aligned}$$

4) Settlement of the pile due to the working load along the pile shaft can be calculated:

$$\begin{aligned}
Se_3 &= \left(\frac{Q_{ws}}{p \times L}\right) \frac{D}{E_s} (1 - \mu_s^2) I_{ws} \\
&= \left(\frac{282,6}{2,826 \times 12}\right) \times \frac{0,9}{23618} \times (1 - 0,4^2) \times 3,278 \\
&= 0,00087 \text{ m}
\end{aligned}$$

5) So, the total settlement is:

$$\begin{aligned}
S_e &= Se_1 + Se_2 + Se_3 \\
&= 0,00268 + 0,00118 + 0,00087 \\
&= 0,0047 \text{ m} \approx 4,7 \text{ mm}
\end{aligned}$$

b. Settlement in group pile

To determine settlement in group pile can be calculated:

$$\begin{aligned}
S_g &= S_e \times \sqrt{\frac{B_g}{D}} \\
&= 0,0047 \times \sqrt{\frac{6,3}{0,9}} \\
&= 0,0125 \text{ m} \approx 12,5 \text{ mm}
\end{aligned}$$

5.8.4 Elastic Settlement for Meyerhof Method in Dimension Variations 0,95 m.

a. Settlement in single pile

1) Elastic shortening of the pile can be calculated:

$$\begin{aligned} Se_1 &= \frac{(Q_{wp} + \xi \times Q_{ws})L}{A_p E_p} \\ &= \frac{(819,656 + 0,5 \times 298,3) \times 12}{0,708 \times 2574296,02} \\ &= 0,0032 \text{ m} \end{aligned}$$

2) Settlement of the pile due to the working load at the pile point can be calculated:

$$\begin{aligned} Se_2 &= \frac{C_p \times Q_{wp}}{D \times q_p} \\ &= \frac{C_p \times Q_{wp}}{D \times (90 \times C_u)} \\ &= \frac{C_p \times Q_{wp}}{D \times \left(90 \times \left(\frac{2}{3} \times N_{spt} \times 10\right)\right)} \\ &= \frac{0,05 \times 819,656}{0,95 \times \left(90 \times \left(\frac{2}{3} \times 60 \times 10\right)\right)} \\ &= 0,00120 \text{ m} \end{aligned}$$

3) For calculating influence factor, I_{ws} , can be calculated:

$$\begin{aligned} I_{ws} &= 2 + 0,35 \sqrt{\frac{L}{D}} \\ &= 2 + 0,35 \sqrt{\frac{12}{0,95}} \\ &= 3,244 \end{aligned}$$

4) Settlement of the pile due to the working load along the pile shaft can be calculated:

$$\begin{aligned} Se_3 &= \left(\frac{Q_{ws}}{p \times L}\right) \frac{D}{E_s} (1 - \mu_s^2) I_{ws} \\ &= \left(\frac{298,3}{2,983 \times 12}\right) \times \frac{0,95}{23618} \times (1 - 0,4^2) \times 3,244 \\ &= 0,00091 \text{ m} \end{aligned}$$

5) So, the total settlement is:

$$\begin{aligned} S_e &= Se_1 + Se_2 + Se_3 \\ &= 0,0032 + 0,0012 + 0,00091 \end{aligned}$$

$$= 0,0053 \text{ m} \approx 5,3 \text{ mm}$$

b. Settlement in group pile

To determine settlement in group pile can be calculated:

$$\begin{aligned} S_g &= S_e \times \sqrt{\frac{B_g}{D}} \\ &= 0,0055 \times \sqrt{\frac{5,886}{0,95}} \\ &= 0,0132 \text{ m} \approx 13,2 \text{ mm} \end{aligned}$$

5.8.5 Elastic Settlement for Reese and Wright Method in Existing Dimension

a. Settlement in single pile

1) Elastic shortening of the pile can be calculated:

$$\begin{aligned} Se_1 &= \frac{(Q_{wp} + \xi \times Q_{ws})L}{A_p E_p} \\ &= \frac{(242,351 + 0,5 \times 575,667) \times 12}{0,785 \times 2574296,02} \\ &= 0,00194 \text{ m} \end{aligned}$$

2) Settlement of the pile due to the working load at the pile point can be calculated:

$$\begin{aligned} Se_2 &= \frac{C_p \times Q_{wp}}{D \times q_p} \\ &= \frac{C_p \times Q_{wp}}{D \times (90 \times C_u)} \\ &= \frac{C_p \times Q_{wp}}{D \times \left(90 \times \left(\frac{2}{3} \times N_{spt} \times 10\right)\right)} \\ &= \frac{0,05 \times 242,351}{1 \times \left(90 \times \left(\frac{2}{3} \times 60 \times 10\right)\right)} \\ &= 0,00034 \text{ m} \end{aligned}$$

3) For calculating influence factor, I_{ws} , can be calculated:

$$\begin{aligned} I_{ws} &= 2 + 0,35 \sqrt{\frac{L}{D}} \\ &= 2 + 0,35 \sqrt{\frac{12}{1}} \\ &= 3,212 \end{aligned}$$

- 4) Settlement of the pile due to the working load along the pile shaft can be calculated:

$$\begin{aligned} Se_3 &= \left(\frac{Q_{ws}}{p \times L} \right) \frac{D}{E_s} (1 - \mu_s^2) I_{ws} \\ &= \left(\frac{575,667}{3,14 \times 12} \right) \times \frac{0,85}{23618} \times (1 - 0,4^2) \times 3,212 \\ &= 0,00175 \text{ m} \end{aligned}$$

- 5) So, the total settlement is:

$$\begin{aligned} S_e &= Se_1 + Se_2 + Se_3 \\ &= 0,00194 + 0,00034 + 0,00175 \\ &= 0,00402 \text{ m} \approx 4,02 \text{ mm} \end{aligned}$$

- b. Settlement in group pile

To determine settlement in group pile can be calculated:

$$\begin{aligned} S_g &= S_e \times \sqrt{\frac{B_g}{D}} \\ &= 0,00402 \times \sqrt{\frac{7}{1}} \\ &= 0,01 \text{ m} \approx 10 \text{ mm} \end{aligned}$$

5.8.6 Elastic Settlement for Reese and Wright Method in Dimension Variations 0,85 m.

- a. Settlement in single pile

- 1) Elastic shortening of the pile can be calculated:

$$\begin{aligned} Se_1 &= \frac{(Q_{wp} + \xi \times Q_{ws})L}{A_p E_p} \\ &= \frac{(183,854 + 0,5 \times 489,316) \times 12}{0,567 \times 2574296,02} \\ &= 0,00113 \text{ m} \end{aligned}$$

- 2) Settlement of the pile due to the working load at the pile point can be calculated:

$$\begin{aligned} Se_2 &= \frac{C_p \times Q_{wp}}{D \times q_p} \\ &= \frac{C_p \times Q_{wp}}{D \times (90 \times C_u)} \\ &= \frac{C_p \times Q_{wp}}{D \times \left(90 \times \left(\frac{2}{3} \times N_{spt} \times 10 \right) \right)} \end{aligned}$$

$$= \frac{0,05 \times 183,854}{0,85 \times \left(90 \times \left(\frac{2}{3} \times 60 \times 10\right)\right)}$$

$$= 0,0003 \text{ m}$$

3) For calculating influence factor, I_{ws} , can be calculated:

$$I_{ws} = 2 + 0,35 \sqrt{\frac{L}{D}}$$

$$= 2 + 0,35 \sqrt{\frac{12}{0,85}}$$

$$= 3,315$$

4) Settlement of the pile due to the working load along the pile shaft can be calculated:

$$Se_3 = \left(\frac{Q_{ws}}{p \times L}\right) \frac{D}{E_s} (1 - \mu_s^2) I_{ws}$$

$$= \left(\frac{489,316}{2,669 \times 12}\right) \times \frac{1}{23618} \times (1 - 0,4^2) \times 3,315$$

$$= 0,00153 \text{ m}$$

5) So, the total settlement is:

$$S_e = Se_1 + Se_2 + Se_3$$

$$= 0,00113 + 0,0003 + 0,00153$$

$$= 0,003 \text{ m} \approx 3 \text{ mm}$$

b. Settlement in group pile

To determine settlement in group pile can be calculated:

$$S_g = S_e \times \sqrt{\frac{B_g}{D}}$$

$$= 0,003 \times \sqrt{\frac{5,267}{0,85}}$$

$$= 0,0074 \text{ m} \approx 7,4 \text{ mm}$$

5.8.7 Elastic settlement for Reese and Wright Method in Dimension Variations 0,9 m.

a. Settlement in single pile

1) Elastic shortening of the pile can be calculated:

$$Se_1 = \frac{(Q_{wp} + \xi \times Q_{ws})L}{A_p E_p}$$

$$= \frac{(207,776 + 0,5 \times 518,1) \times 12}{0,636 \times 2574296,02}$$

$$= 0,00138 \text{ m}$$

- 2) Settlement of the pile due to the working load at the pile point can be calculated:

$$\begin{aligned} \text{Se}_2 &= \frac{C_p \times Q_{wp}}{D \times q_p} \\ &= \frac{C_p \times Q_{wp}}{D \times (90 \times C_u)} \\ &= \frac{C_p \times Q_{wp}}{D \times \left(90 \times \left(\frac{2}{3} \times N_{spt} \times 10\right)\right)} \\ &= \frac{0,05 \times 207,776}{0,9 \times \left(90 \times \left(\frac{2}{3} \times 60 \times 10\right)\right)} \\ &= 0,00032 \text{ m} \end{aligned}$$

- 3) For calculating influence factor, I_{ws} , can be calculated:

$$\begin{aligned} I_{ws} &= 2 + 0,35 \sqrt{\frac{L}{D}} \\ &= 2 + 0,35 \sqrt{\frac{12}{0,9}} \\ &= 3,278 \end{aligned}$$

- 4) Settlement of the pile due to the working load along the pile shaft can be calculated:

$$\begin{aligned} \text{Se}_3 &= \left(\frac{Q_{ws}}{p \times L}\right) \frac{D}{E_s} (1 - \mu_s^2) I_{ws} \\ &= \left(\frac{196,205}{2,826 \times 12}\right) \times \frac{0,9}{23618} \times (1 - 0,4^2) \times 3,278 \\ &= 0,0016 \text{ m} \end{aligned}$$

- 5) So, the total settlement is:

$$\begin{aligned} S_e &= \text{Se}_1 + \text{Se}_2 + \text{Se}_3 \\ &= 0,00138 + 0,00032 + 0,0016 \\ &= 0,0033 \text{ m} \approx 3,3 \text{ mm} \end{aligned}$$

- b. Settlement in group pile

To determine settlement in group pile can be calculated:

$$S_g = S_e \times \sqrt{\frac{B_g}{D}}$$

$$\begin{aligned}
&= 0,0033 \times \sqrt{\frac{6,3}{0,9}} \\
&= 0,0088 \text{ m} \approx 8,8 \text{ mm}
\end{aligned}$$

5.8.8 Elastic settlement for Reese and Wright Method in Dimension Variations 0,95 m.

a. Settlement in single pile

1) Elastic shortening of the pile can be calculated:

$$\begin{aligned}
Se_1 &= \frac{(Q_{wp} + \xi \times Q_{ws})L}{A_p E_p} \\
&= \frac{(218,611 + 0,5 \times 546,883) \times 12}{0,708 \times 2574296,02} \\
&= 0,00162 \text{ m}
\end{aligned}$$

2) Settlement of the pile due to the working load at the pile point can be calculated:

$$\begin{aligned}
Se_2 &= \frac{C_p \times Q_{wp}}{D \times q_p} \\
&= \frac{C_p \times Q_{wp}}{D \times (90 \times C_u)} \\
&= \frac{C_p \times Q_{wp}}{D \times \left(90 \times \left(\frac{2}{3} \times N_{spt} \times 10\right)\right)} \\
&= \frac{0,05 \times 218,611}{0,95 \times \left(90 \times \left(\frac{2}{3} \times 60 \times 10\right)\right)} \\
&= 0,00032 \text{ m}
\end{aligned}$$

3) For calculating influence factor, I_{ws} , can be calculated:

$$\begin{aligned}
I_{ws} &= 2 + 0,35 \sqrt{\frac{L}{D}} \\
&= 2 + 0,35 \sqrt{\frac{12}{0,95}} \\
&= 3,244
\end{aligned}$$

4) Settlement of the pile due to the working load along the pile shaft can be calculated:

$$\begin{aligned}
Se_3 &= \left(\frac{Q_{ws}}{p \times L}\right) \frac{D}{E_s} (1 - \mu_s^2) I_{ws} \\
&= \left(\frac{2734,42}{2,983 \times 12}\right) \times \frac{0,95}{23618} \times (1 - 0,4^2) \times 3,244
\end{aligned}$$

$$= 0,00167 \text{ m}$$

5) So, the total settlement is:

$$\begin{aligned} S_e &= S_{e1} + S_{e2} + S_{e3} \\ &= 0,00162 + 0,0003 + 0,00167 \\ &= 0,0036 \text{ m} \approx 3,6 \text{ mm} \end{aligned}$$

b. Settlement in group pile

To determine settlement in group pile can be calculated:

$$\begin{aligned} S_g &= S_e \times \sqrt{\frac{B_g}{D}} \\ &= 0,0036 \times \sqrt{\frac{6,65}{0,95}} \\ &= 0,0096 \text{ m} \approx 9,6 \text{ mm} \end{aligned}$$

5.9 Analysis of Bored Pile Consolidation Settlement

5.9.1 Consolidation Settlement for Meyerhof Method for Existing Dimension

To calculate consolidation settlement can be seen below.

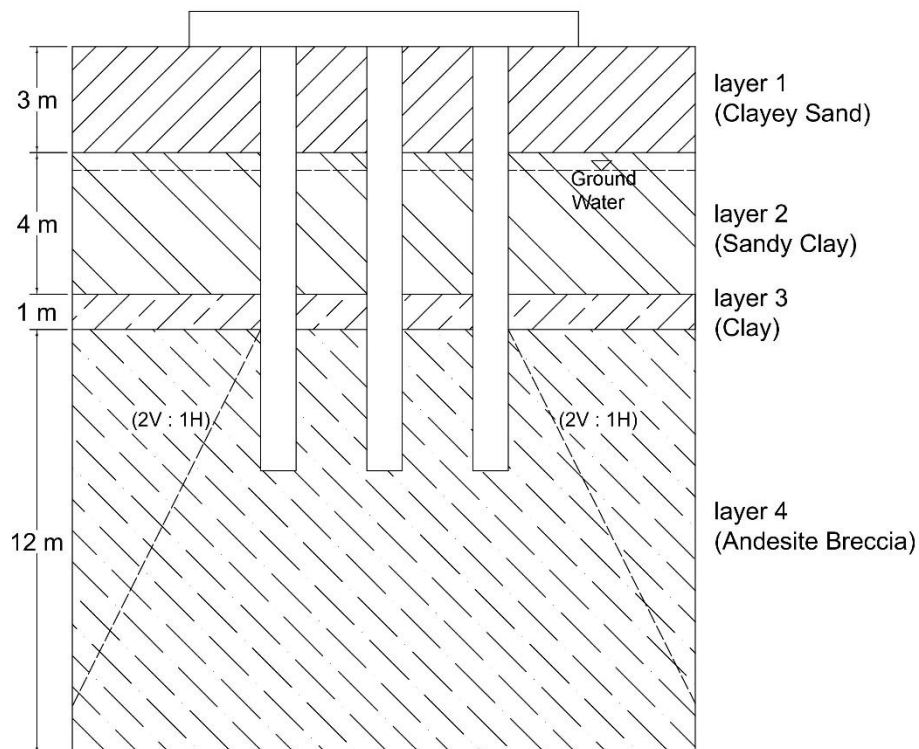


Figure 5.25 Consolidation Settlement for Meyerhof Method Equation with 1m Dimension

To determine consolidation settlement can be calculated:

$$Q_g = 35150,39 \text{ kN}$$

$$B_g = 5,6 \text{ m}$$

$$L_g = 10 \text{ m}$$

$$\begin{aligned}\Delta\sigma' &= \frac{Q_g}{(L_g+z_1)(B_g+z_1)} \\ &= \frac{35126,994}{(10+6)(5,6+6)} \\ &= 18,926 \text{ kN/m}^2\end{aligned}$$

and,

$$\begin{aligned}\sigma'_o &= 3(15,892) + 4(17,805 - 9,81) + 1(16,363 - 9,81) + 12(22,455 - \\ &\quad 9,81) \\ &= 237,951 \text{ kN/m}^2\end{aligned}$$

So,

$$\begin{aligned}\Delta S_c &= \left(\frac{C_c \times H_1}{1+e_o}\right) \log \left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right) \\ &= \left(\frac{0 \times 12}{1+0,224}\right) \log \left(\frac{237,951+18,926}{237,951}\right) \\ &= 0,00745 \text{ m} = 7,45 \text{ mm}\end{aligned}$$

5.9.2 Consolidation Settlement for Meyerhof Method in Dimension Variations 0,85 m.

To calculate consolidation settlement can be seen below.

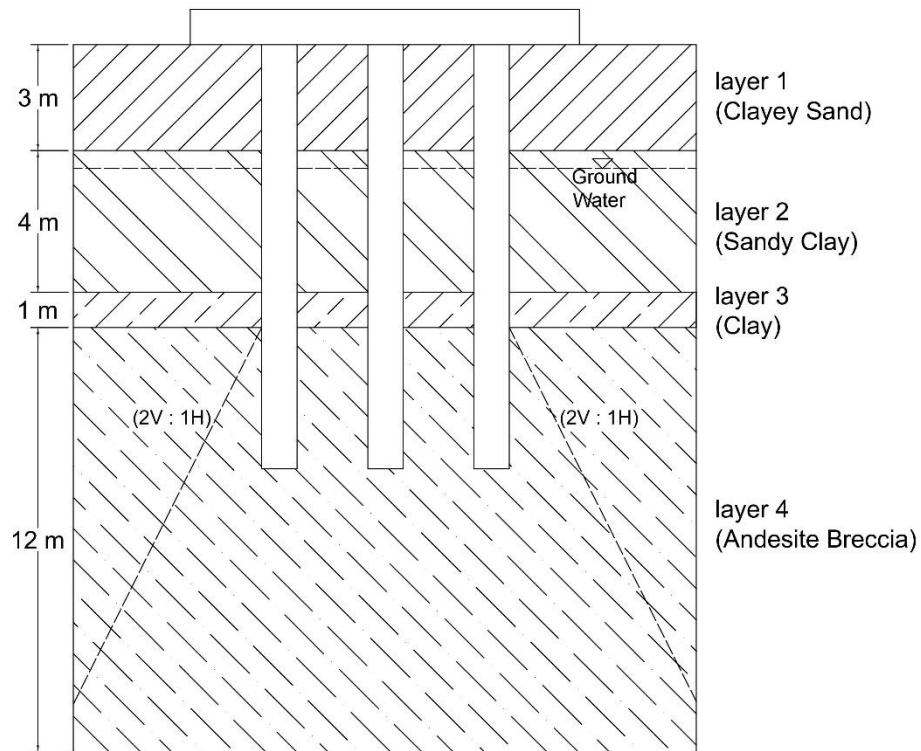


Figure 5.26 Consolidation Settlement for Meyerhof Method Equation with 0,85m Dimension

To determine consolidation settlement can be calculated:

$$Q_g = 35150,39 \text{ kN}$$

$$B_g = 5,267 \text{ m}$$

$$L_g = 8,5 \text{ m}$$

$$\begin{aligned} \Delta\sigma' &= \frac{Q_g}{(L_g + z_1)(B_g + z_1)} \\ &= \frac{35150,39}{(8,5+6)(5,267+6)} \\ &= 21,502 \text{ kN/m}^2 \end{aligned}$$

and,

$$\begin{aligned} \sigma'_o &= 3(15,892) + 4(17,805 - 9,81) + 1(16,363 - 9,81) + 12(22,455 - 9,81) \\ &= 237,951 \text{ kN/m}^2 \end{aligned}$$

So,

$$\Delta S_c = \left(\frac{C_c \times H_1}{1 + e_o} \right) \log \left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o} \right)$$

$$= \left(\frac{0 \times 12}{1 + 0,224} \right) \log \left(\frac{237,951 + 21,502}{237,951} \right)$$

$$= 0,00842 \text{ m} = 8,42 \text{ mm}$$

5.9.3 Consolidation Settlement for Meyerhof Method in Dimension Variations 0,9 m.

To calculate consolidation settlement can be seen below.

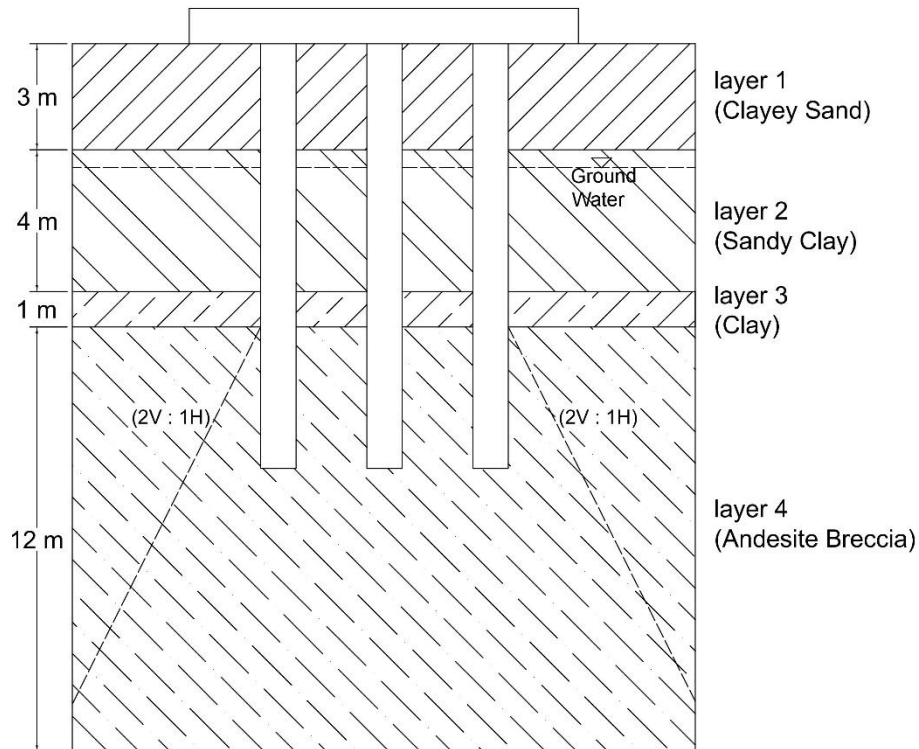


Figure 5.27 Consolidation Settlement for Meyerhof Method Equation with 0,9m Dimension

To determine consolidation settlement can be calculated:

$$Q_g = 35150,39 \text{ kN}$$

$$B_g = 6,3 \text{ m}$$

$$L_g = 6,3 \text{ m}$$

$$\Delta \sigma' = \frac{Q_g}{(L_g + z_1)(B_g + z_1)}$$

$$= \frac{35150,39}{(6,3 + 6)(6,3 + 6)}$$

$$= 23,218 \text{ kN/m}^2$$

and,

$$\begin{aligned}\sigma'_o &= 3(15,892) + 4(17,805 - 9,81) + 1(16,363 - 9,81) + 12(22,455 - 9,81) \\ &= 237,951 \text{ kN/m}^2\end{aligned}$$

So,

$$\begin{aligned}\Delta S_c &= \left(\frac{C_c \times H_1}{1+e_o}\right) \log \left(\frac{\sigma'_o + \Delta \sigma'}{\sigma'_o}\right) \\ &= \left(\frac{0 \times 12}{1+0,224}\right) \log \left(\frac{237,951+23,218}{237,951}\right) \\ &= 0,00906 \text{ m} = 9,06 \text{ mm}\end{aligned}$$

5.9.4 Consolidation Settlement for Meyerhof Method in Dimension Variations 0,95 m.

To calculate consolidation settlement can be seen below.

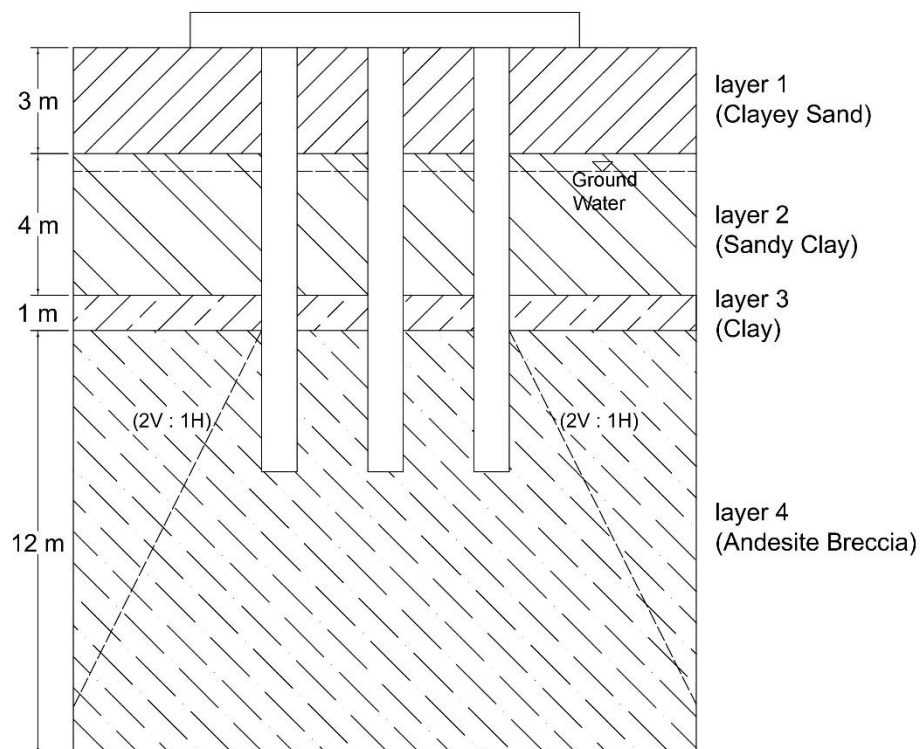


Figure 5.28 Consolidation Settlement for Meyerhof Method Equation with 0,95m Dimension

To determine consolidation settlement can be calculated:

$$Q_g = 35150,39 \text{ kN}$$

$$B_g = 5,886 \text{ m}$$

$$L_g = 6,65 \text{ m}$$

$$\begin{aligned}\Delta\sigma' &= \frac{Q_g}{(L_g+z_1)(B_g+z_1)} \\ &= \frac{35150,39}{(5,886+6)(6,65+6)} \\ &= 23,362 \text{ kN/m}^2\end{aligned}$$

and,

$$\begin{aligned}\sigma'_o &= 3(15,892) + 4(17,805 - 9,81) + 1(16,363 - 9,81) + 12(22,455 - \\ &\quad 9,81) \\ &= 237,951 \text{ kN/m}^2\end{aligned}$$

So,

$$\begin{aligned}\Delta S_c &= \left(\frac{C_c \times H_1}{1+e_o}\right) \log \left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right) \\ &= \left(\frac{0 \times 12}{1+0,224}\right) \log \left(\frac{237,951+23,362}{237,951}\right) \\ &= 0,00911 \text{ m} = 9,11 \text{ mm}\end{aligned}$$

5.9.5 Consolidation Settlement for Reese and Wright Method for Existing Dimension

To calculate consolidation settlement can be seen below.

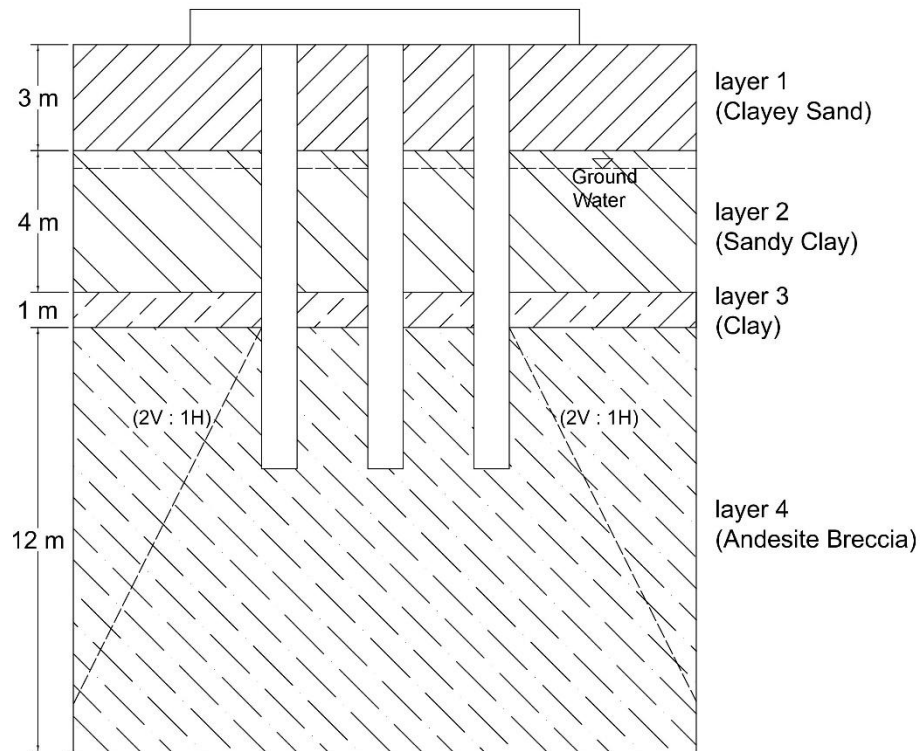


Figure 5.29 Consolidation Settlement for Reese and Wright Method Equation with 1m Dimension

To determine consolidation settlement can be calculated:

$$Q_g = 35150,39 \text{ kN}$$

$$B_g = 6,196 \text{ m}$$

$$L_g = 10 \text{ m}$$

$$\begin{aligned} \Delta\sigma' &= \frac{Q_g}{(L_g + z_1)(B_g + z_1)} \\ &= \frac{35150,39}{(10+6)(6,196+6)} \\ &= 18 \text{ kN/m}^2 \end{aligned}$$

and,

$$\begin{aligned} \sigma'_o &= 3(15,892) + 4(17,805 - 9,81) + 1(16,363 - 9,81) + 12(22,455 - 9,81) \\ &= 237,951 \text{ kN/m}^2 \end{aligned}$$

So,

$$\Delta S_c = \left(\frac{C_c x H_1}{1 + e_o} \right) \log \left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o} \right)$$

$$= \left(\frac{0 \times 12}{1 + 0,224} \right) \log \left(\frac{237,951 + 18}{237,951} \right)$$

$$= 0,00709 \text{ m} = 7,09 \text{ mm}$$

5.9.6 Consolidation Settlement for Reese and Wright Method in Dimension Variations 0,85 m

To calculate consolidation settlement can be seen below.

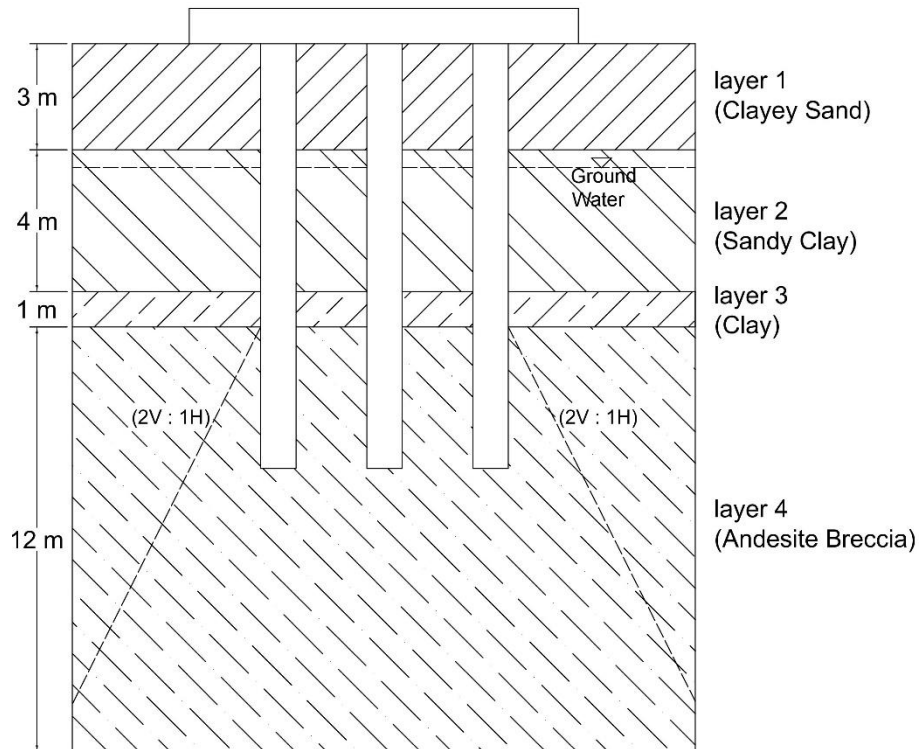


Figure 5.30 Consolidation Settlement for Reese and Wright Method Equation with 0,85m Dimension

To determine consolidation settlement can be calculated:

$$Q_g = 35150,39 \text{ kN}$$

$$B_g = 5,267 \text{ m}$$

$$L_g = 8,5 \text{ m}$$

$$\Delta\sigma' = \frac{Q_g}{(L_g + z_1)(B_g + z_1)}$$

$$= \frac{35150,39}{(12,75 + 6)(6,7 + 6)}$$

$$= 21,502 \text{ kN/m}^2$$

and,

$$\begin{aligned}\sigma'_o &= 3(15,892) + 4(17,805 - 9,81) + 1(16,363 - 9,81) + 12(22,455 - 9,81) \\ &= 237,951 \text{ kN/m}^2\end{aligned}$$

So,

$$\begin{aligned}\Delta S_c &= \left(\frac{C_c \times H_1}{1+e_o} \right) \log \left(\frac{\sigma'_o + \Delta \sigma'}{\sigma'_o} \right) \\ &= \left(\frac{0 \times 12}{1+0,224} \right) \log \left(\frac{237,951+21,502}{237,951} \right) \\ &= 0,00842 \text{ m} = 8,42 \text{ mm}\end{aligned}$$

5.9.7 Consolidation Settlement for Reese and Wright Method in Dimension Variations 0,9 m

To calculate consolidation settlement can be seen below.

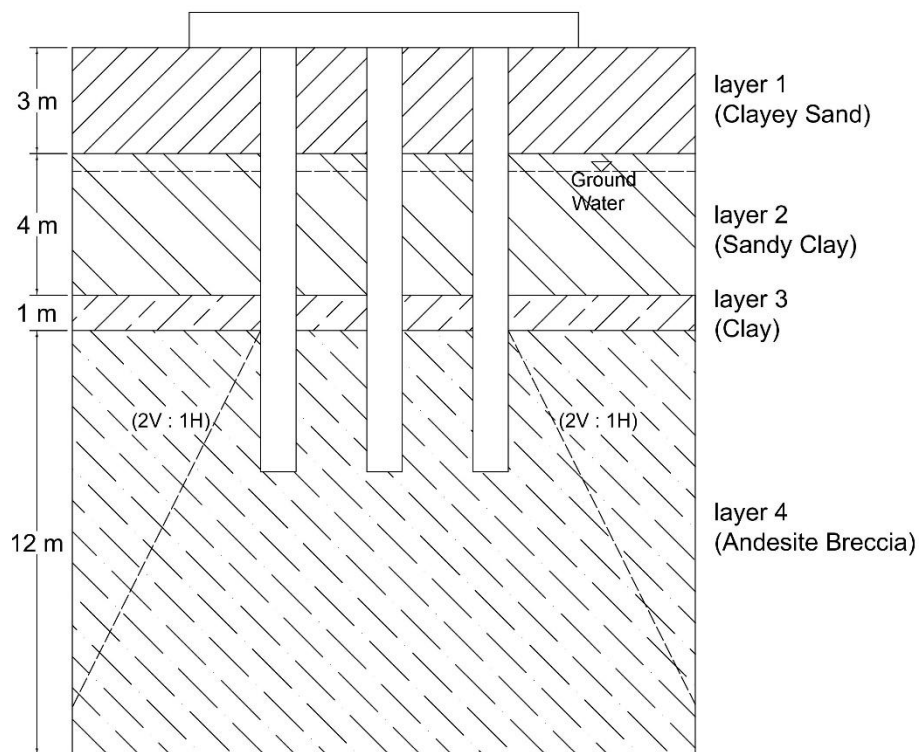


Figure 5.31 Consolidation Settlement for Reese and Wright Method Equation with 0,9m Dimension

To determine consolidation settlement can be calculated:

$$Q_g = 35150,39 \text{ kN}$$

$$B_g = 6,3 \text{ m}$$

$$L_g = 6,3 \text{ m}$$

$$\begin{aligned}\Delta\sigma' &= \frac{Q_g}{(L_g+z_1)(B_g+z_1)} \\ &= \frac{35150,39}{(6,3+6)(6,3+6)} \\ &= 23,218 \text{ kN/m}^2\end{aligned}$$

and,

$$\begin{aligned}\sigma'_o &= 3(15,892) + 4(17,805 - 9,81) + 1(16,363 - 9,81) + 12(22,455 - \\ &\quad 9,81) \\ &= 237,951 \text{ kN/m}^2\end{aligned}$$

So,

$$\begin{aligned}\Delta S_c &= \left(\frac{C_c \times H_1}{1+e_o}\right) \log \left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right) \\ &= \left(\frac{0 \times 12}{1+0,224}\right) \log \left(\frac{237,951+23,218}{237,951}\right) \\ &= 0,00906 \text{ m} = 9,06 \text{ mm}\end{aligned}$$

5.9.8 Consolidation Settlement for Reese and Wright Method in Dimension
Variations 0,95 m

To calculate consolidation settlement can be seen below.

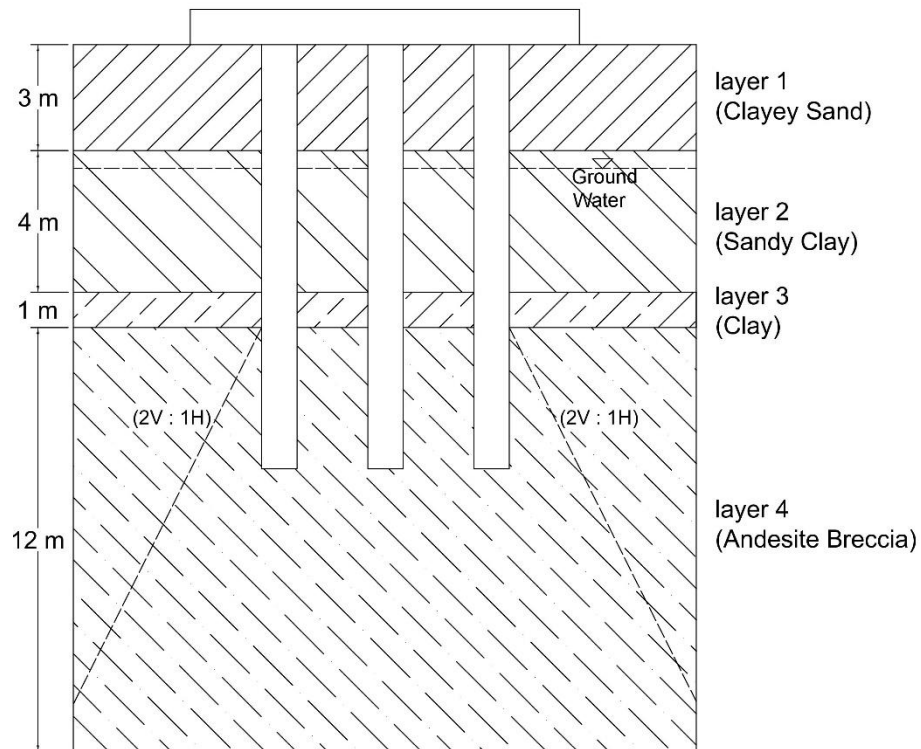


Figure 5.32 Consolidation Settlement for Reese and Wright Method Equation with 0,95m Dimension

To determine consolidation settlement can be calculated:

$$Q_g = 35150,39 \text{ kN}$$

$$B_g = 6,65 \text{ m}$$

$$L_g = 9,5 \text{ m}$$

$$\begin{aligned} \Delta\sigma' &= \frac{Q_g}{(L_g + z_1)(B_g + z_1)} \\ &= \frac{35150,39}{(9,5 + 6)(6,65 + 6)} \\ &= 17,915 \text{ kN/m}^2 \end{aligned}$$

and,

$$\begin{aligned} \sigma'_o &= 3(15,892) + 4(17,805 - 9,81) + 1(16,363 - 9,81) + 12(22,455 - 9,81) \\ &= 237,951 \text{ kN/m}^2 \end{aligned}$$

So,

$$\Delta S_c = \left(\frac{C_c \times H_1}{1 + e_o} \right) \log \left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o} \right)$$

$$= \left(\frac{0 \times 12}{1+0,224} \right) \log \left(\frac{237,951+17,915}{237,951} \right)$$

$$= 0,00706 \text{ m} = 7,06 \text{ mm}$$

5.10 Bearing Capacity of Bored Pile Foundation Using Plaxis 3D V20 Software

To perform analysis in the plaxis program, a laboratory test data is needed regarding the values and properties in each type of soil. the following soil data from the laboratory on the Tawang-Ngalang bridge and road construction project as in Table 5.28.

Table 5.29 Soil Properties

Index properties	Depth			
	0,00 m to -3,00 m	-3,00 m to -7,00 m	-7,00 m to – 8,00 m	-8,00 m to -20,00 m
Soil description	Clayey sand, radish browm	Sandy clay, brown	Clay, dark grey	Andesit breccia
Modulus Young, E_s (kN/m ²)	197,6	230,7	238,5	236180
Poisson number, V	0,3	0,3	0,3	0,3
Cohesion, c (kN/m ²)	10,787	19,613	18,633	-
Unsaturated volume weight, γ_{unsat} (kN/m ²)	15,215	13,616	10,762	21,837
Saturated volume weight, γ_{sat} (kN/m ²)	15,892	17,805	16,363	22,455
Material Model	Mohr–Coulomb	Mohr–Coulomb	Mohr–Coulomb	Hoek–Brown

(Source: PT. Geomine Bara Studio)

5.10.1 Analysis Based on Meyerhof method in Existing Conditions.

Analysis result from Plaxis 3D Software before earthquake load is applied showed the settlement value of 11,03 mm. As for the calculation of Plaxis 3D can be seen in figure 5.33.

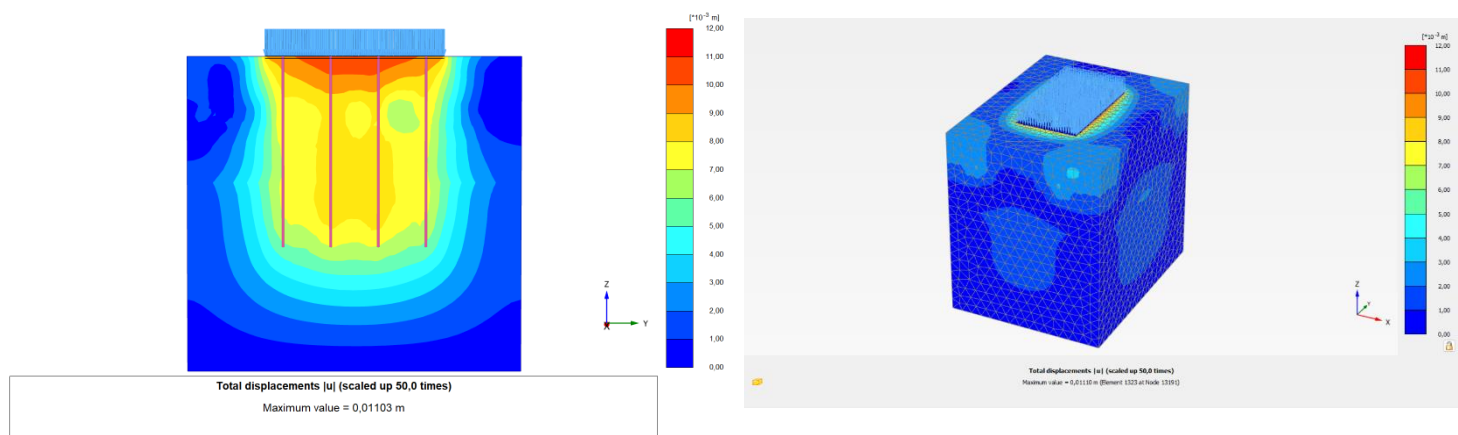


Figure 5.33 Results of a 1m Diameter Settlement Using Meyerhof Method Based on The Plaxis Software Before the Application Earthquake Load

Analysis result from Plaxis 3D Software after earthquake load is applied showed the settlement value of 336 mm. As for the calculation of Plaxis 3D can be seen in figure 5.34.

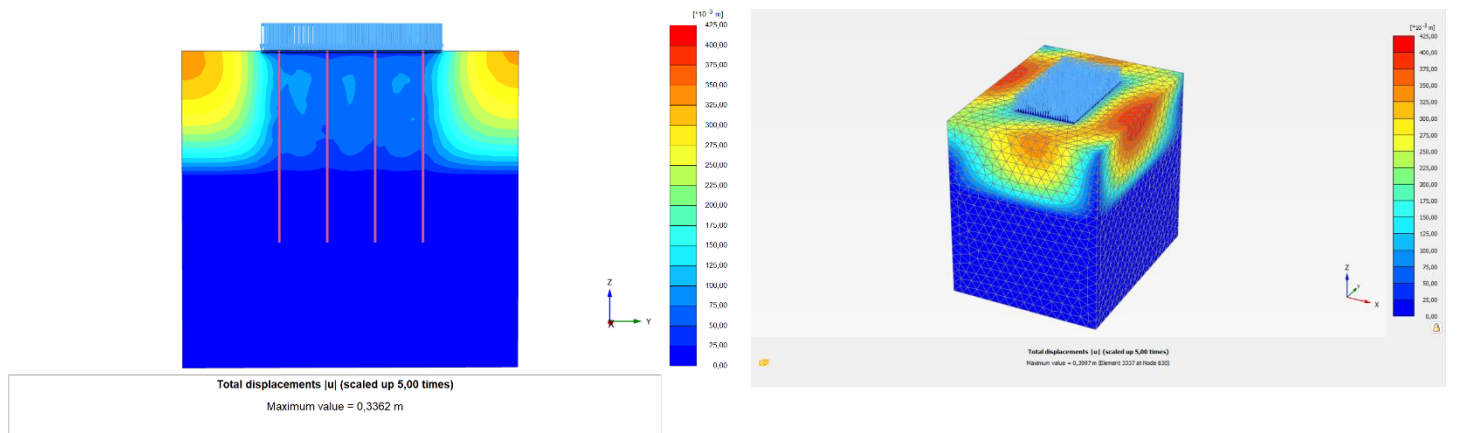


Figure 5.34 Results of a 1m Diameter Settlement Using Meyerhof Method Based on The Plaxis Software After the Application Earthquake Load

5.10.2 Analysis Based on Meyerhof Method in Dimension Variations 0,85 m.

Analysis result from Plaxis 3D Software before earthquake load is applied showed the settlement value of 15,54 mm. As for the calculation of Plaxis 3D can be seen in figure 5.35.

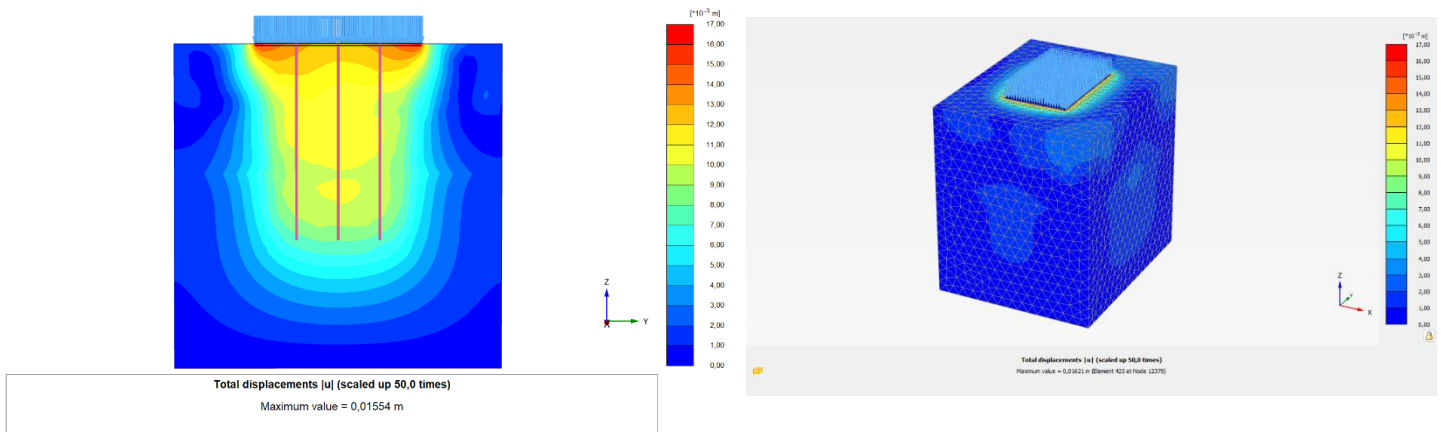


Figure 5.35 Results of a 0,85m Diameter Settlement Using Meyerhof Method Based on The Plaxis Software Before the Application Earthquake Load

Analysis result from Plaxis 3D Software after earthquake load is applied showed the settlement value of 350 mm. As for the calculation of Plaxis 3D can be seen in figure 5.36.

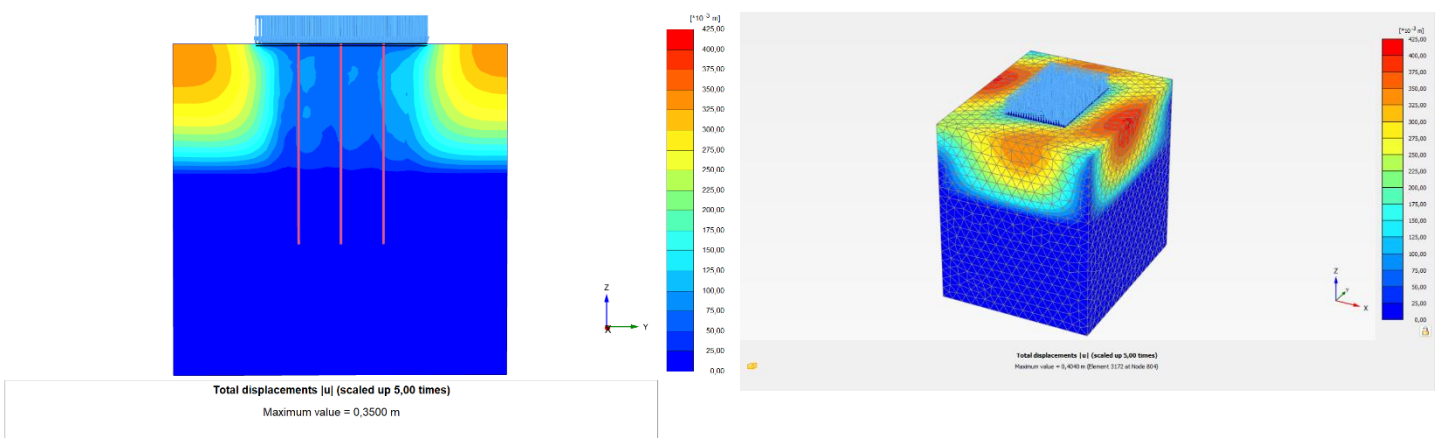


Figure 5.36 Results of a 0,85m Diameter Settlement Using Meyerhof Method Based on The Plaxis Software After the Application Earthquake Load

5.10.3 Analysis Based on Meyerhof Method in Dimension Variations 0,9 m.

Analysis result from Plaxis 3D Software before earthquake load is applied showed the settlement value of 13,63 mm. As for the calculation of Plaxis 3D can be seen in figure 5.37.

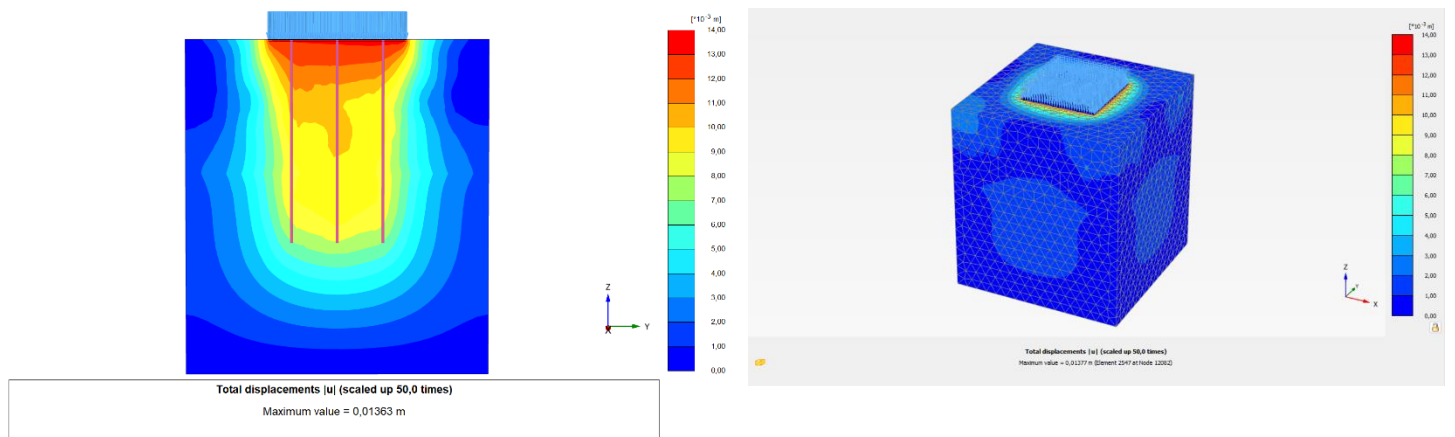


Figure 5.37 Results of a 0,9m Diameter Settlement Using Meyerhof Method Based on The Plaxis Software Before the Application Earthquake Load

Analysis result from Plaxis 3D Software after earthquake load is applied showed the settlement value of 365 mm. As for the calculation of Plaxis 3D can be seen in figure 5.38.

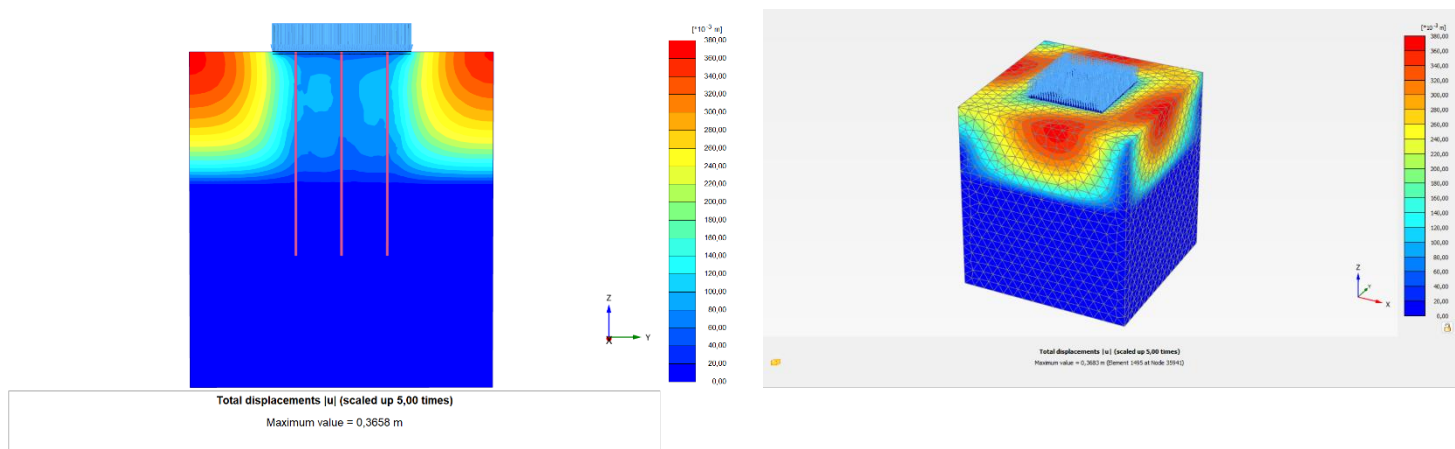


Figure 5.38 Results of a 0,9m Diameter Settlement Using Meyerhof Method Based on The Plaxis Software After the Application Earthquake Load

5.10.4 Analysis Based on Meyerhof Method in Dimension Variations 0,95 m.

Analysis result from Plaxis 3D Software before earthquake load is applied showed the settlement value of 15,17 mm. As for the calculation of Plaxis 3D can be seen in figure 5.39.

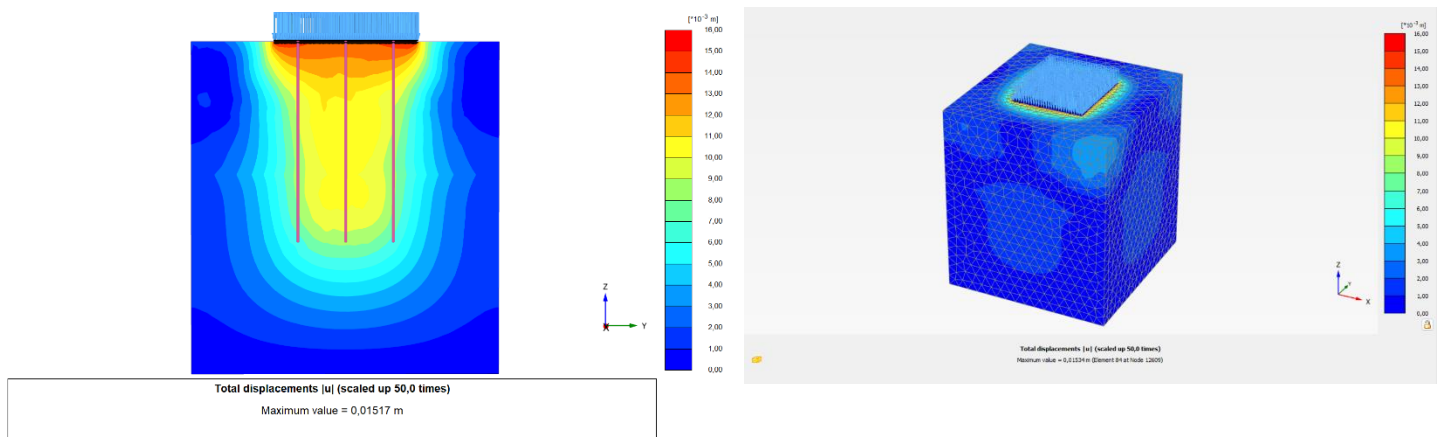


Figure 5.39 Results of a 0,95m Diameter Settlement Using Meyerhof Method Based on The Plaxis Software Before the Application Earthquake Load

Analysis result from Plaxis 3D Software after earthquake load is applied showed the settlement value of 365 mm. As for the calculation of Plaxis 3D can be seen in figure 5.40.

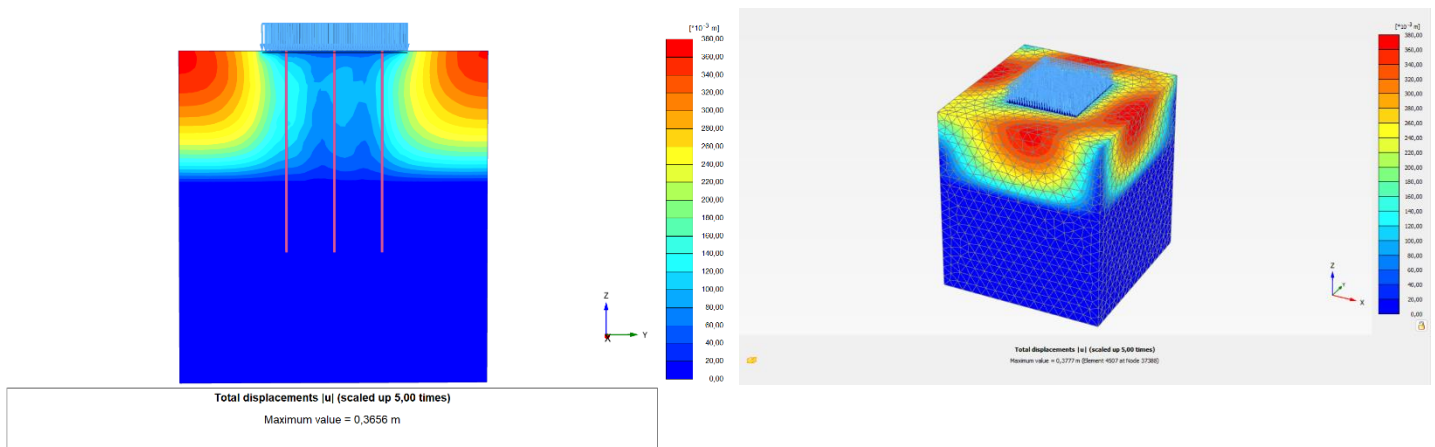


Figure 5.40 Results of a 0,95m diameter settlement using Meyerhof method based on the Plaxis Software after the Application Earthquake Load

5.10.5 Analysis Based on Reese and Wright Method in Existing Conditions.

Analysis result from Plaxis 3D Software before earthquake load is applied showed the settlement value of 11,27 mm. As for the calculation of Plaxis 3D can be seen in figure 5.41.

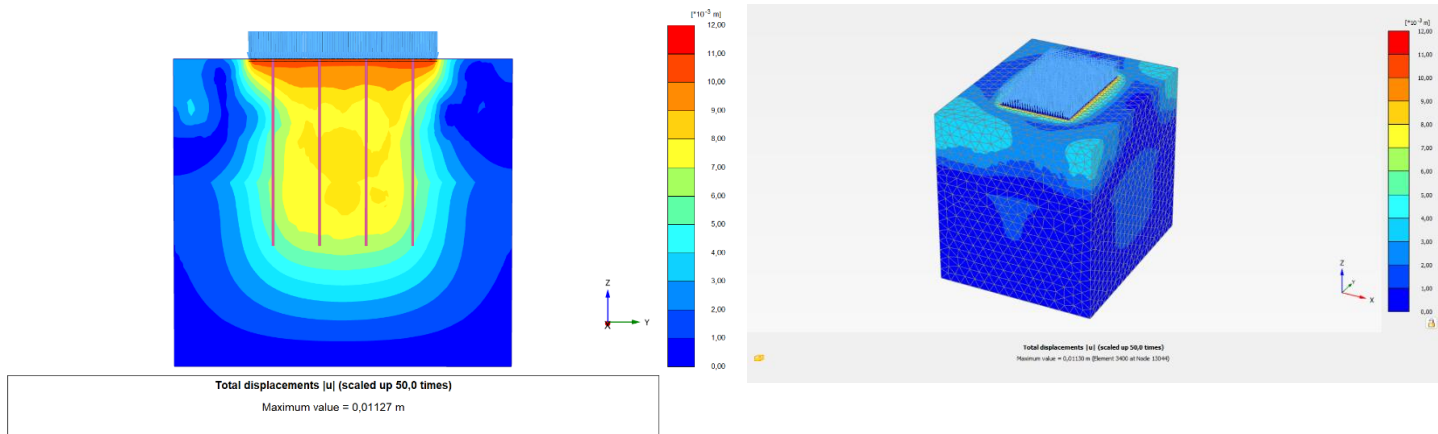


Figure 5.41 Results of a 1m Diameter Settlement Using Reese and Wright Method Based on The Plaxis Software Before the Application Earthquake Load

Analysis result from Plaxis 3D Software after earthquake load is applied showed the settlement value of 376 mm. As for the calculation of Plaxis 3D can be seen in figure 5.42.

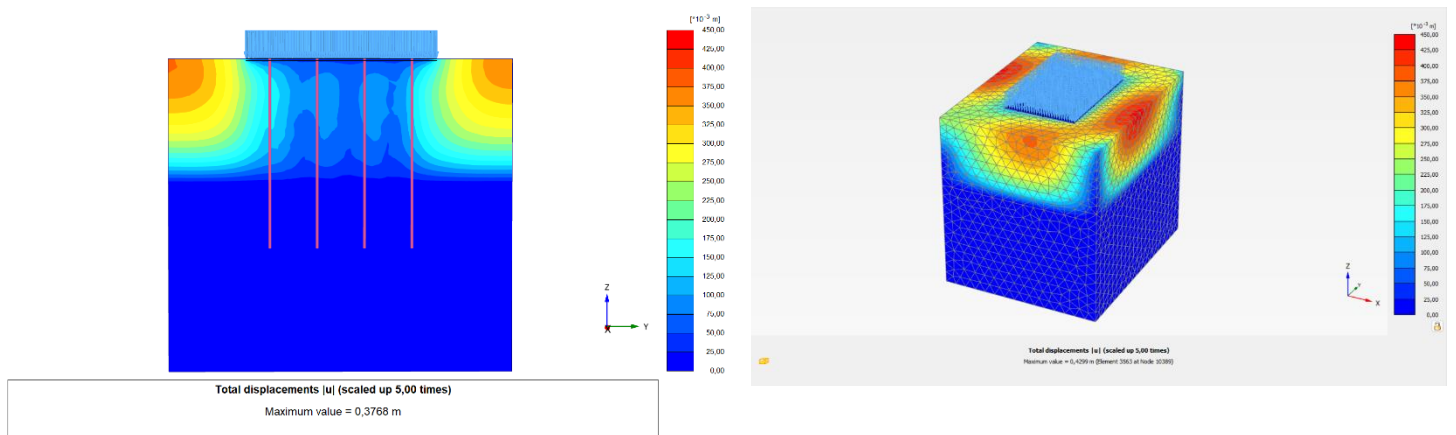


Figure 5.42 Results of a 1m Diameter Settlement Using Reese and Wright Method Based on The Plaxis Software After the Application Earthquake Load

5.10.6 Analysis Based on Reese and Wright Method in Dimension Variations 0,85 m.

Analysis result from Plaxis 3D Software before earthquake load is applied showed the settlement value of 10,5 mm. As for the calculation of Plaxis 3D can be seen in figure 5.43.

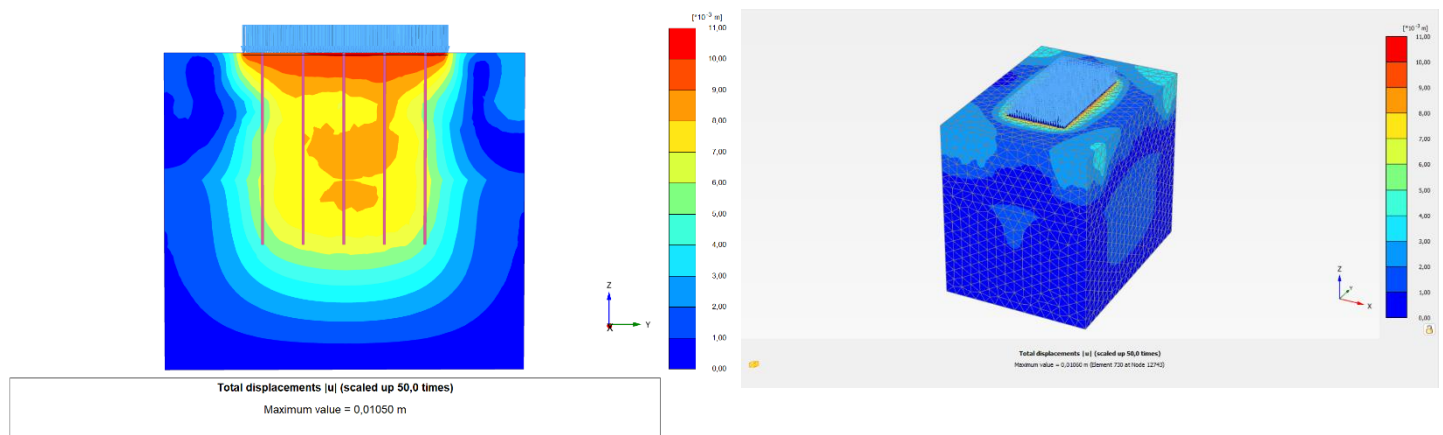


Figure 5.43 Results of a 0,85m Diameter Settlement Using Reese and Wright Method Based on The Plaxis Software Before the Application Earthquake Load

Analysis result from Plaxis 3D Software after earthquake load is applied showed the settlement value of 341 mm. As for the calculation of Plaxis 3D can be seen in figure 5.44.

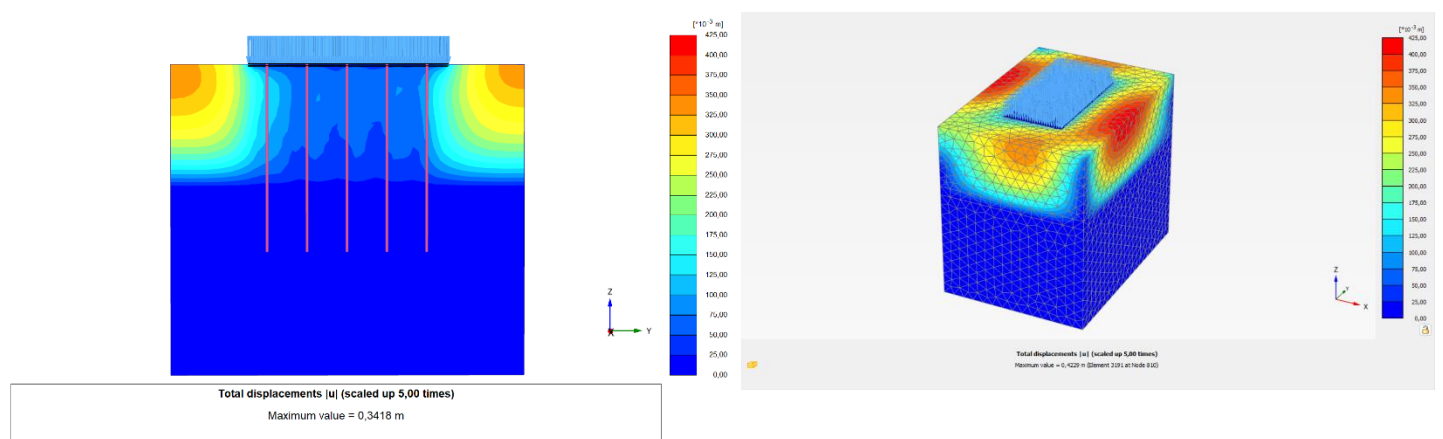


Figure 5.44 Results of a 0,85m Diameter Settlement Application Earthquake Load

5.10.7 Analysis Based on Reese and Wright Method in Dimension Variations 0,9 m.

Analysis result from Plaxis 3D Software before earthquake load is applied showed the settlement value of 10,46 mm. As for the calculation of Plaxis 3D can be seen in figure 5.45.

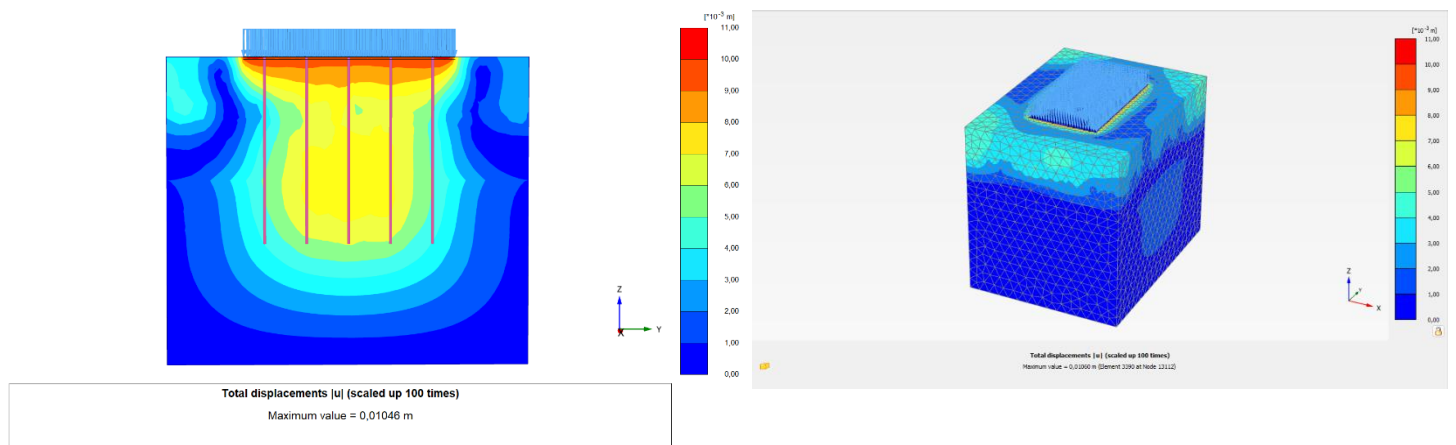


Figure 5.45 Results of a 0,9m Diameter Settlement Using Reese and Wright Method Based on The Plaxis Software Before the Application Earthquake Load

Analysis result from Plaxis 3D Software after earthquake load is applied showed the settlement value of 373 mm. As for the calculation of Plaxis 3D can be seen in figure 5.46.

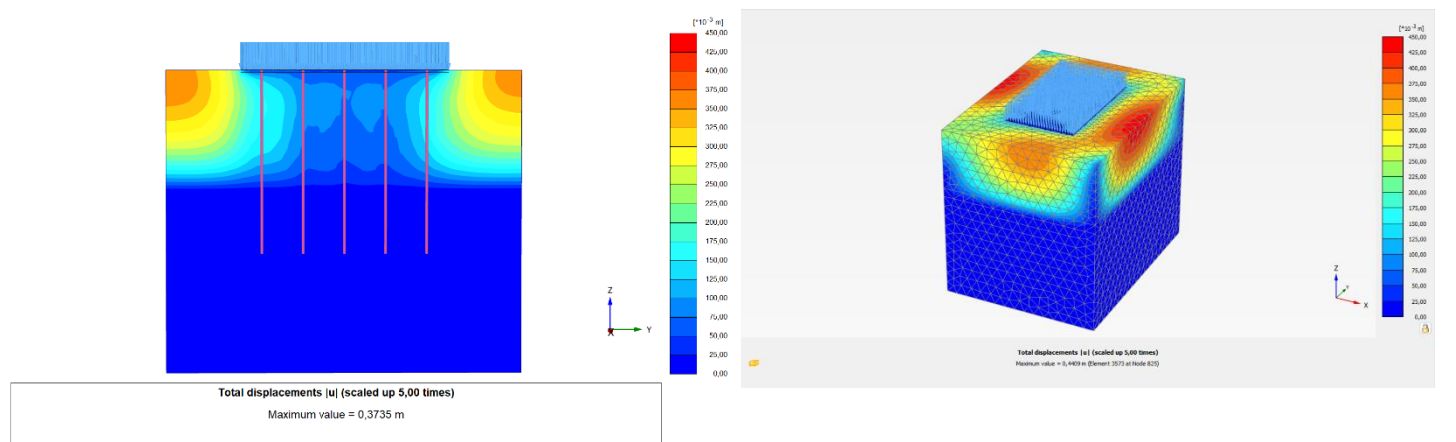


Figure 5.46 Results of a 0,9m Diameter Settlement Using Reese and Wright Method Based on The Plaxis Software After the Application Earthquake Load

5.10.8 Analysis Based on Reese and Wright Method in Dimension Variations 0,95 m.

Analysis result from Plaxis 3D Software before earthquake load is applied showed the settlement value of 10,27 mm. As for the calculation of Plaxis 3D can be seen in figure 5.47.

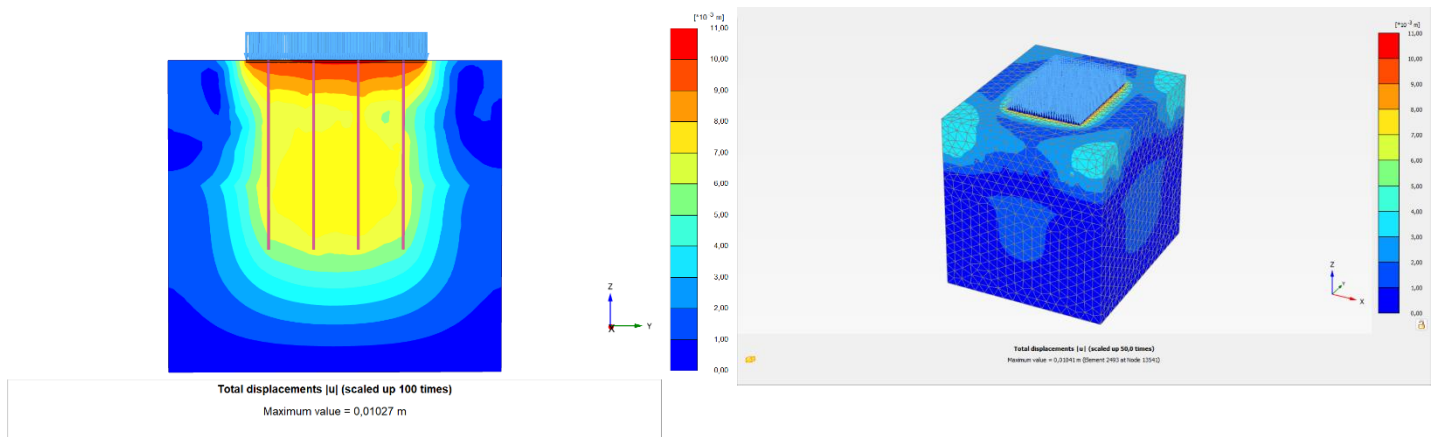


Figure 5.47 Results of a 0,95m Diameter Settlement Using Reese and Wright Method Based on The Plaxis Software Before the Application Earthquake Load

Analysis result from Plaxis 3D Software before earthquake load is applied showed the settlement value of 379 mm. As for the calculation of Plaxis 3D can be seen in figure 5.48.

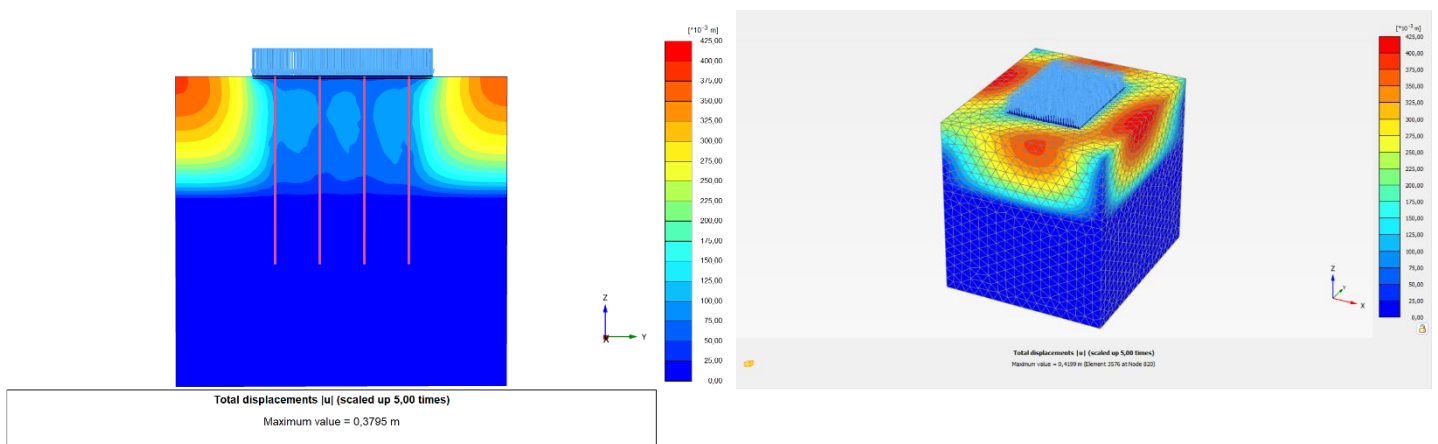


Figure 5.48 Results of a 0,95m Diameter Settlement Using Reese and Wright Method Based on The Plaxis Software After the Application Earthquake Load

5.11 Discussion

Building construction planning is inextricably linked to foundation planning. The foundation of a building is designed and constructed in such a way that it can withstand the loads occurring on it without compromising stability or causing significant deformations in the building and its surroundings. To address this issue, the foundation design does the following two things:

1. Fulfill the strength requirements, both for the foundation structure and for the the soil layer supporting the foundation (strength requirement).
2. Meet the specified settlement requirements (serviceability requirement).

To support this, it is necessary to pay attention to soil investigation data, loads carried by the foundation, pile dimensions, distance between piles, supporting data such as foundation depth and concrete quality.

5.11.1 Results of Bored Pile Foundation Bearing Capacity Analysis

The calculation of the bearing capacity of the bored pile foundation is compared between the dimensions of 0,85 m, 0,9 m, 0,95 m, and the existing dimension (1 m) to draw conclusions from the results of the pile foundation bearing capacity analysis. From the results obtained from previous calculations, the value of each type of foundation and its dimensions can be seen in Table 5.29 and Figure 5.49.

Table 5.30 Comparison of Results from Bearing Capacity of Single and Group Pile

Method	Number pile	Length Pile	Diameter	Q single	Q Group		Pu	Check
		m	m	T	T		T	
Meyerhof	10	12	0,85	379,136	3773,741	>	3512,699	Safe
	9	12	0,9	418,403	3747,676	>		Safe
	8	12	0,95	447,182	3561,474	>		Safe
	12	12	1	488,88	5836,876	>		Safe
Reese and Wright	14	12	0,85	269,268	3750,503	>		Safe
	13	12	0,9	290,350	3755,872	>		Safe
	12	12	0,95	269,268	3655,765	>		Safe
	11	12	1	327,207	3581,734	>		Safe

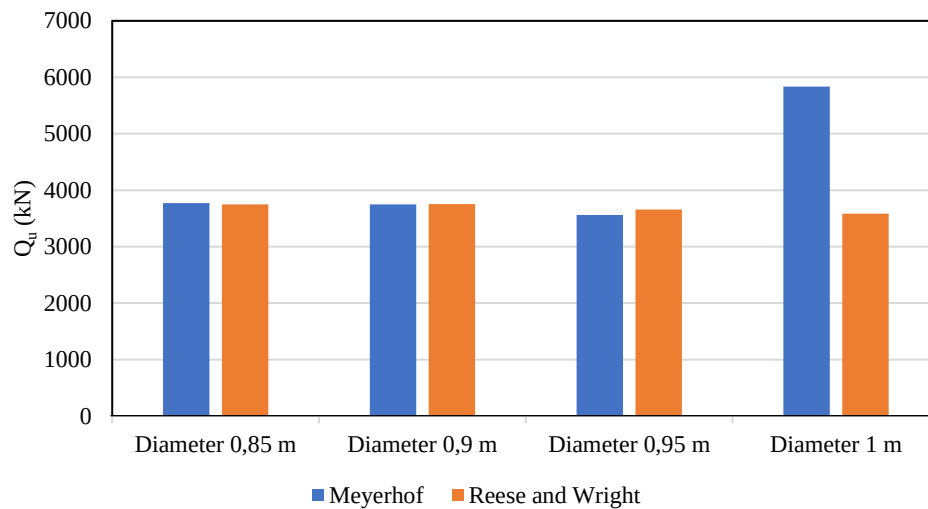


Figure 5.49 Comparison Chart of Results from Bearing Capacity of Group Pile

From the calculation results above, it can be concluded that the Meyerhof method and the Reese and Wright method are eligible.

5.11.2 Results of Bored Pile Foundation Elastic Settlement Analysis

The calculation of the bearing capacity of the bored pile foundation is compared between the dimensions of 0,85 m, 0,9 m, 0,95 m, and the existing dimension (1 m) to draw conclusions from the results of the pile foundation elastic settlement analysis. From the results obtained from previous calculations, the value of each type of foundation and its dimensions can be seen in Table 5.30, Figure 5.50, and Figure 5.51 below.

Table 5.31 The Comparasion of Settlement Value

Method	Number pile	Length Pile	Diameter	Elastic Settlement	Consolidation Settlement	Elastic Settlement from Plaxis 3D		Allowable settlement (SNI 8460:2017)	Check
		m	m	mm	mm	mm		mm	
Meyerhof	10	12	0,85	10	8,42	15,54	<	25	Safe
	9	12	0,9	11,592	9,06	13,63	<		Safe
	8	12	0,95	13,366	9,11	15,17	<		Safe

	12	12	1	14,866	7,45	11,03	<		Safe
Reese and Wright	14	12	0,85	7,145	8,42	10,5	<		Safe
	13	12	0,9	7,974	9,06	10,4	<		Safe
	12	12	0,95	8,735	7,06	10,27	<		Safe
	11	12	1	9,698	7,09	11,27	<		Safe

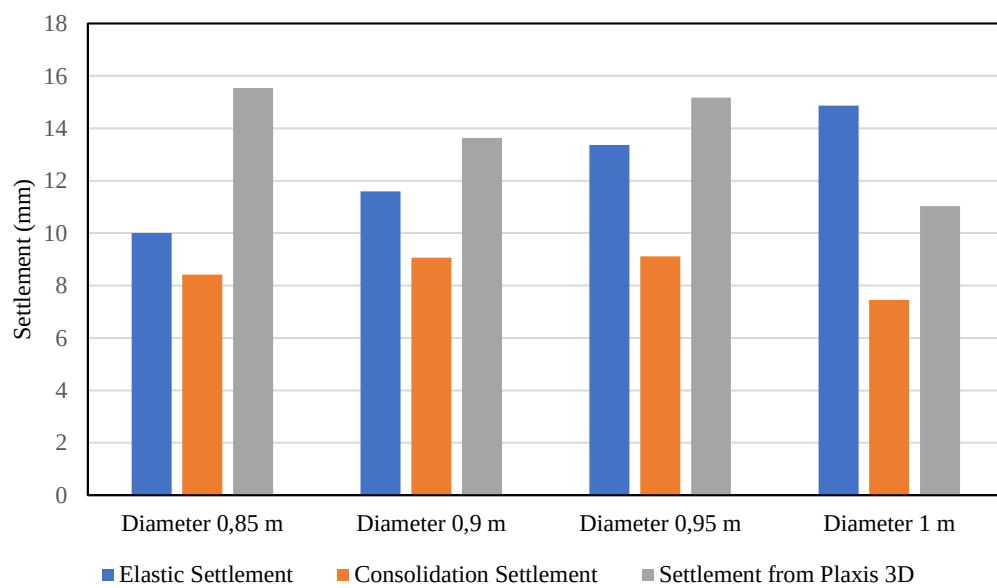


Figure 5.50 Comparison Chart of Results from Settlement on Meyerhof Method Equation

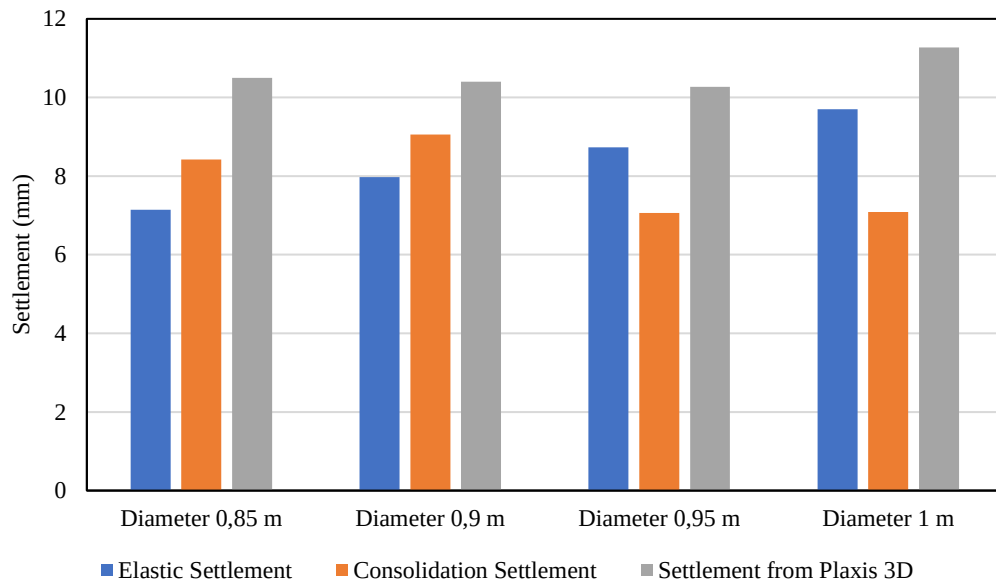


Figure 5.51 Comparison Chart of Results from Settlement on Reese and Wright Equation

From the results of the above calculations, it can be concluded that all elastic settlement calculations meet the criteria of SNI 8460:2017 where the maximum allowable settlement for foundations on bridges is 25 mm.

CHAPTER VI

CONCLUSION AND SUGGESTIONS

6.1 Conclusion

As for this research, the following conclusions were:

1. The results of bearing capacity calculation of the single pile on existing condition for 1 m in diameter, and 12 m in length using Meyerhoff and Reese and Wright methods are 488,88 Ton and 327,207 Ton.
2. The results of bearing capacity calculation of the single pile on variation condition for 0,85 m, 0,9 m, and 0,95 m in diameter, and 12 m in length Meyerhof method are 379,136 Ton, 418,403 Ton, and 447,182 Ton and with the Reese and Wright method are 269,268 Ton, 290,350 Ton, and 306,198 Ton.
3. The results of bearing capacity calculation of the group pile on existing condition for 1 m in diameter, and 12 m in length using Meyerhof and Reese and Wright methods are 5836,876 Ton and 3581,734 Ton.
4. The results of bearing capacity calculation of the group pile on variation condition for 0,85 m, 0,9 m, and 0,95 m in diameter, and 12 m in length Meyerhof method are 3773,741 Ton, 3747,676 Ton, and 3561,474 Ton and with the Reese and Wright method are 3750,503 Ton, 3755,872 Ton, and 3750,503 Ton.
5. The results of settlement from static method of the group pile on variation condition for 0,85 m, 0,9 m, 0,95 m, and 1 m in diameter, and 12 m in length Meyerhof method are 10,211 mm, 12,529 mm, 13,225 mm, and 14,467 mm and with the Reese and Wright method are 7,381 mm, 8,750 mm, 9,575 mm, and 10,015 mm.
6. The results of settlement from Plaxis 3D of the group pile on variation condition for 0,85 m, 0,9 m, 0,95 m, and 1 m in diameter, and 12 m in length

Meyerhof method are 15,54 mm, 13,63 mm, 15,17 mm, and 11,03 mm and with the Reese and Wright method are 10,5 mm, 10,4 mm, 10,27 mm, and 11,27 mm.

6.2 Suggestions

The suggestions that can be given by the author for further research are as follows:

1. Completeness of soil data from the study case will affect the accuracy of analysis result.
2. It is possible to analyze the bearing capacity of bored piles using other static and finite element methods, if the parameter needed for those calculation is available from the database and conducted at the same point of location.
3. The addition for number of piles from its original calculation is necessary to achieve safe condition for the foundation.

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APPENDIX

(STANDARD PENETRATION TEST (SPT) DATA)



PT. GEOMINE BARA STUDIO

MINING PROJECT VALUATION, MINING SERVICE, GEOLOGY RESOURCES & EXPLORATION, GEOTECHNICAL SERVICE

Head Office : Dusun Karanglo Grenjeng, RT 01/RT 01, Purwomartani, Kalasan, Kabupaten Sleman. Telp(0274) 4395538

Email : info@geomine-studio.com

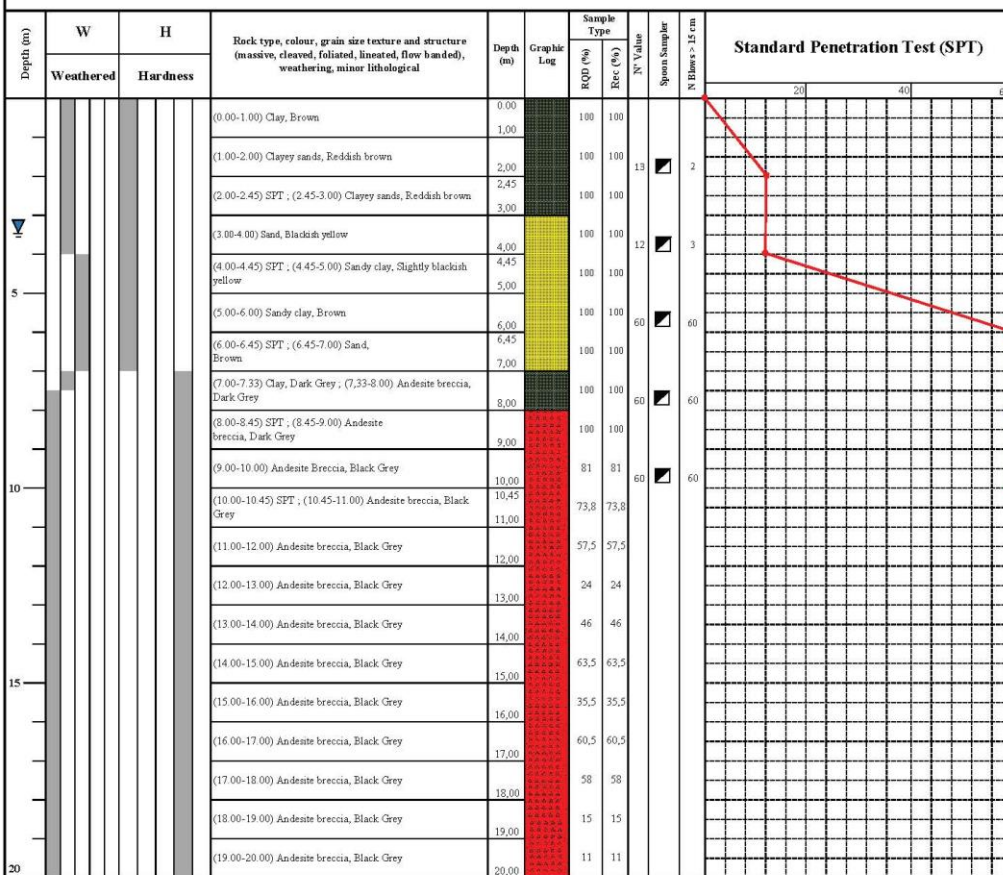
Website : www.geomine-studio.com

BORE HOLE INVESTIGATION SUBSURFACE OF GEOTECHNICS

PROJECT NAME : PEMBANGUNAN JALAN & JEMBATAN TAWANG-GALANG SEGMENT IV
HOLE ID : BH - 01
FIELD GEOLOGIST : ANDESTA GRANITO IRWAN
CHECKED BY : NOVANDRI KUSUMA WARDANA
LOCATION : RUAS JALAN TAWANG-NGALANG SEGMENT IV
GUNUNG KIDUL (KAB)
PELAKSANA : CV. CITRA MATARAM KONSTRUKSI (CMK)

COORDINATES X : 0450554
Y : 9130443
GROUND ELV. (m) : 198.95
DEPTH OF CASING (m) : -
DRILLING METHOD : Rotary Drilling

STARTED DATE : 22/03/2022
COMPLETED DATE : 24/03/2022
TOTAL DEPTH (m) : 20
GWT FROM GWL (m) : 3.5
INCLINATION : VERTICAL



REMARKS

- STANDARD PENETRATION SPLIT SPOON SAMPLER
- STABILIZED GROUNDWATER
- GROUNDWATER AT TIME OF DRILLING



Kepada Yth,
Pimpinan
PT Geomin Bara Studio
di
Tempat

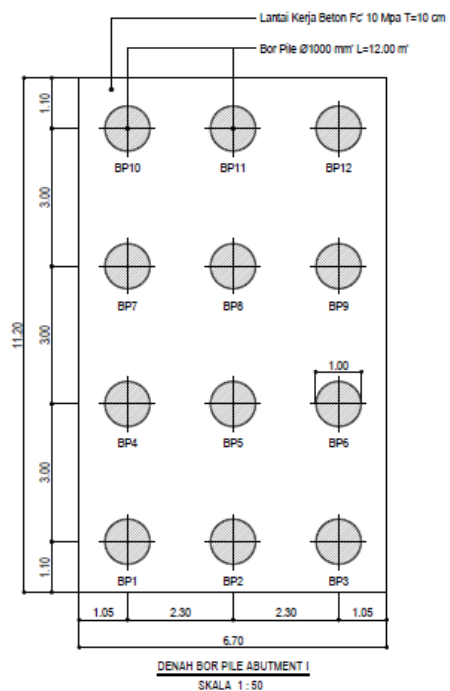
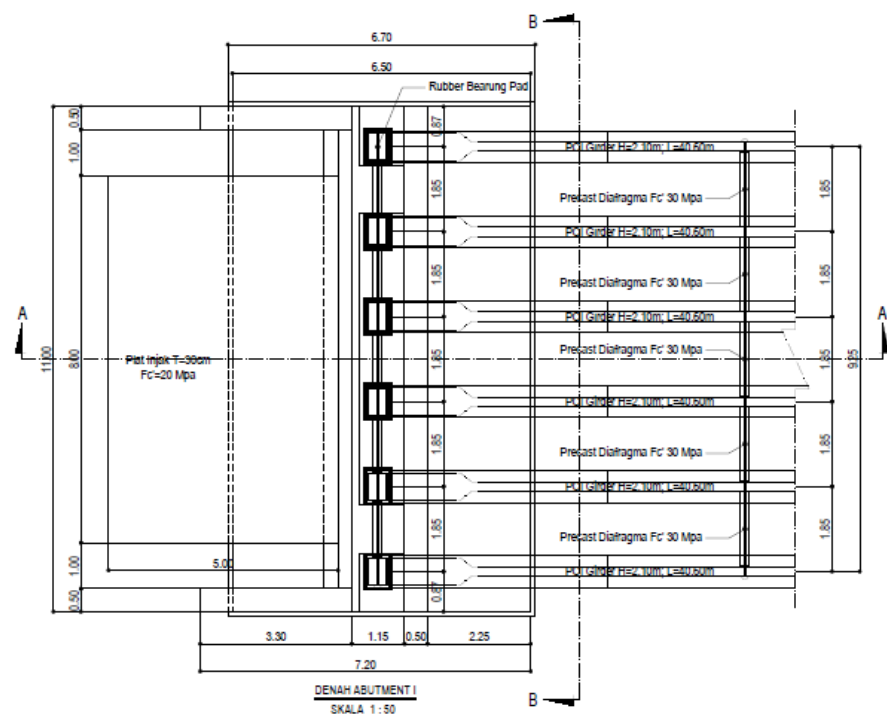
No	Parameter Uji	Salinan	Hasil Uji											
A. Soil Properties			D10	D10	D10	D10	D10	D10	D10	D10	D10	D10	D10	D10
			GF/TW07 BH	GF/TW03 BH	GF/TW02 BH	GF/TW01 BH	GF/TW04 BH	GF/TW05 BH	GF/TW06 BH	GF/TW09 BH	GF/TW08 BH	GF/TW10 BH	GF/TW11 BH	GF/TW12 BH
Soil Properties	Water sat. (Natural Water Content, w)	%	51.997	44.70	32.238	32.238	2.810	30.797	36.679	4.470	17.996	9.957	30.797	49.765
	Best fit's Ternary Specific Gravity of soil (Gs)		2.601	2.594	2.594	2.594	2.672	2.599	2.601	2.675	2.598	2.631	2.724	2.536
	Best fit's Ternary Initial Liquid Ratio (L)	g/cm ³	2.180	1.628	1.652	2.289	1.651	1.698	2.023	1.674	2.023	2.101	1.815	1.916
	Best fit's Ternary Initial Liquid Ratio (L)	g/cm ³	1.097	2.087	1.231	1.249	2.226	1.262	1.242	1.551	1.714	1.388	1.405	1.689
	Best fit's Ternary Initial Liquid Ratio (L)		1.371	1.078	1.077	1.077	0.224	1.117	1.092	0.678	0.560	0.898	0.939	0.503
	Porosity, n	%	57.63	19.53	51.88	51.88	18.28	52.76	52.20	40.40	35.90	25.30	47.25	48.42
	Porosity, n	%	57.63	19.53	51.88	51.88	18.28	52.76	52.20	40.40	35.90	25.30	47.25	48.42
	Densitas bulk (ρ _b)	g/cm ³	18.69	47.76	76.51	77.66	34.23	73.68	87.30	17.16	85.95	75.21	90.44	68.00
	Densitas bulk (ρ _b)	g/cm ³	18.69	47.76	76.51	77.66	34.23	73.68	87.30	17.16	85.95	75.21	90.44	68.00
	Densitas bulk (ρ _b)	g/cm ³	18.69	47.76	76.51	77.66	34.23	73.68	87.30	17.16	85.95	75.21	90.44	68.00
Grain size analysis	Karakteristik Kiri (Gravel part)	%	0.00	42.57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	Karakteristik Kiri (Sand part)	%	23.78	39.90	52.17	52.17	39.27	16.63	46.23	33.95	29.22	33.95	48.45	38.45
	Karakteristik Kiri (Silt part)	%	57.53	17.53	38.64	38.64	51.59	74.38	42.68	59.70	57.05	59.70	33.80	33.80
	Karakteristik Kiri (Clay part)	%	18.69	0.00	9.20	9.20	9.15	8.98	11.08	17.75	17.75	17.75	17.75	17.75
	D10 (mm)	mm	-	0.0282	0.0023	0.0023	0.0023	0.0026	0.0016	0.0013	0.0026	0.0013	0.0026	-
	D50 (mm)	mm	0.0077	0.0047	0.0184	0.0047	0.0059	0.0087	0.0088	0.0100	0.0046	0.0100	0.0046	0.0546
	D90 (mm)	mm	0.0405	0.0346	0.0639	0.0346	0.0520	0.0397	0.0259	0.0500	0.0626	0.0500	0.0626	0.0626
	Uniformity Coefficient, C _u		1.2266	27.8959	15.1484	15.1484	13.7998	15.1484	15.7241	39.3034	23.8492	39.3034	23.8492	23.8492
	cc	-	-	0.024	2.2889	2.2889	0.4739	0.7233	1.0767	1.3852	18.1700	1.3852	18.1700	-
	q _u (kN)	kg/cm ²	0.27	166.47	1.42	2.22	168.39	0.52	0.31	0.18	174.71	193.51	0.28	0.87
Engineering Properties	Friction angle (Triaxial test UU) φ	°	10.75	24.53	14.58	14.58	16.48	33.53	15.41	5.51	22.28	5.51	22.28	9.39
	Cohesion, c (Triaxial test UU) φ	kg/cm ²	0.19	0.53	0.15	0.15	0.01	0.34	0.11	0.20	0.48	0.20	0.48	0.24
	Koeff. Permeabilitas, c (constant head permeameter)	cm/det	-	2.14E-08	-	-	3.08E-08	-	-	2.23E-08	2.02E-08	2.23E-08	2.02E-08	1.57E-08
	Koeff. Permeabilitas, c (falling head permeameter)	cm/d												

Alas keperawatan dan kerjasamanya disampaikan terimakasih

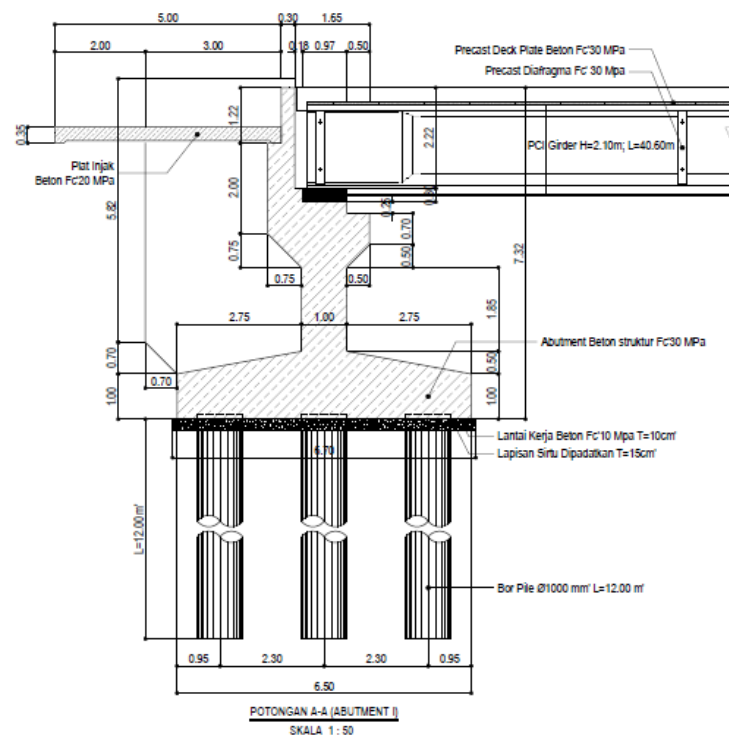
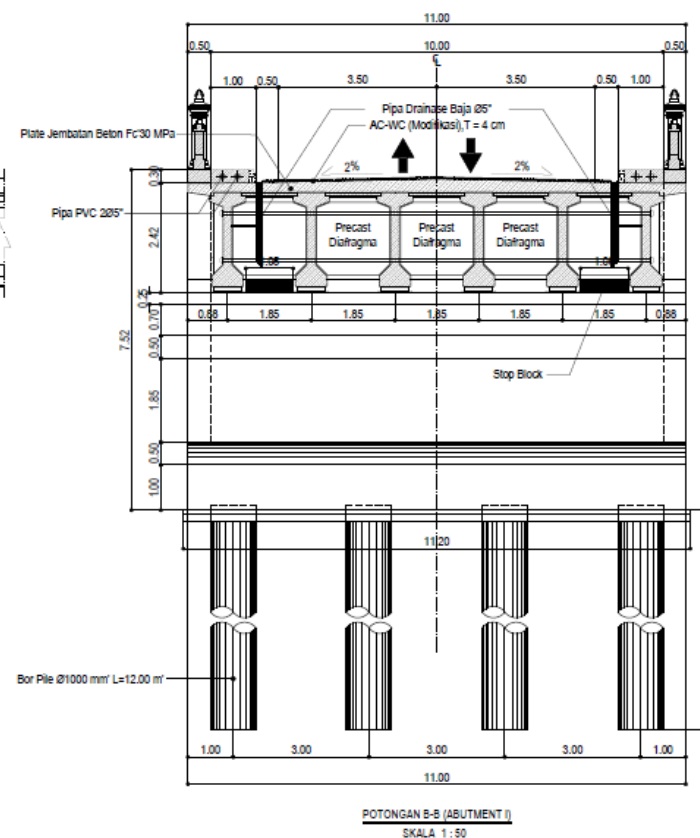
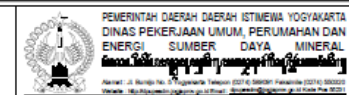


APPENDIX

(SHOPDRAWING PROJECT)



 PEMERINTAH DAERAH DAERAH ISTIMEWA YOGYAKARTA DINAS PEKERJAAN UMUM, PERUMAHAN DAN ENERGI SUMBER DAYA MINERAL <small>Alamat : Jl. Sekeloa No. 1, Yogyakarta, Telpom : 0271-528281, Faksimili : 0271-528222 E-mail : mda@yogyakarta.go.id / pda@yogyakarta.go.id</small>		
PROGRAM : PROGRAM PENYELENGGARAAN KEISTIMEWAAN YOGYAKARTA URUSAN TATA RUANG PEKERJAAN : PEMBANGUNAN RUAS JALAN DAN JEMBATAN TAWANG - NGALANG SEGMENT IV		
SHOP DRAWINGS		
GAMBAR :		
DETAIL ABUTMENT I		
DIAJUKAN : KONTRAKTOR PT. BSP - CMK, KSO		
ARYA AGUNG PRATOTO, ST MANAGER PROYEK		
DIPERIKSA & DISETUJUI : KONSULTAN PT. TRI PATRA		
ISWARINTA, ST SITE ENGINEER		
DIKETAHUI & DISETUJUI : PPK JASA KONSTRUKSI PADA SUB KEGIATAN PEMANFAATAN SATUAN RUANG STRATEGIS KARST GUNUNG SEMU		
ANDI KURNIAWAN DHARMA, ST., M.URP NIP. 19520727 201101 1 007		
SKALA :	NO LEMBAR :	JML LEMBAR :
1 : 50	02	39



PROGRAM :
PROGRAM PENYELENGGARAAN KEISTIMEWAAN
YOGYAKARTA URUSAN TATA RUANG
PEKERJAAN :
PEMBANGUNAN RUAS JALAN DAN JEMBATAN
TAWANG - NGALANG SEGMENT IV

SHOP DRAWINGS

GAMBAR :

POTONGAN ABUTMENT I

DIAJUKAN :
KONTRAKTOR
PT. BSP - CMK, KSO

ARYA AGUNG PRATOTO, ST
MANAGER PROYEK

DIPERIKSA & DISETUJUI :
KONSULTAN
PT. TRI PATRA

ISWARINTA, ST
SITE ENGINEER

DIKETAHUI & DISETUJUI :
PPK JASA KONSTRUKSI PADA SUB KEGIATAN
PEMANFAATAN SATUAN RUANG STRATEGIS
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1 : 50	03	39