DISSERTATION:

STABILITY RESEARCH OF RIVER EMBANKMENT ON SOFT GROUND USING TRADITIONAL REINFORCEMENT SYSTEM IN INDONESIA

インドネシアの軟弱地盤に構築する伝統的補強法を用いた河川堤防の設計法の提案

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Abstract

The ground near seashores or rivers in Indonesia is mostly covered by thick soft soil. In these areas, there are many houses and buildings standing on the soft soil and by the water. To prevent the natural disasters of flooding or big waves, local engineers have to construct tall and long embankments on the soft soil with low bearing capacity and stiffness.

In order to increase the bearing capacity of the soft soil, a traditional reinforcement method using timber piles has been widely used. However, the local design guidelines published by the Ministry of Public Works of Indonesia do not give details of the stability and settlement criteria, but recommend trial construction of a test embankment prior to the real design and construction.

In this study, a simple and practical calculation method using several classical ideas is proposed to obtain the ultimate bearing capacity, the factor of safety and the final settlement for cases with and without reinforcements. The effectiveness of the proposed method is evaluated based on trial embankments conducted in Indonesia and two-dimensional finite element analysis (FEA).

The following are the contents of this research. First of all, the background of this research is introduced, with the natural situation of the target area and technical problems in constructing reasonable countermeasures in Indonesia. After that, the objectives of this research are explained. Secondly, the proposed calculation scheme is explained using the conventional design for the traditional reinforcement system of soft clay. In this section, the required bearing capacity performance is established by the proposed method. Next, new design concepts are adopted within the traditional reinforcement system on thick soft clay using the proposed calculations. The proposed design is applied to determine the bearing capacity criterion, the height of the embankment and the settlement respectively.

It is found that the results from the proposed method can reproduce the bearing capacity and settlement of trial embankments qualitatively. In order to satisfy this research, it is proposed to use a small footing on a mattress overlying soft clay ground supported by timber piles. Then the calculation scheme is provided using an empirical method to establish a reasonable design rule for the required stability and settlement criteria for the embankment. In order to calculate the design criteria ofthe embankment for the traditional reinforcement, elastic and plastic material models are selected in the two-dimensional FEA.

Finally, the results of the empirical calculation and the FEA results are compared with the monitored data from a trial construction. Through these results, the proposed method is demonstrated to be very useful for Indonesian local engineers in charge of the design and construction of embankments on soft clay ground.

Keywords: Design criterion, factor of safety, embankment, soft clay, traditional reinforcement, timber pile.

Chapter 1. Introduction

1.1 Research Background

Flooding and wave attacks are common natural hazards that occuring Indonesia. People living around the seashore or along river banks have to construct high river dikes on soft soils to reduce the damage. However, river dikes usually have problems, mainly centred on natural and technical situations.

1.1.1 Typical natural problems

The typical natural situations in Indonesia are thick soft soils, floods, earthquakes, and wave attacks from seashores or rivers. According to the Ministry of Public Works, about 2,150 floods were recorded from October 1st, 2009 to February 28th, 2015 [1] and 1,566 of these occurred in the eastern parts of Sumatra, northern parts of Java, and southern parts of Kalimantan. Satibi (2009) studied the soft soil areas for all the islands of Indonesia. He plotted these areas in green as shown in the map [2]. The number of floods and the areas of soft soil in the islands of Sumatra, Java and Kalimantan are shown in Figure 1.1.

Figure 1.1 Numbers of floods and areas of soft soil in Indonesia [1, 2]

Floods and big sea waves are two types of natural hazards that frequently occur in all the Indonesian islands. These hazards have led to severe damage to the residential areas located along the coast as well as along the dikes of rivers. It is impossible to stop these natural hazards, but the local government can apply appropriate technologies in constructing dikes to mitigate the severe damage they can cause.

For this study, two cases of river dikes were used to explain the background, namely the dikes along the Tembilahan and Siak Rivers in Sumatra [3, 4].

In the case of Tembilahan River, the dike was built on soft soil to support the construction of a main access road for the local people in the sub-district.However, the dike collapsed due to the condition of the soil and frequent heavy rainfall, as shown in Figure 1.2 [3].

Figure 1.2 Tembilahan River dike after collapse [3]

A deep borehole sampling was conducted by the local government at two different points around the collapsed dike. These samples were used for triaxial compression, consolidation and Atterberg limit tests to investigate the soil properties. The results obtained are listed in Table 1.1 [3].

Table 1.1 Soil properties at the Tembilahan River

Depth (m)	Soil properties				Remark
	γ_s (kN/m^3)	c_u (kPa)	*PI* (%)	ϕ_s (°)	
0.0~6.0	14.8	18	52	3	Soft clay-1
6.0 ~ 21.0	16.0	10	83	3	Very soft clay-2
21.0 ~ 23.0	16.8	25	22	10	Mediumclay
23.0 ~ 30.0	17.2	5	-	30	Sand
30.0 ~ 45.0	17.5	23	33	6	Stiff

Table 1.1 shows the effect of the change of depth on the undrained cohesion of soil c_u, the plasticity index *PI* and the angle of internal friction. When the plasticity index *PI* is more than 22%, the angle of internal friction of the soil ϕ_s is very small at any depth of the clay layer. These soil layers include very soft clay to soft clay consistencies, with the undrained cohesion c_u less than 25 kN/m^2 [5].

Moreover, the local government also conducteda field investigation using a Standard Penetration Test (SPT). The results obtained are presented in Table 1.2 [3].

This river dike was reconstructed where the old dike had collapsed, over an area 300 m long and 53 m wide. The construction design adopted was a concrete plate bridge supported byconcrete piles.

Table 1.2 The N-SPT tests results at Tembilahan River dike

Depth (m)	N-SPT	Remark
0.0~6.0	1	Softsoil
6.0 ~ 21.0	1 ~ 3	Very soft soil
21.0 ~ 23.0	15 ~ 27	Medium
23.0 ~ 30.0	18 ~ 21	Sand
30.0 ~ 45.0	> 43	Stiff

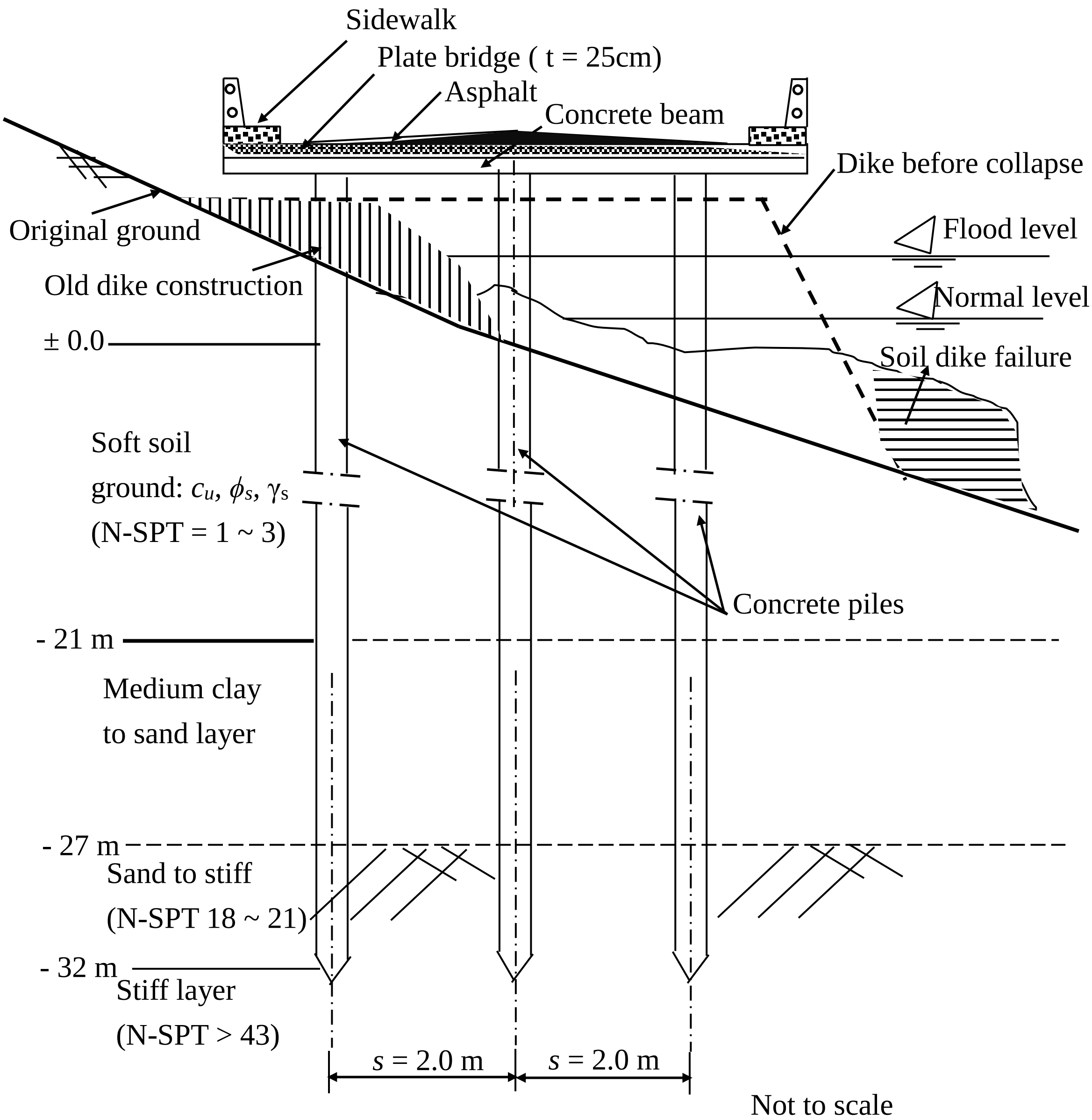

Figure 1.3 Cross-section of the typical design of a concrete plate bridge [3]

Figure 1.3 shows the typical design of the concrete plate bridge constructed on the Tembilahan River dike. The concrete pile installation was arranged in as quare pattern with a pile diameter *d* of 600 mm, and pile spacing *s* of 2 m perpendicular to the river dike and 5 m parallel to the dike [3].

In the case of the Siak River dike, the local government recommended the reconstruction of a dike 600 m in length to overcome flood hazards and big wave attacks from the adjacent river. This led to the provision of a design with the height of the dike to be 5m from the ground surface (GS) as shown in Figure 1.4 [4].

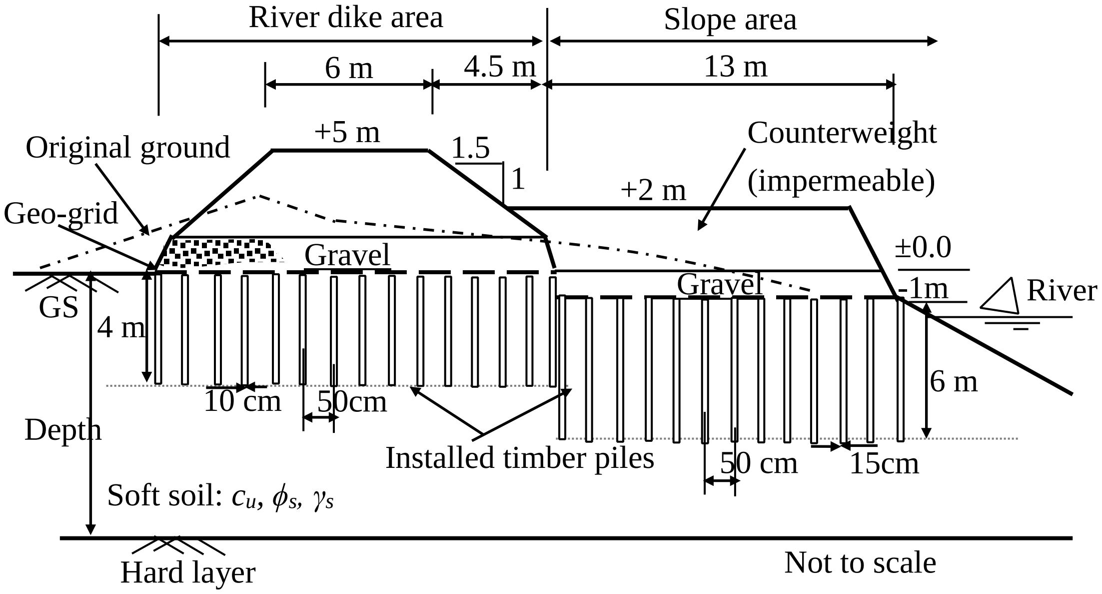

Figure 1.4 Cross-sectional design of Siak River dike [4]

Figure 1.4 shows the properties of the soft soil including the unit weight γ_s of the soil, undrained cohesion c_u and the angle of internal friction ϕ_s of the soil. The embankment was designed on soft soil using the traditional reinforcement method involving

(i) installing timber piles to increase the stiffness of the soft soil as the foundation,

(ii) laying geo-grid on the timber piles to reinforce the gravel layer,

(iii) spreading and compacting gravel on the geo-grid as a mattress to reinforce and distribute the load from the embankment, and

(iv) constructing an embankment on the mattress for the dike.

The Cone Penetration (CPT) and Standard Penetration (SPT) tests were conducted to determine the soil consistency such as cone resistance q_{nc}, and N-SPT down to a depth of 20 m. The results obtained are shown in Table 1.3 [4].

Table 1.3 The CPT and N-SPT tests results at Siak River

Depth(m)	CPT, q_{nc} (kg/cm^2)	N-SPT	Remark
0.0~ 2.1	3 ~ 5	1 ~ 4	Soft soil
2.1 ~ 10.2	2 ~ 3	0 ~ 1	Very soft soil
10.2 ~ 12.0	5 ~ 50	2 ~ 7	Soft soil
12.0~ 20.0	50 ~ 250	> 34	Stiff

Triaxial compression and Atterberg limit tests were conducted to determine the properties of the soil. The results obtained are listed in Table 1.4 [4].

Table 1.4 Soil properties at Siak River

Depth (m)	Soil properties				Remark
	γ_s (kN/m^3)	c_u (kPa)	*PI* (%)	ϕ_s (°)	
0.0~2.1	17	22	64.5	3	Soft clay
2.1 ~ 10.2	11.0	9	83.5	7	Very soft clay
10.2 ~ 12.0	15	14	29.5	9	Soft clay
12.0 ~ 20.0	20	0	-	25	Stiff

Table 1.4 shows the results of the plasticity index *PI*, undrained cohesion c_u of 25 kN/m^2 and the angle of internal friction ϕ_s, in which c_u was less than 25 kN/m^2 [4, 5].

The results of these investigations of dikes for Tembilahan and Siak River showed that they were constructed on soft clay [3, 4]. The consistency of the undrained cohesion c_u with the undrained shear strength of soil s_u can be expressed based on the Mohr–Coulomb failure criterion in Figure 1.5.

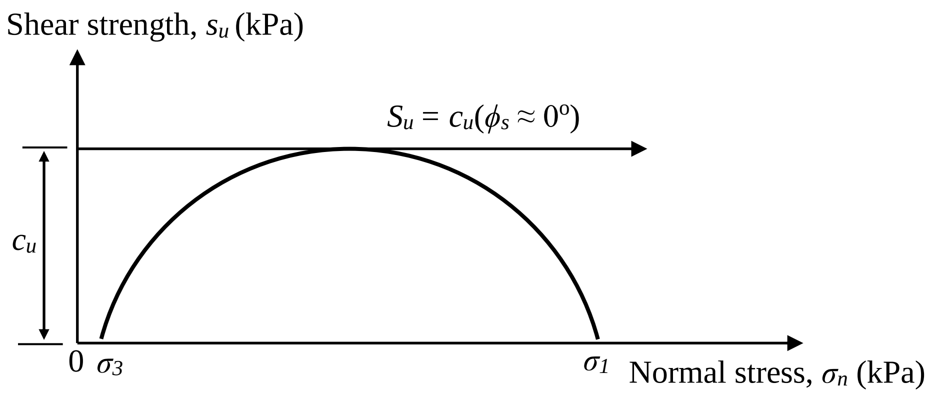

Figure 1.5 Mohr–Coulomb failure criterion of soil using triaxial compression test

Figure 1.5 shows σ_1 as the major stress and σ_3 the minor stress in the undrained triaxial compression test. Further calculations proposed in the next sections may involve the use of this undrained shear strength of soft clay s_u.

1.1.2 Technical situation of reinforced soft soil

To understand the technical situation of the reinforcement of soft soil, a traditional construction method is first introduced. To increase the bearing capacity of soft soil, a traditional reinforcement method using timber piles or bamboo has been widely used for embankments in Indonesia. These piles are called *Cerucuk*-piles locally in Indonesian.

Indonesian guidelines have been published by the Ministry of Public Works for embankment construction using the traditional reinforcement method [6, 7]. The steps in the procedure for embankment construction on soft soil for roads in the guidelines are explained in Figures 1.6 to 1.10 [6. 7].

(1) Cutting the surface of the ground.

Figure 1.6 shows the cutting of the original ground, where it is planned to have the same thickness for the gravel layer D_m and the width of the bottom surface B_b for the embankment. The groundwater table (GWT) is set as the same as the surface of the ground after cutting (GS).

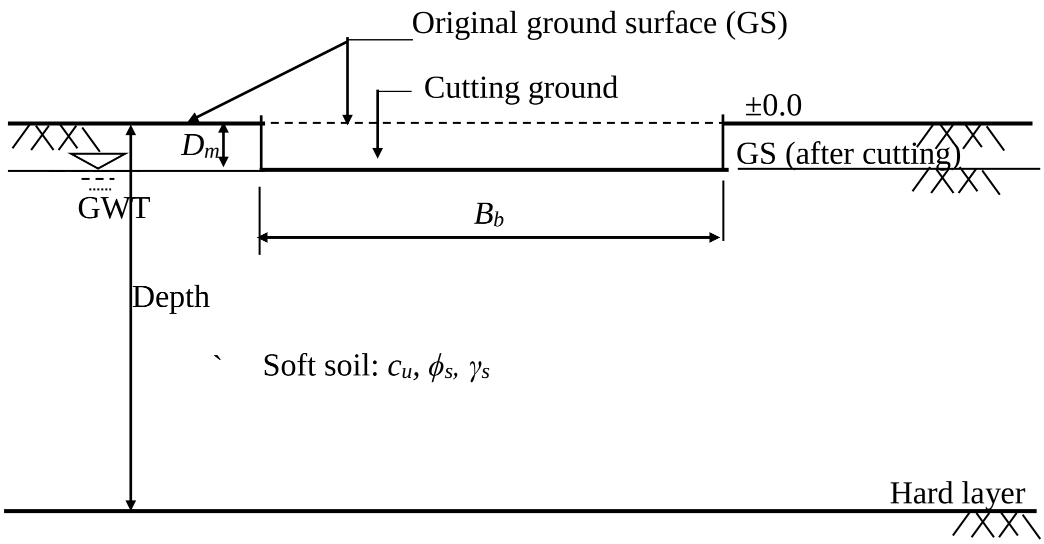

Figure 1.6 Cutting the ground surface

(2) Installing timber piles into soft soil

Figure 1.7 shows the manual installation of timber piles to reinforce the soft soil, arranged in a square pattern. Local material is available to use for piles, which are 6 m long with a diameter d of 8~15 cm.

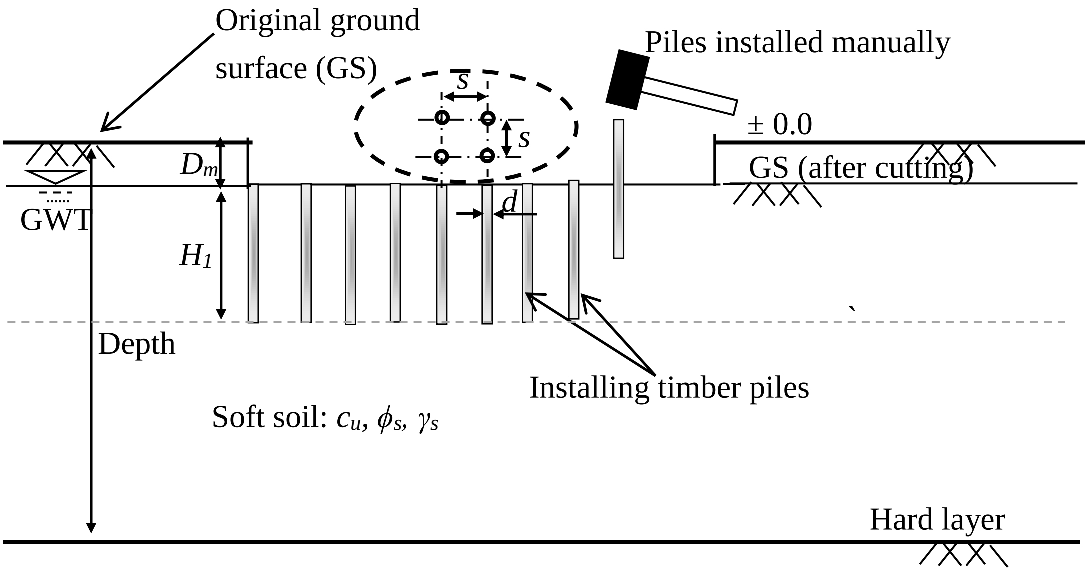

Figure 1.7 Installing timber piles on the soft soil

(3) Laying geo-synthetic on top of the timber piles

Figure 1.8 shows the geo-synthetic laid on top of the timber piles and each tips of the geo-synthetic leaves free about 1 m for each side. The geo-synthetic is used to reinforce the gravel layer. There are two options for using geo-synthetics: (i) geo-textile which is for small size gravel, and (ii) geo-grid for large size gravel.

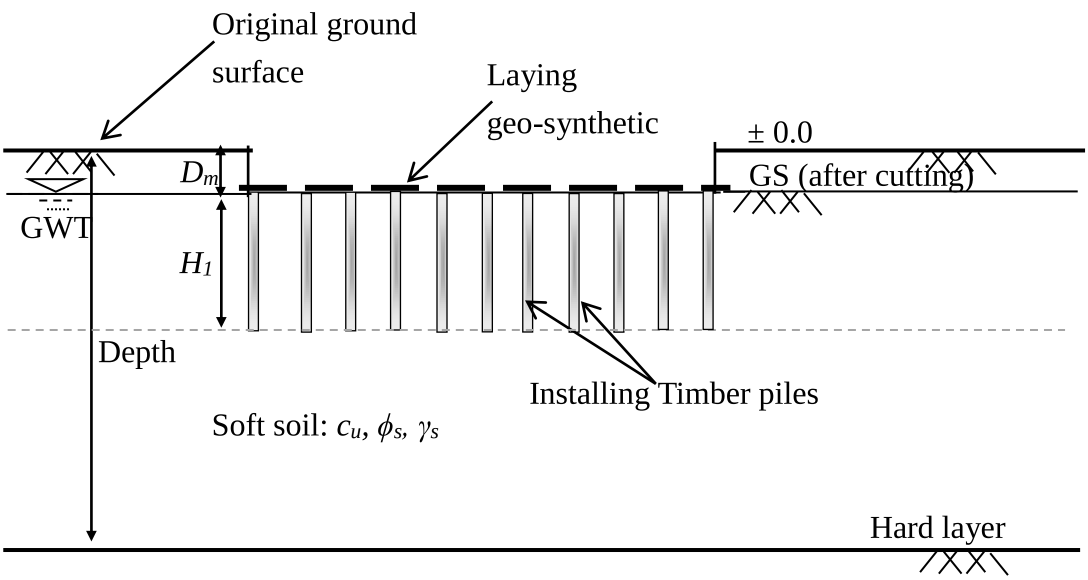

Figure 1.8 Laying geo-synthetic on top of the timber piles

(4) Spreading and compacting gravel

Figure 1.9 shows the spreading and compacting of gravel on the geo-synthetic. The combination of the geo-synthetic and gravel layer for reinforcing the embankment is called the mattress in the guidelines.

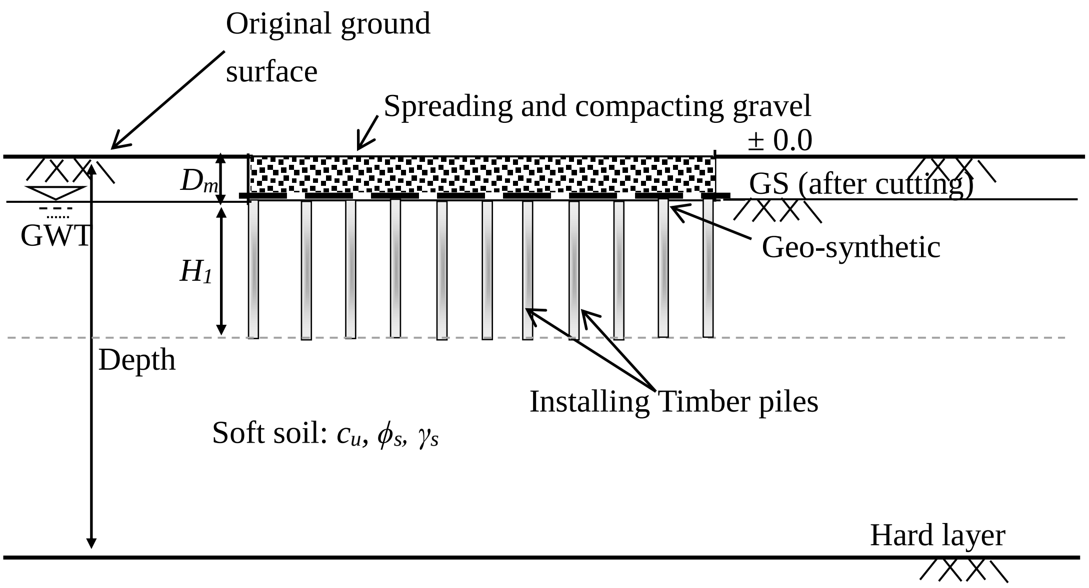

Figure 1.9 Spreading and compacting gravel for mattress

(5) Filling up embankment.

Finally, an embankment is constructed on the soft soil using the traditional reinforcement method. These traditional reinforcement materials provide the following dimensions and strength parameters:

(i) Installation of timber piles of length H_1 for reinforced soft soil;

(ii) Mattress construction with thickness D_m for distributing load from the embankment, which is reinforced by geo-synthetic with tensile strength T and angle of friction ϕ_r between the mattress and geo-synthetic; and

(iii) The dimensions using the embankment design are height H_b, gradient of slope n, width at the top of the embankment B_a and width at the bottom of the embankment B_b as shown in Figure 1.10.

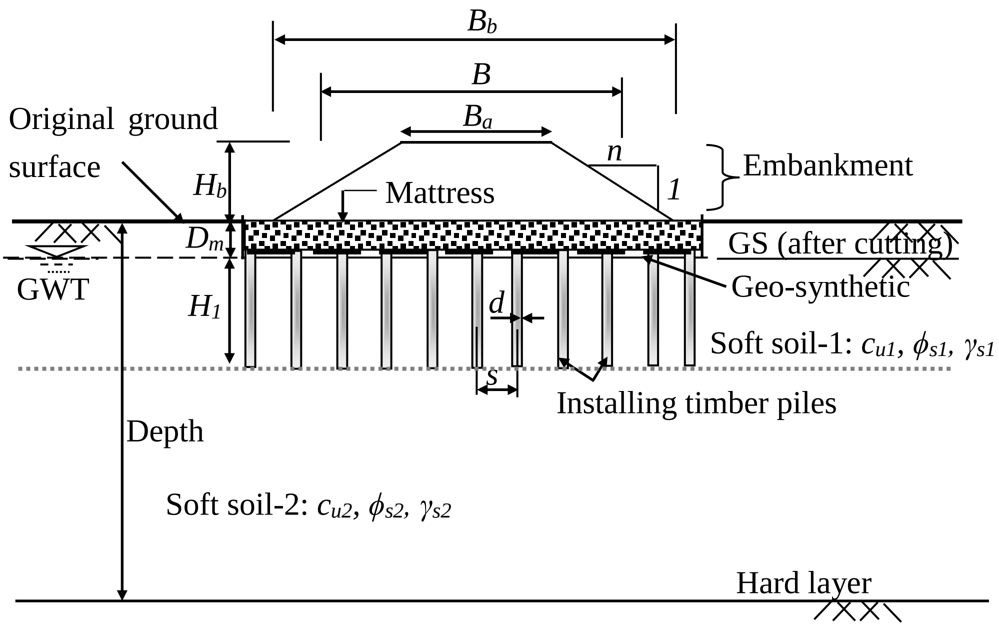

Figure 1.10 Cross-section of embankment on the mattress with underlying soft soil supported by timber piles [6, 7]

Figure 1.10 shows the parameters such as the unit weight of submerged soil layer 1 γ_{s1},the unit weight of soil layer 2 γ_{s2}, the unit weight of water γ_w, the average width of the embankment at mid-height B, the unit weight of embankment γ_b, the gradient of slope of embankment n, the length of pile H_1, the equivalent diameter of the timber pile and the diameter of a single timber pile d.

These dimensions and parameters of the embankment construction procedure and the stability and settlement criteria are briefly explained in the guidelines.

In respect to the technical situation, the idea of increasing the bearing capacity of soft soil using bamboo piles Q_{up} has been studied by the Ministry of Public Works [8]. In their final report, full-scale plate loading tests in the field (ASTM-D-1194-1972) were carried out at three sites in West Java (Karawang, Cirebon, and Banjar). The investigation found that the increase of the bearing capacity of soft soil with bamboo pile reinforcements compared with the original soft soil was 315% at the Karawang site, 242% at Cirebon and 215% at Banjar. These experimental results were proposed for a river dike, road and building constructions respectively [8]. However, the bearing capacities obtained can not be applied to other sites, because the condition of soft soil varies at each site.

In order to determine the appropriate reinforcement of soft soil, the guidelines recommend constructing a full-scale trial embankment and give an outline of the design technique for using a trial embankment for road construction. In these guidelines, the cases of two trial models are explained briefly.

(1) For the first model, a road embankment construction is planned on peat soil with a preloading method without any reinforcement [9]. To determine the stability of the embankment, a stability analysis for a factor of safety is conducted using the limit equilibrium method. This conventional design method is considered to determine the stability of embankment criterion.

(2) For the second trial, a shallow foundation of soft soil using cement and timber piles for road embankment construction was introduced [10]. Generally, with mixed soil–cement materials for road construction on an embankment, the height of the embankment is limited to 3 m without pile reinforcement. However, for this construction in the guidelines, the height required for the embankment on soft soil is 3 m or more. So, soil–cement mixing with varying thicknesses and timber or bamboo piles were applied for the required stability and settlement during construction. When the monitored settlement rate is within the rangein the guidelines, the engineer can continue monitoring to check the settlement. Finally, using the monitored data, finite element analysis (FEA) is conducted to simulate the settlement criteria for embankment design. But, it is too difficult for local engineers to conduct the FEA and the recommended process leads to a loss of time and money. Usually, in order for local engineers to establish the design criteria, the currently used design with the traditional method of reinforcement for embankments on soft soil is obtained from a trial construction on site [10].

The traditional reinforcement method for soft soil is still popularly used by Indonesian local engineers and local government. Therefore, an appropriate calculation scheme is required to enable the engineers to design the stability and settlement criteria. In order to prevent natural disasters, a reasonable design rule of construction is required to achieve a good bearing capacity and minimal settlement. However, the criteria for construction to check the stability and the settlement are given without any explanation in the guidelines. The annual settlement rate of an embankment through consolidation is examined to determine whether to design theconstruction using a rigid or a flexible pavement for the road.

The embankment is required to satisfy the performance criteriafor the particular kind of road, so roads constructed on embankments are grouped as (i) a rigid pavement construction for higher importance roads, and (ii) a flexible pavement construction for lower importance roads.

The criterion for settlement of the embankment in the guidelinesis that it must be less than 20 mm/year for higher importance roads and less than 30 mm/year for lower importance roads [8]. The stability of the embankment also requires a factor of safety *Fs* of 1.40 for higher importance roads and 1.30 for lower importance roads [7].

1.2 Research Objectives

These research objectives are to present a geotechnical solution for the appropriate stability and settlement performance ofa river dike on soft clay ground with the traditional reinforcement system. The aims of the research using an empirical calculation method are to be achieved by taking the following things into consideration:

(1) a mattress constructed on top of the timber piles installation to reinforce and to distribute loads from the embankment, and

(2) an embankment constructed on the mattress, which is supported by timber piles.

A reasonable design rule based on geotechnics for design criteria for the traditional reinforcement method can be provided by an empirical calculation scheme. The first study is to identify the load-spreading within the mattress from the surface of the ground, which is calculated by the friction angle of load-spreading. The mattress considered is both unreinforced and reinforced by timber piles, and loaded by a small width of footing [11, 12].

This empirical calculation is applied by using the dataset of the Siak River dike construction. However, the monitored deformation data is not available to compare the design application (see Figure 1.4).

The second study is to propose an empirical calculation scheme using a large width of footing of the embankment.

The dataset from a previous trial construction of an embankment at East Kalimantan, Indonesia, is adopted to evaluate the proposed calculation method in the design application [13]. In order to have the required bearing capacity and settlement for the embankment, a trial embankment is constructed by using the traditional reinforcement method.

The author uses FEA to simulate the stability of the embankment and to check the settlement beneath the embankment for the trial construction dataset mentioned above. However, in the case of trial construction in East Kalimantan, a mattress did not used. The construction procedure of the embankment on soft clay using a mattress for the traditional reinforcement system is required in the guidelines (see Figures 1.6 to 1.10).

Then the simulation result obtained from the FEA is used for comparison with the empirical calculation method. Finally, the stability and settlement criteria of an embankment on soft clay reinforced by geo-textile and timber piles can be presented.

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Chapter 2. Proposed Empirical Calculations

2.1 Conventional Method of Traditional Reinforcement

Local governments have been building river dikes on soft soil using the traditional reinforcement system in Indonesia [1, 2]. The procedure for embankment construction on soft soil has been introduced in the guidelines [3, 4]. The design method for road embankment construction on soft soil using the traditional reinforcement system includes the recommendation by the Ministry of Public Works to perform a trial construction, and that includes monitoring of the settlement at the mattress beneath the embankment [5].

In order to propose an empirical calculcation method using the traditional reinforcement system for embankments, the steps in the procedure for the trial construction for the conventional design of embankments are expressed in Figures 2.1 to 2.5 [3~5].

(1) Cutting the surface of the ground

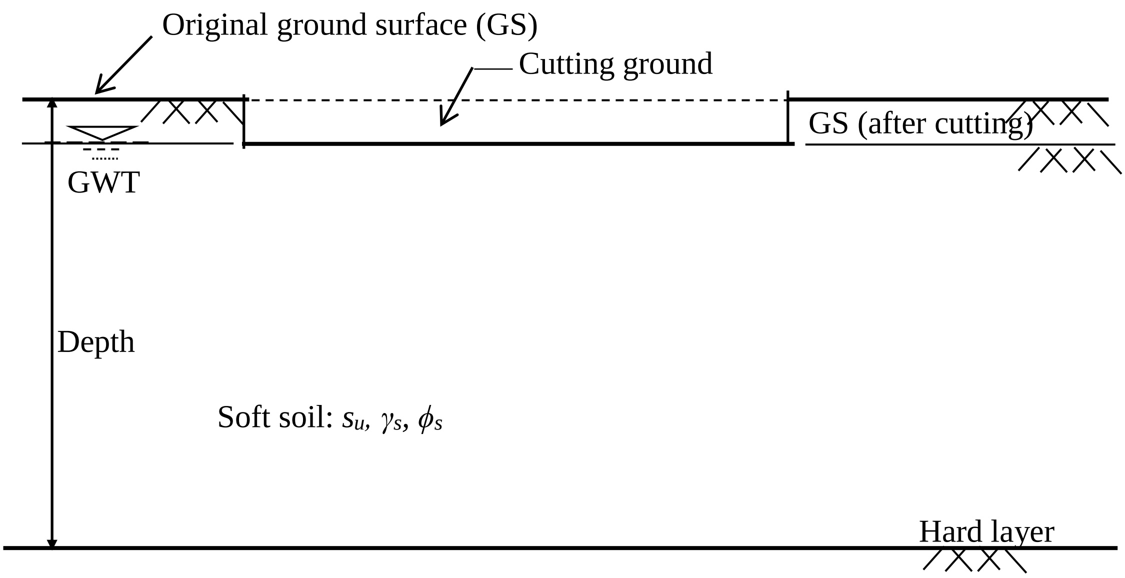


Figure 2.1 Cutting the ground surface

(2) Installing timber piles into soft soil

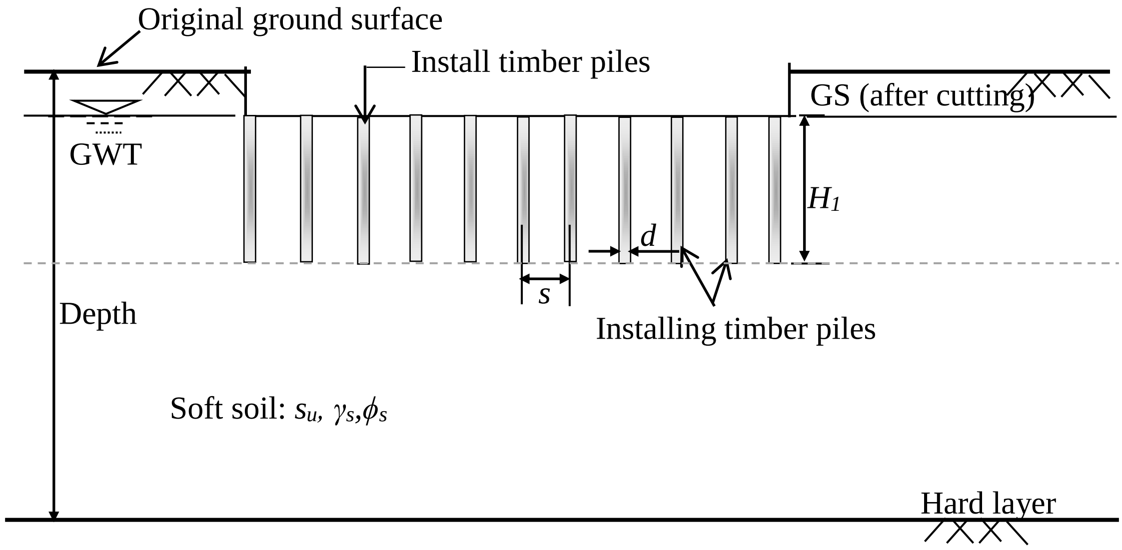


Figure 2.2 Installing timber piles in the soft soil

Figure 2.2 shows the situation of pile installation, where H_1 is the length of the piles embedded in the soft soil, s is the spacing of the piles, d is the diameter of the piles, GWT is the groundwater table and GS is the ground surface after cutting.

(3) Laying geo-synthetic on top of the timber piles

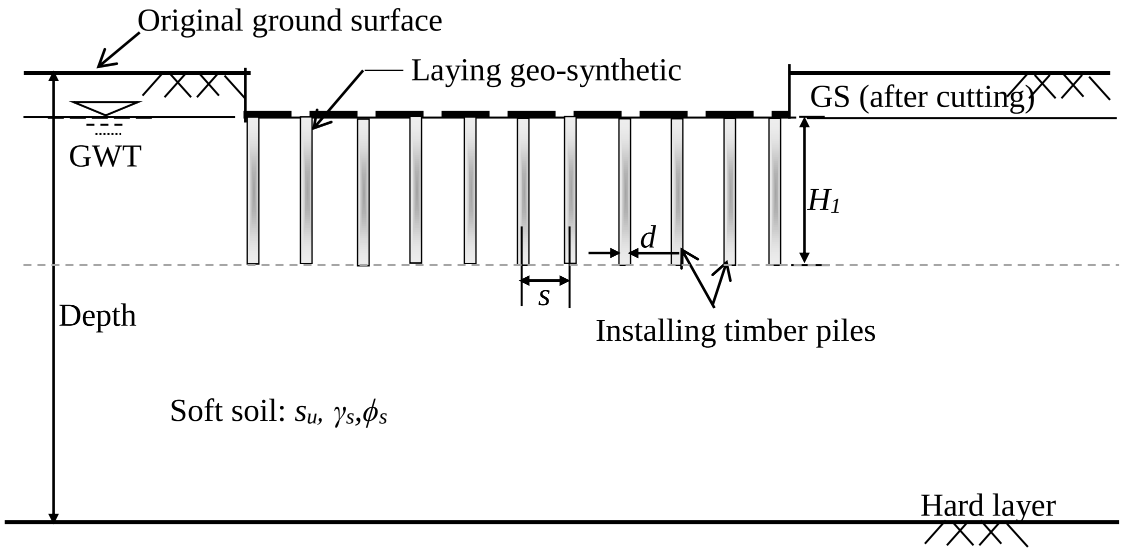


Figure 2.3 Laying geo-synthetic on top of the timber piles

(4) Spreading and compacting gravel

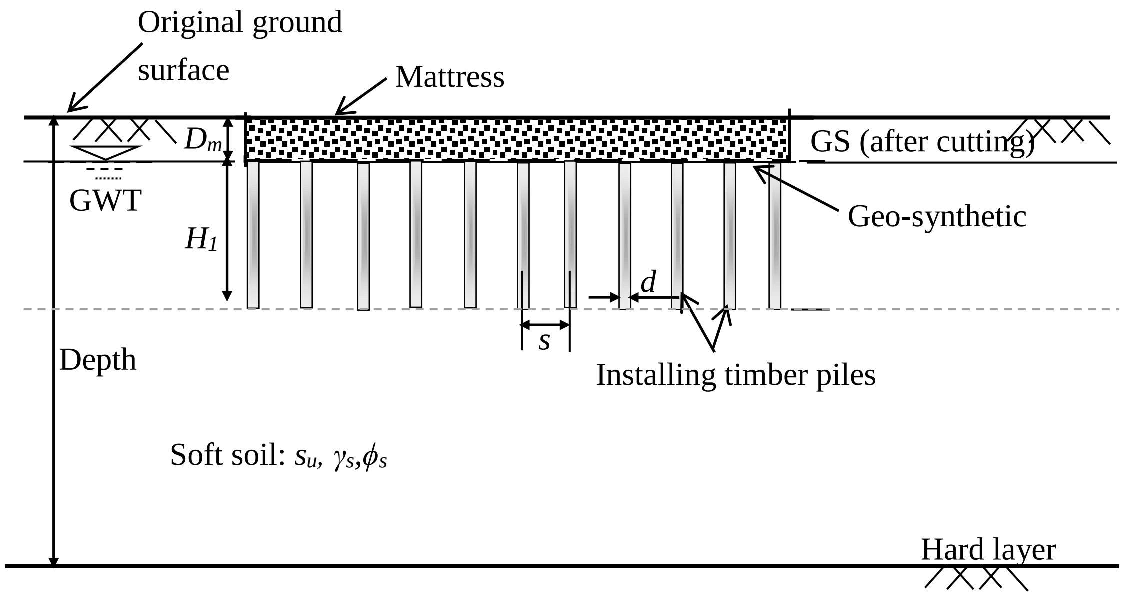


Figure 2.4 Spreading and compacting gravel for mattress

(5) Constructing embankment on the mattress

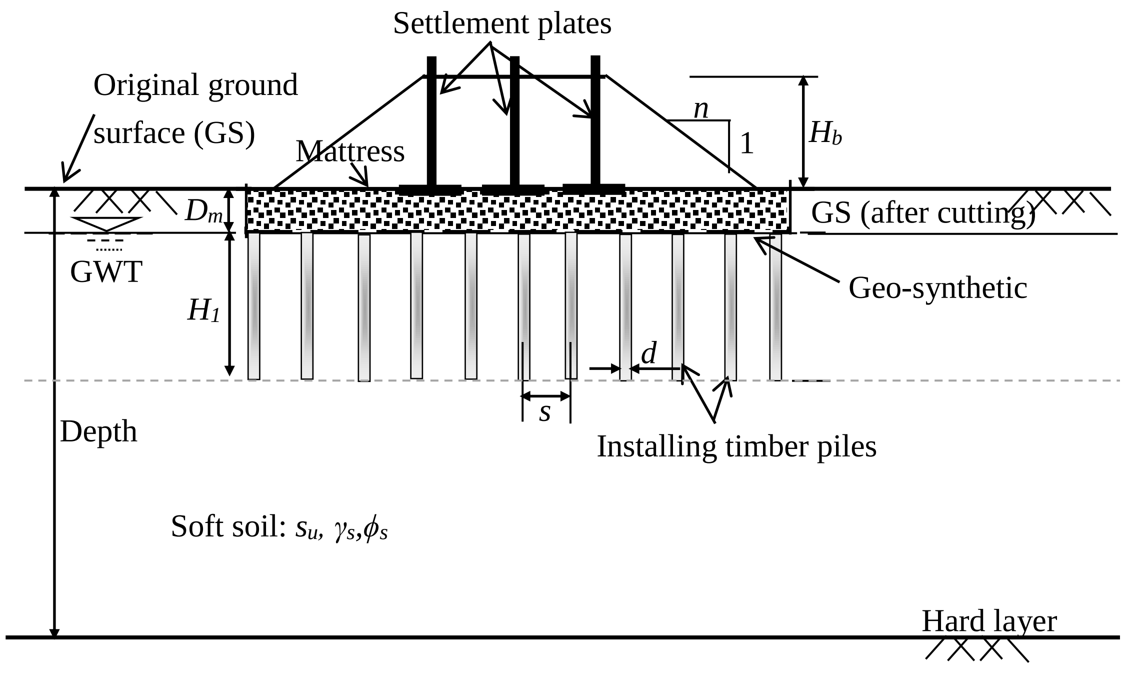


Figure 2.5 Sectional view of trial construction on the mattress with underlying soft soil supported by timber piles [5]

In the application of the traditional reinforcement method for embankments, the local government has to construct an embankment with the required height of more than 3 m. Usually, soil–cement mixing was applied asa mattress construction, with the wet mixed material spread and compacted on top of the installation of timber piles or bamboo piles before filling the embankment. The role of this mattress is to increase the bearing capacity and to reduce settlement in the trial construction [5]. The dataset from a previous trial construction will be used to simulate the criteria for design of the embankment. The outline of the conventional method is shown in Figure 2.6 [3~5].

Start

↓

Monitor settlements of trial embankment

↓

Trial embankment

↓

Simulate the settlement with FEA

↓

Identify design criteria

↓

Construct real embankment

↓

Finish

Figure 2.6 Conventional method of embankment design on soft soil

In Figure 2.6, when the monitored settlement rate is within the range of the guidelines, the engineer can continue the monitoring to check the settlement. The monitored dataset is used to simulate settlement for designing the real construction of the embankment [5] and the local engineer should use this dataset to conduct FEA to identify the design criteria for the embankment. Finally, the design of embankment can be obtained based on the FEA results and the monitored data. But, it is difficult for local engineers and local government in Indonesia to perform FEA, so the recommended process for construction design leads to a loss of time and money.

Therefore, an empirical calculation using the conventional method of traditional reinforcement for embankment design is proposed in this study by using a trial construction dataset in Indonesia.

2.2 Design Considerations for Calculation

2.2.1 Design reinforcement system

In order to determine the performance of the foundation soil with the traditional reinforcement system for an embankment, two model cases of reinforcement systems are prepared, as listed in Table 2.1 [3, 4].

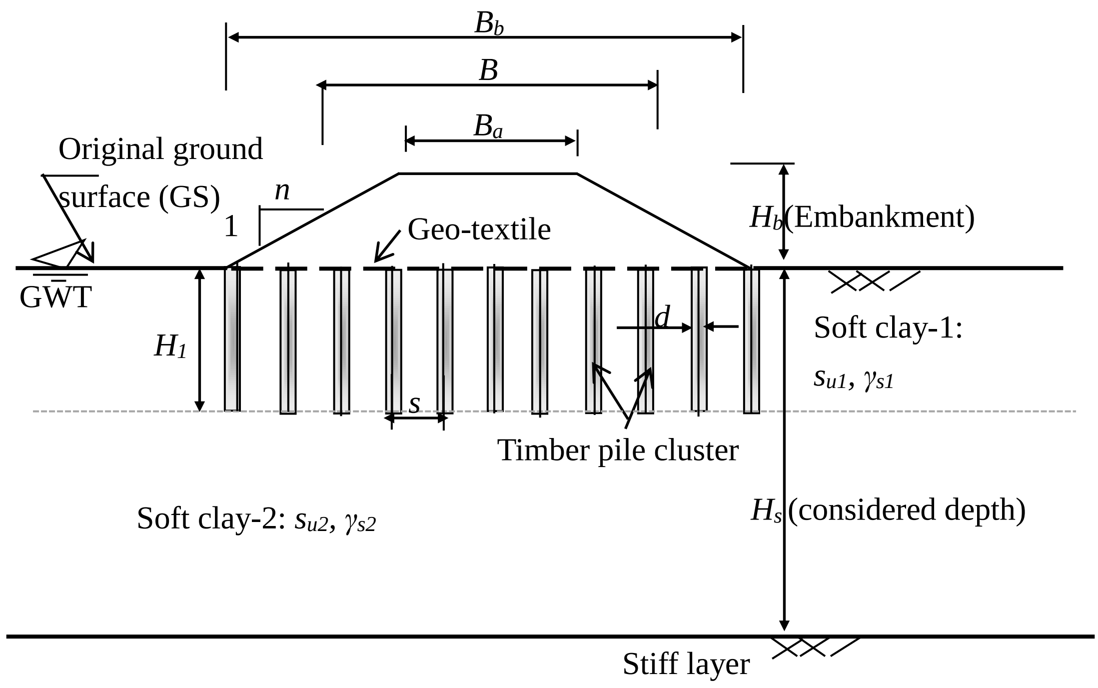
Table 2.1 Reinforcement system for foundation soil [3, 4]

Reinforcement system	Timber pile	Geo-synthetic	Mattress
Case 1	Without	With	Gravel
Case 2	With	With	Gravel

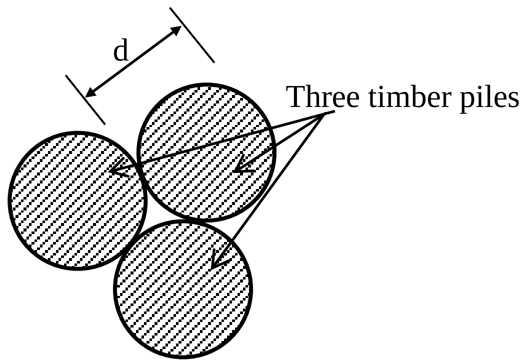
Table 2.1 shows that the mattress used for Case 1 is without timber piles, while Case 2 has foundations where the mattress is supported by timber piles. In Case 2, the position of the pile heads is assumed to be just beneath the mattress. The timber piles are installed without caps to connect between piles.

2.2.2 Trial construction dataset

To simulate the design criteria for the embankment using the empirical method in the calculation scheme, the dataset from a previous trial construction at East Kalimantan in Indonesia is adopted. The trial construction is shown in Figure 2.7 [6, 7].



(a) Cross-section of embankment in the trial construction



(b) Timber pile cluster used in the trial construction

Figure 2.7 Cross-section of a trial embankment construction on soft clay ground supported by timber pile cluster [6, 7]

Figure 2.7 **(**a) shows the trial embankment which was constructed on normally consolidated soft clay. The shear strength of soft clay layer 1 (s_{u1}) and soft clay layer 2 (s_{u2}) are used to define the soil parameters for the design considerations.

Figure 2.7 (b) shows the timber pile clusters used in the trial construction. A model of a two-dimensional timber pile cluster driven in soft clay is used to calculate the friction and end bearing capacity. The equivalent diameter of the timber pile cluster d_e is calculated using two equations.

For friction capacity, the diameter equivalent of pile cluster d_{ef} is defined (Appendix A) as

$$d_{ef} = 2.5d \tag{2.1}$$

For the bearing capacity of the tip, the diameter equivalent of the timber pile cluster d_{eb} is defined as

$$d_{eb} = \sqrt{3} \times d \tag{2.2}$$

where d is the diameter of a single timber pile.

2.3 Loading Pressure on the Mattress

In order to prepare the empirical calculation method for the required stability and settlement criteria, the foundation can be defined intwo strip footing models:

(i) a small strip footing model, which is simplified by using a static load from truck tyres, and

(ii) a large strip footing model, which is modelled from the full embankment.

2.3.1 Load pressure of small footing

In this model, a foundation is constructed on normally consolidated clay. Gravel or sandy gravel material is used for the mattress, and it is reinforced by geo-grid and timber piles [2]. For mattress construction, we need the thickness, angle of friction and bulk density. Therefore, in the real construction a lightweight roller machine is used for compacting the gravel layers. The loading pressure of a small footing on the mattress construction can be idealised by static loading from tyres, which areassumed to be from a truck's rear axle P_{0r}. This is shown in Figure 2.8 [8 ~10].

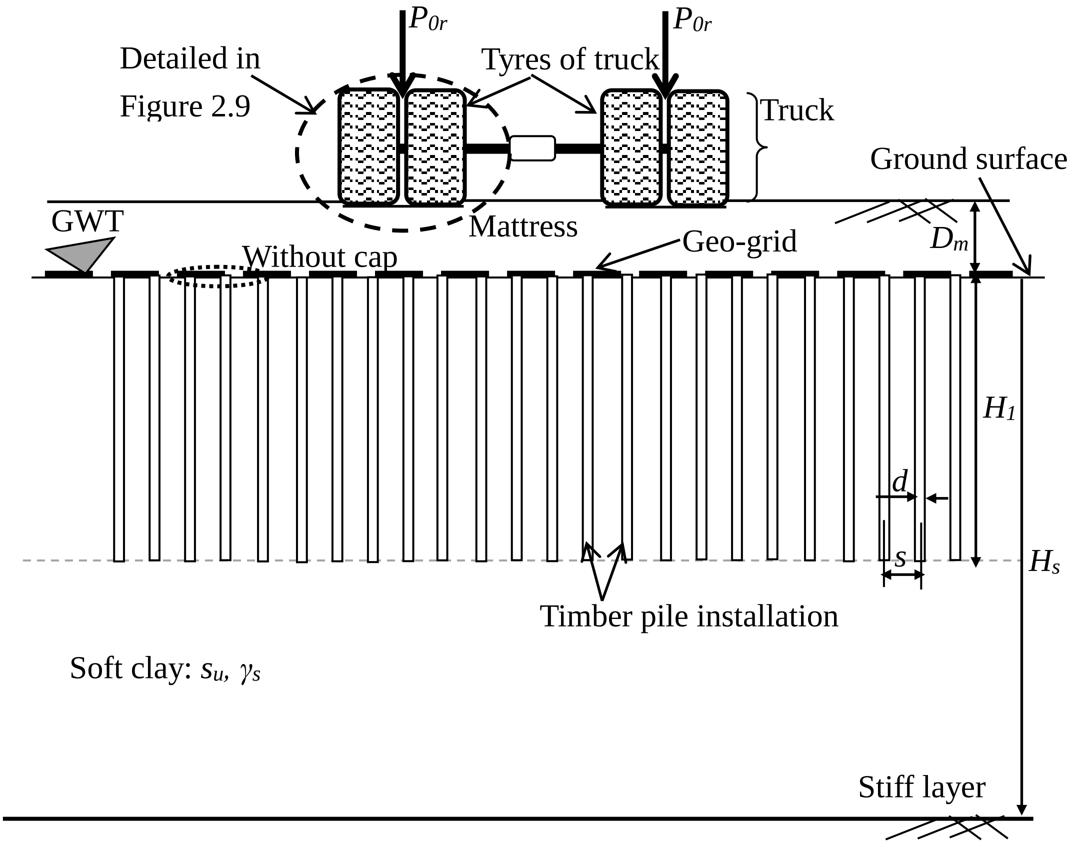


Figure 2.8 Truck tyres loaded on the mattress supported by timber piles

Here, D_m is the thickness of the mattress, H_1 is the length of the piles embedded in soft clay, d is the diameter of the piles, s is the spacing of the piles, H_s is the depth of the soft clays.

The static load pressure p_0 with width footing $2B_0$ from the mattress surface through the mattress to the ground surface is set up by a two-dimensional model in the calculation. This is shown in Figure 2.9 [9, 10].

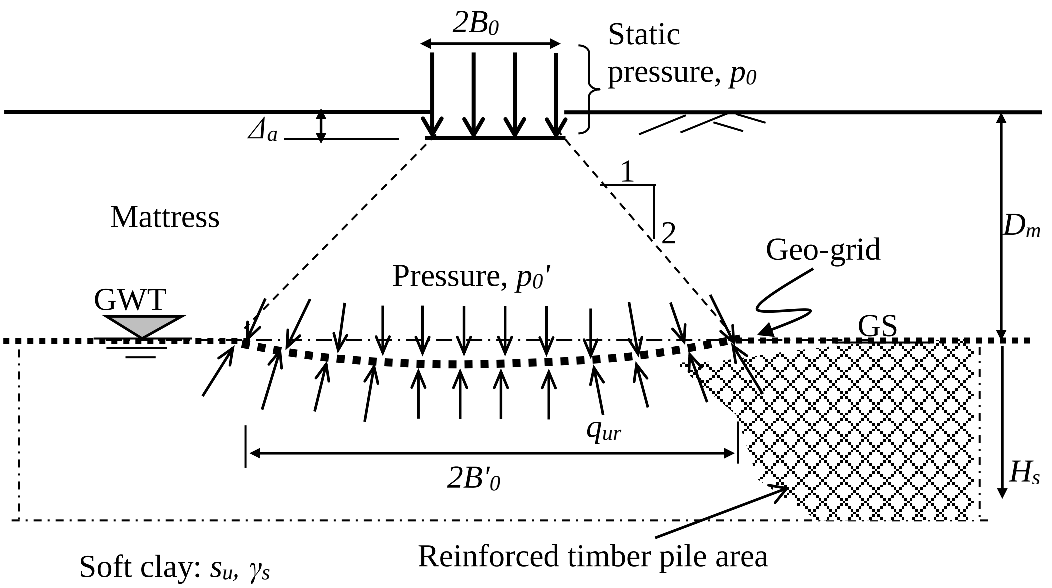


Figure 2.9 Static loading pressure p_0 acting on the mattress supported by timber piles

Here, $2B_0$ is the width of the strip footing for two tyres, $2B'_0$ the width of the strip footing at the ground surface, p_0 is the static loading pressure, p_0' is the load pressure distributed at the ground surface, q_{ur} is the ultimate bearing capacity of the soft clay reinforced with geo-grid and the bearing capacity of the timber piles driven in soft clay.

To determine the tensile strength capacity of the geo-grid on the soft clay, the vertical deformation Δ_a is allowed by load pressure p_0 at the mattress surface, as required in the guidelines [4].

2.3.2 Load pressure of large footing

In order to presentthe proposed empirical calculation method using a two-dimensional model, the load pressure of a large footing of the embankment is applied. The trial construction on the soft clay using the traditional reinforcement system is adopted for the proposed calculation (see Figure 2.7) [6, 7]. Therefore a large footing width B_b of an embankment for the required design criteria is distinguished as two sections for discussion below.

(1) The loading pressure model of the large footing on the gravel layer reinforced with geo-textile (as mattress) is shown in Figure 2.10.

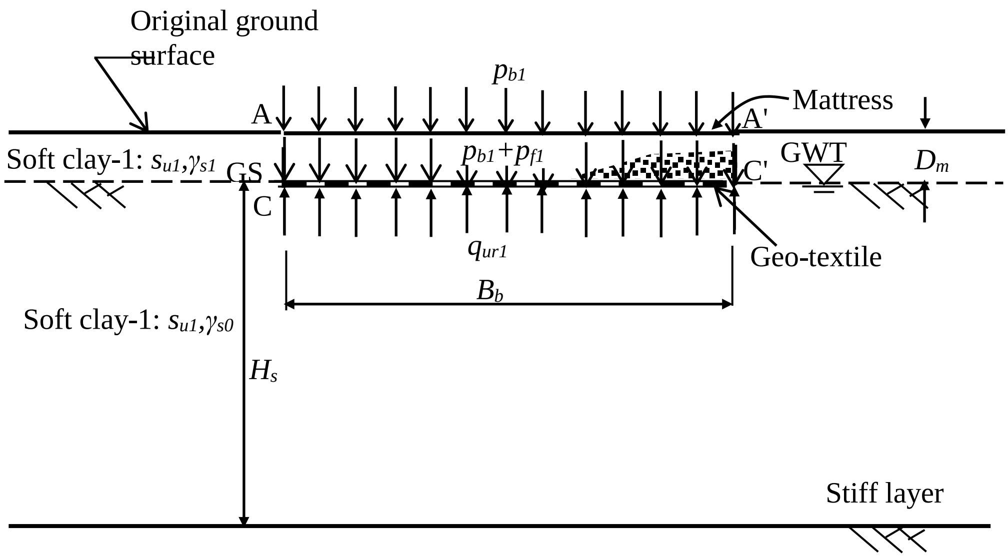


Figure 2.10 Static loading pressure of large footing on the mattress on soft clay

In the figure, the following parameters are defined: the loading pressure on the large footing from embankment p_{b1}, the loading presssure of mattress p_{f1}, the ultimate bearing capacity of original soft clay q_s, the width of the footing at the bottom of the embankment B_b**,** the thickness of the mattress D_m and the depth of the soft clay H_s.

(2) The loading pressure of the large footing on the mattress construction supported by timber piles is shown in Figure 2.11.

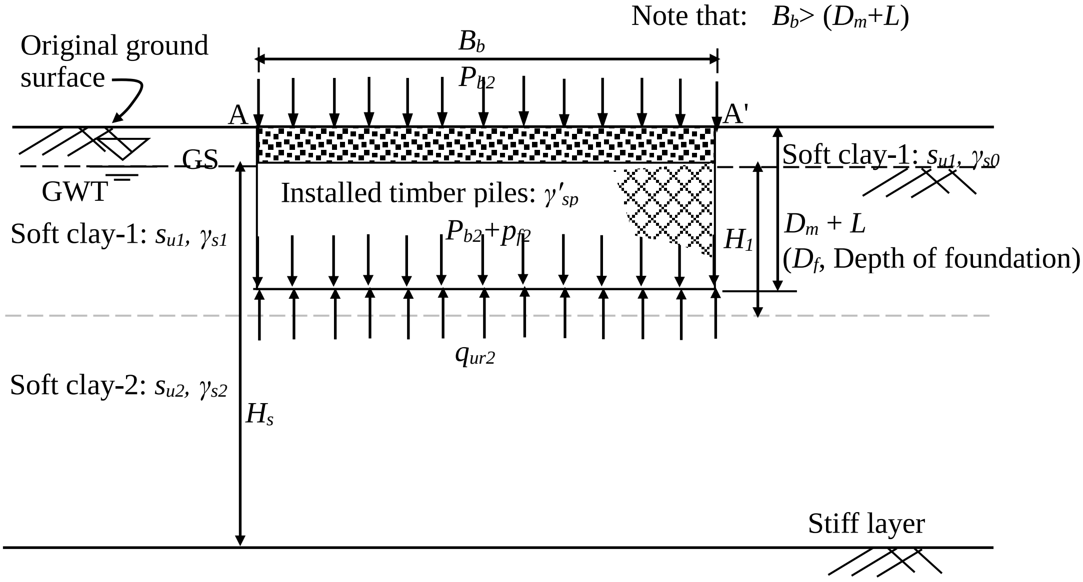


Figure 2.11 Static loading pressure of the large footing on the mattress supported by timber piles

Here, p_{b2} is the loading pressure of the large footing on the embankment ($p_{b1} = p_{b2}$), p_{f2} is the loading presssure of the mattress ($p_{f1} = p_{f2}$), q_{ur2} is the ultimate bearing capacity of the soft clay with geo-textile supported by timber piles, D_m is the thickness of the mattress and γ_m is the unit weight of the mattress.

The load pressure from the embankment p_{b2} is supported by a shallow large footing consisting of timber piles and the mattress. It is assumed that the area reinforced with timber piles and the mattress behaves as a rigid block.

Therefore, the traditional reinforcement system for soft clay can be analysed using the Terzaghi–Peck classical formula, which is prepared with the large footing width B_b. This classical formula refers toa shallow foundation by the ratio of $(D_m+L) / B_b < 1.0$, where (D_m+L) is the depth of the shallow foundation soil [11, 12].

2.4 Proposed Empirical Method

This chapter addresses an empirical calculation method to design the criteria of an embankment on soft clay ground using the traditional reinforcement system (see Table 2.1). However, the empirical calculation method of a small width footing's stability criterion will be presented in Chapter 3. The design of the stability and settlement criteria is presented through two cases respectively.

2.4.1 Stability criterion

2.4.1.1 Stability criterion for Case 1

The model of Case 1 is an embankment on a mattress. The design criterion of the embankment on the mattress is required as [13, 14]

$$p_{b1} + p_{f1} \leq q_{ur1} \tag{2.3}$$

Where q_{ur1} is the ultimate bearing capacity of soft clay, p_{b1} is the load pressure distributed at the ground surface and p_{f1} is the mattress pressure as the foundation.

Hence, the ultimate bearing capacity of soft clay q_{ur1} is provided with the tensile strength of the geo-textile, and can be defined as (see Figure 2.10) [14, 15]

$$q_{ur1} = q_1 + q_2 \tag{2.4}$$

where q_1 is the bearing capacity of the soft clay, q_2 is the capacity of the geo-textile laid at the ground.

The ultimate bearing capacity of the soft soil q_1 for a large strip footing width B_b is usually calculated by [12, 14]

$$q_1 = c_{u1}N_c + \gamma_{s0}D_f N_q + \tfrac{1}{2}B_b\gamma_{s1}N_\gamma \tag{2.5}$$

where N_c,N_q, N_γ are the soil bearing capacity factors, defined as

$$N_q = \exp(\pi \tan\phi_s)\times\tan^2(45^o + \frac{\phi_s}{2}) \tag{2.6a}$$

$$N_c = (N_q - 1)\cot\phi_s \tag{2.6b}$$

$$N_\gamma = 2(N_q + 1)\tan\phi_s \tag{2.6c}$$

where D_f is the depth of the footing, ϕ_s is the the internal friction of the soil, γ_{s0} is the unit weight of soft soil at the top of the ground watertable, andγ_{s1} is the unit weight of saturated soft soil layer 1.

In this case, the angle of internal friction of soft soil ϕ_s is very small, so ϕ_s is assumed to be zero. The factors for the soft soil bearing capacity obtained are N_c of 5.14, N_q of 1.0 and N_γ of zero [14].

The bearing capacity factors in Equation (2.5) for the ultimate bearing capacity of the soil are given by

$$q_1 = s_{u1}N_c + \gamma_{s0}D_f \tag{2.7}$$

In this soft clay, the undrained shear strength of soft clay-1 s_{u1} is used in the calculation (see Figure 2.10), where γ_{s0} is the unit weight of soft clay-1 above the groundwater level, and D_f is the depth of the foundation.

The tensile strength capacity of the geo-textile q_2 per unit meter for width footing B_b is predicted as [15, 17]

$$q_2 = \frac{2T_{gt}\sin\phi_r}{B_b} \tag{2.8}$$

where ϕ_r is the interface friction between the geo-textile and the mattress at both points C-C' in Figure 2.10.

The width at the bottom of the embankment B_b is defined as (see Figure 2.7)

$$B_b = B_a + 2nH_b \tag{2.9}$$

2.4.1.2 Stability criterion for Case 2

The model of Case 2 is an embankment on a mattress supported by timber piles, for which the design criterion is [13, 14]

$$p_{b2} + p_{f2} \leq q_{ur2} \tag{2.10}$$

$$p_{b1} = p_{b2} \tag{2.11}$$

The loading pressures of the embankment p_{b2} and the soil foundation of the mattress p_{f2} at the ground surface can be derived as

$$p_{b2} + p_{f2} = \gamma_b H_b + \left(\gamma_m D_m + \gamma'_{sp} L\right) \tag{2.12}$$

where γ'_{sp} is the (bulk) unit weight of the soft clay with piles, γ_m is the unit weight of the mattress, and D_m is the thickness of the mattress ($D_m \approx D_f$).

The ultimate bearing capacity of the mattress supported by timber piles at the width points A-A' q_{ur2} in Figure 2.11 is

$$q_{ur2} = c_{u2} N_c + \gamma_{s0} D_m + \gamma'_{s1} L \tag{2.13}$$

The effective unit weight of soft clay-1 γ'_{sp} after installing piles is

$$\gamma'_{sp} = \left[1 - n_p \left(\pi d_e^{\,2} / 4s^2\right)\right]\gamma'_{s1} + n_p \left(\pi d_e^{\,2} / 4\right)\gamma_p \tag{2.14}$$

where n_p is the number of timber piles driven per meter square (n_p = 1.0), γ_p is the unit weight of timber piles, and γ'_{s1} is the effective weight of soft clay-1.

2.4. 2 Settlement criterion

2.4.2.1 Settlement criterion for Case 1

The model of Case 1 is an embankment on a mattress construction. The total settlement of the ground surface beneath the embankment on the mattress Δh_{r1} is defined as [16]

$$\Delta h_{r1} = \delta h_{em} + \delta h_0 \tag{2.15}$$

where δh_{em} is the deformation of the embankment on the mattress layer with geo-textile, and δh_0 is the primary settlement by consolidation of the soft clay beneath the mattress.

The calculation of the settlement rate of the soft clay Δh_t by consolidation is considered with the time period t [4, 16]:

$$\Delta h_t = \delta h_m + \delta h_0 \times U_{z1} \tag{2.16}$$

where U_{z1} is the degree of consolidation of soft clay-1.

2.4.2.2 Settlement criterion for Case 2

The model of Case 2 is an embankment on a mattress with geo-textile supported by timber piles. The total settlement of the soft clay surface beneath the embankment on the mattress supported by timber piles Δh_{r2} is defined as

$$\Delta h_{r2} = \delta h_m + \delta h_1 + \delta h_2 \tag{2.17}$$

where δh_m is the deformation of the mattress. By the mattress layer is assumed as a rigid footing, it is not allowed vertical deformation ($\delta h_m \approx 0$) (see Figure 2.11). In which, δh_1 is the elastic deformation of the timber piles and δh_2 is the primary settlement by consolidation of the soft clay.

The definitions of these parameters for the embankment of Case 2 are expressed in Figure 2.12.

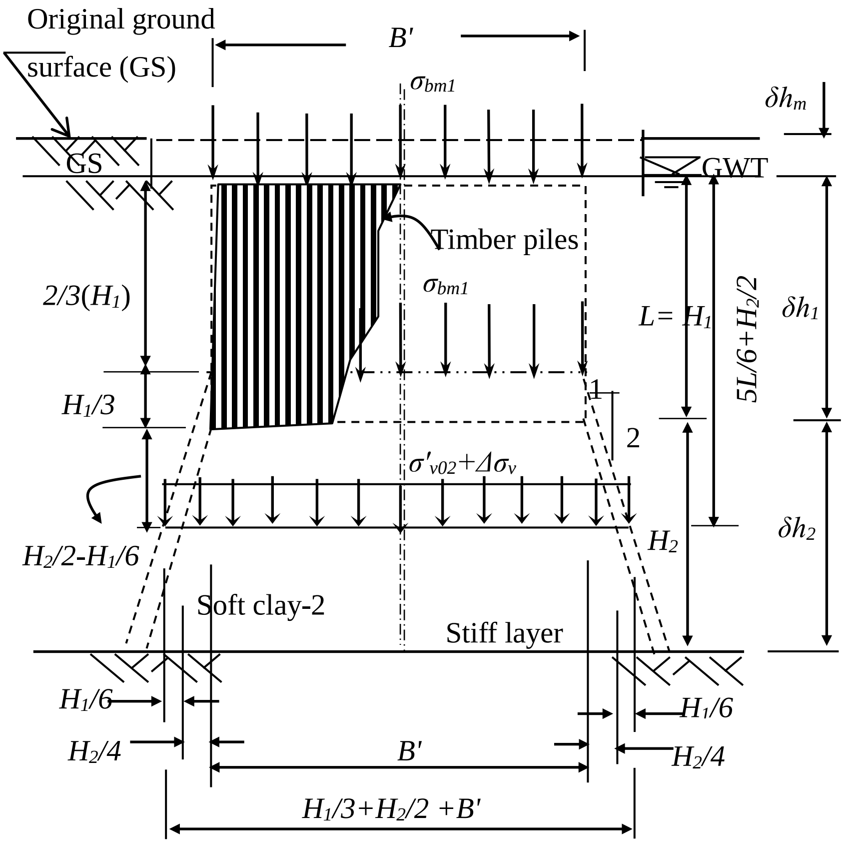


Figure 2.12 Definition of parameters for considered design of settlement [17]

The settlement rate of the soft clay by consolidation (Δh_{t*}) can be given as

$$\Delta h_{t*} = \delta h_1 + \delta h_2 \times U_{z2} \qquad (2.18)$$

where U_{z2} is the degree of consolidation of soft clay-2 in Case 2.

2.5 Comparison with Finite Element Analysis

In order to compare the results from the empirical calculation method for the embankment using the traditional reinforcement method, two-dimensional FEA is used to simulate the design criteria for embankments with the two kinds of reinforcement: (i) a mattress construction as a small footing by tyres loading, and (ii) a mattress construction loaded by a large footing for the full embankment .

2.5.1 Small footing model for mattress

A mattress construction on soft clay ground reinforced with timber piles is modelled as small strip footing width $2B_0$ with pressure p_0 from truck tyres. To determine the soil stress beneath the mattress, FEA is carried out on the loading pressure p_0. The calculation steps for the mattress are shown in Figure 2.13 [18].

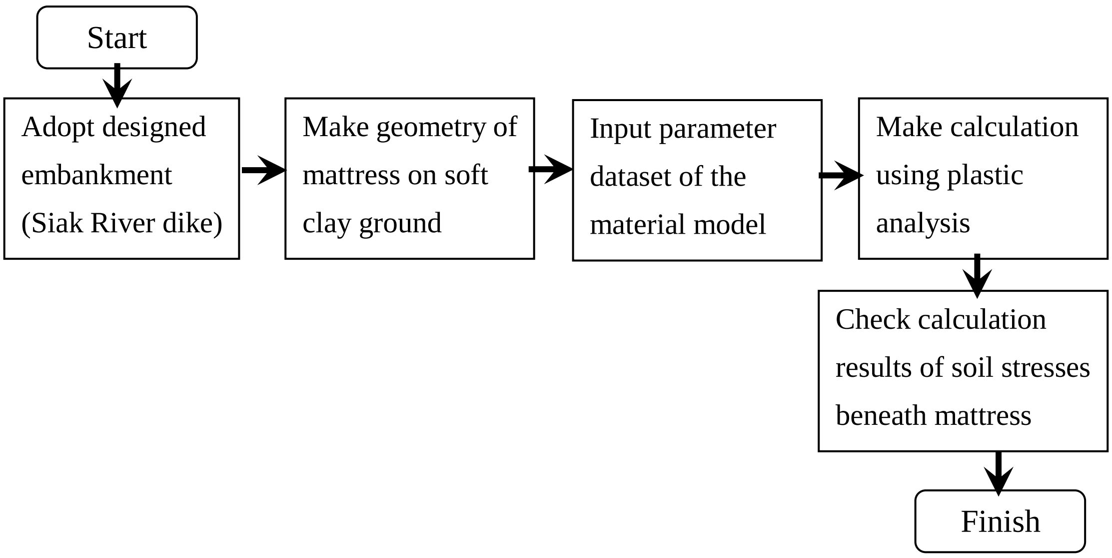


Figure 2.13 Outline of calculation steps for mattress using FEA

Figure 2.13 shows the flow ofcalculations for the embankment at the Siak River dike adopted to apply FEA for the stability criterion (see Figure 1.4). The dimensions and parameters of the geo-grid and timber piles for reinforced soft clay shown in Figure1.4 are used in the calculations.

2.5.1.1 Making the geometry of the mattress

In order to simulate the mattress construction on the soft clay ground with FEA, the geometry of the mattress is represented by a thickness D_m of 0.80 m and a depth of soft clays of about 12 m, as modelled in Figure 2.14 [18].

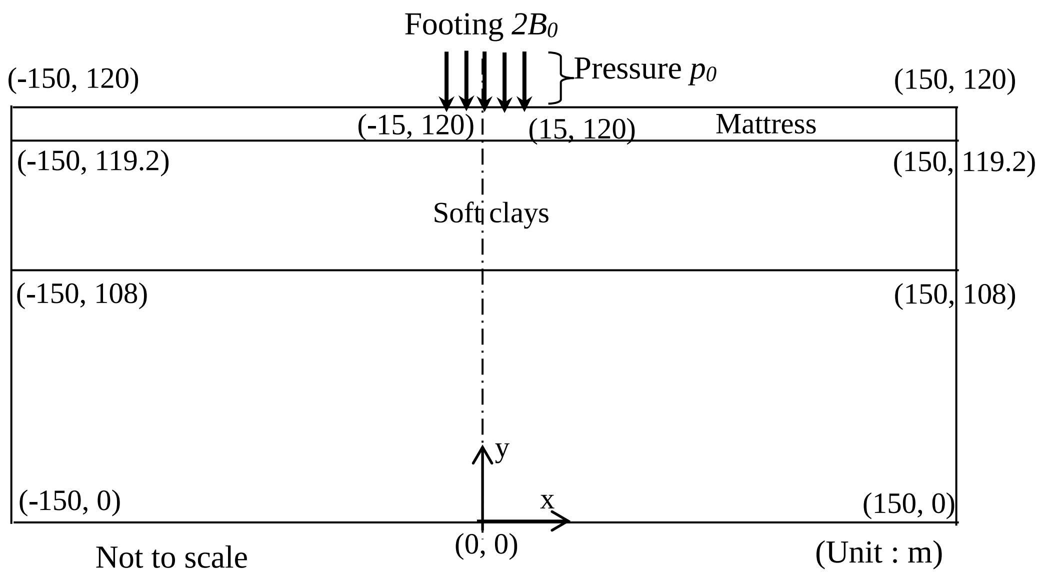


Figure 2.14 Geometry of mattress construction model

Figure 2.14 shows the geometry of the mattress model created by the two-dimensional symmetric condition with an x,y coordinate system, in which x is the horizontal and y is the vertical coordinate. The boundary conditions of the mattress construction are set up by x and y fixed pon the bottom boundary and x fixed on the side boundary.

The geo-grid laid on top of the timber piles at the ground surface beneath the mattress construction is expressed in Figure 2.15.

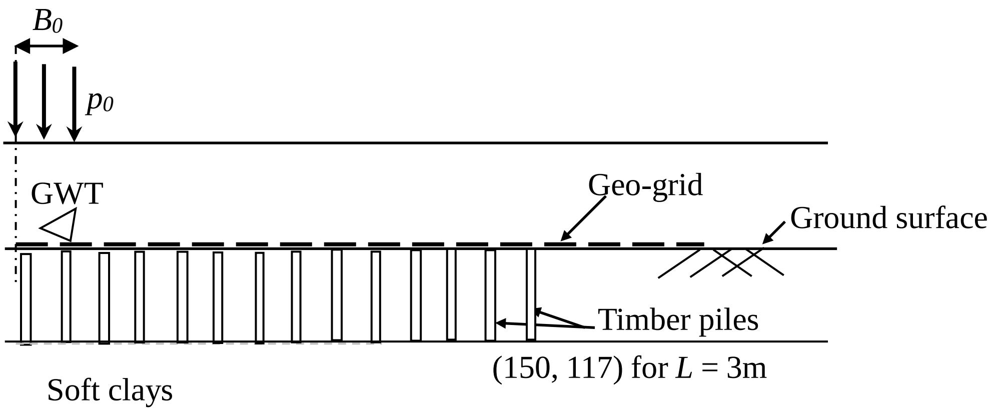


Figure2.15 Modelled half-width strip footing on the mattress

2.5.1.2 Input soil parameters of material model

The soft clay parameters for the finite element simulation referto the final design report of the embankment at Siak River [2]. To calculate the soil stresses at any point beneath the mattress construction, an analysis of undrained soil is modelled in a plastic analysis using the Mohr–Coulomb (MC) model in the FEA. The soft clay properties at the Tembilahan River and Siak River dikes are considered in this section, following Chapter 1 (see Table 1.1 and Table 1.4).

To simulate the soil's elastic and plastic behaviour in the FEA, the Mohr–Coulomb (MC) model is used for all the soil layers of the ground. The MC model involves five parameters: the Young's modulus of soil E_s, Poisson's ratio υ_s for soil elasticity, angle of internal friction ϕ_s, cohesion c for soil plasticity, and angle of dilatancy ψ.

The input parameters of the soils and of the gravel material for the MC model used in FEA are listed in Table 2.2 and Table 2.3.

Table 2.2 The parameters of soft clay ground for the MC model used in the FEA [2, 3]

Soil layers:		Soft clay		Medium clay	Stiff layer
Depth (m)	Unit	0 – 6	6 – 21	21 – 30	> 30
Material type		Undrained	Undrained	Undrained	Undrained
-Unsaturated γ_{unsat}	(kN/m^3)	13.6	14.1	15.2	13
-Saturated γ_{sat}	(kN/m^3)	14.8	16	16.8	14.8
Permeability k_x	(m/day)	9E-05	9E-05	9E-05	9E-05
Permeability k_y	(m/day)	9E-05	9E-05	9E-05	9E-05
Cohesion c	(kN/m^2)	10	18	25	5
Internal friction ϕ	(^{o})	3	3	10	30
Young's modulus E_s	(kN/m^2)	2,500	2,500	3,000	3,500
Poisson's ratio υ_s	-	0.35	0.35	0.35	0.30
Dilatancy ψ	(^{o})	0	0	0	0

Table 2.3 The parameters of gravel for the MC model used in the FEA [19]

Gravel material	Unit	Value
-Unsaturated γ_{unsat}	(kN/m^3)	19
-Saturated γ_{sat}	(kN/m^3)	20.5
Permeability k_x	(m/day)	1
Permeability k_y	(m/day)	1
Cohesion c	(kN/m^2)	1
Internal friction ϕ	(^{o})	46
Young's modulus E	(kN/m$^{2)}$	4,000
Poisson's ratio υ	-	0.35
Dilatancy ψ	(^{o})	16

2.5.1.3 Input parameters of reinforcement models

There are two kinds of reinforcement for the traditional system. The Siak River dike is built on soft clay using geo-grid reinforcement and supported by ordinary timber piles (see Figure 2.15) [2]. These material reinforcements are explained respectively below.

(1) Geo-grid reinforcement

In the model of the geo-grid used to reinforce the gravel layer as amattress (see Figure 2.15), the tensile strength T_{gg} and strain ε_{gg} properties of the geo-grid are associated to determine the axial stiffness parameter $E_{gg}A_{gg}$, which is defined as [20]

$$E_{gg}A_{gg} = \left(100 \times T_{gg}\right) / \varepsilon_{gg} \tag{2.19}$$

(2) Timber pile reinforcement

The timber piles are installed in soft clay to support the gravel layer. In this section, the timber piles are applied as an anchor type to make the model in the software an elasto-plastic material.

There are four parameters for pile modelling: the axial stiffness for timber piles $E_{op}A_{op}$, spacing of piles s, maximum force of pile compression F_{comp}, and maximum force of pile tension F_{tens}. These are explained respectively as follows.

(i) For the parameter of the timber pile material's axial stiffness E_pA_p, it is defined that the constant spring of timber pile k in the soft clay ground can be written as the following equation [21]:

$$k = \frac{E_{op}A_{op}}{H_1} \tag{2.20}$$

Then the axial stiffness E_pA_p is given by

$$E_p A_p = kH_1 \tag{2.21}$$

where H_1 is the length of the timber piles embedded in the soft clay.

The image of the constant spring of timber pile k is shown in Figure 2.16.

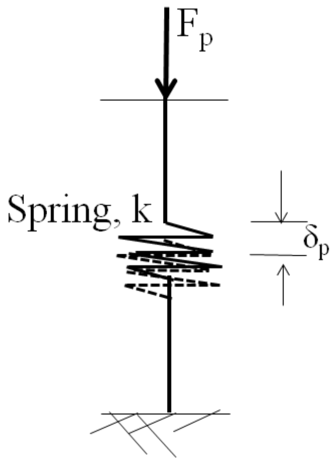


Figure 2.16 Model of constant spring k for timber pile in the soil [21]

The constant spring k can be defined as

$$k = \frac{F_{op}}{\delta_p} \tag{2.22}$$

in which δ_p is the allowed deformation of the timber, and d is the diameter of the timber pile.

By assuming the allowable force F_p loaded on the timber pile, force F_p will be mobilised by allowing a vertical deformation of pile δ_p in the amount [21]

$$\delta_p = \frac{d}{10} \tag{2.23}$$

Then the allowable load capacity of the timber pile driven in soft clay F_{op} can be defined as

$$F_{op} = \frac{P_u}{F_0} \tag{2.24}$$

where F_o is the safety factor for estimating the allowable load capacity of a floating timber pile driven in soft clay.

The ultimate load-bearing capacity of timber pile P_u driven in soft clay is calculated as written in the equations in Appendix B [11].

(ii) The spacing of piles is according to the installation method of the construction procedure published in the guidelines (see Figure 1.7).

(iii) The maximum force of pile compression F_{comp} is assumed to be equal to the ultimate load-bearing capacity of timber pile P_u. The detailed equations of the calculation can be found in Appendix B.

(iv) For the maximum force of pile tension F_{tens} [11], the ultimate load-bearing capacity of timber pile P_u driven in soft clay is calculated. The detailed equations for this calculation are given in Appendix C.

2.5.1.4 Setting of calculation scheme for mattress

In the calculation scheme of the FEA, the initial conditions are calculated with the initial pore water pressure, which comes from the water level at the ground surface. For drainage conditions, the ground surface is drained and the other boundaries of the two vertical sides are undrained.

The calculation steps start by creating a model of the ground, laying the model of the geo-textile on the ground, installing piles into the soft clay, and so making the model of the mattress and calculating the soils tresses (see Figures 2.14 and 2.15).

To calculate the soil stresses beneath the mattress, a plastic analysis is provided in FEA and is set up as below.

(1) making the ground model, placing the geo-grid, constructing a mattress, selecting the plastic analysis and mattress pressure p_0 of a small strip footing width $2B_0$ (see Figure 2.14).

(2) making the ground model, creating the timber pile installation, placing the geo-grid, constructing a mattress, selecting the plastic analysis and load from mattress pressure p_0 with the strip footing (see Figure 2.15).

2.5.2 Large footing model for embankment

A mattress construction on soft clay ground reinforced by timber piles is modelled with a large strip footing as the full embankment. The calculation steps using FEA for the embankment are shown in Figure 2.17.

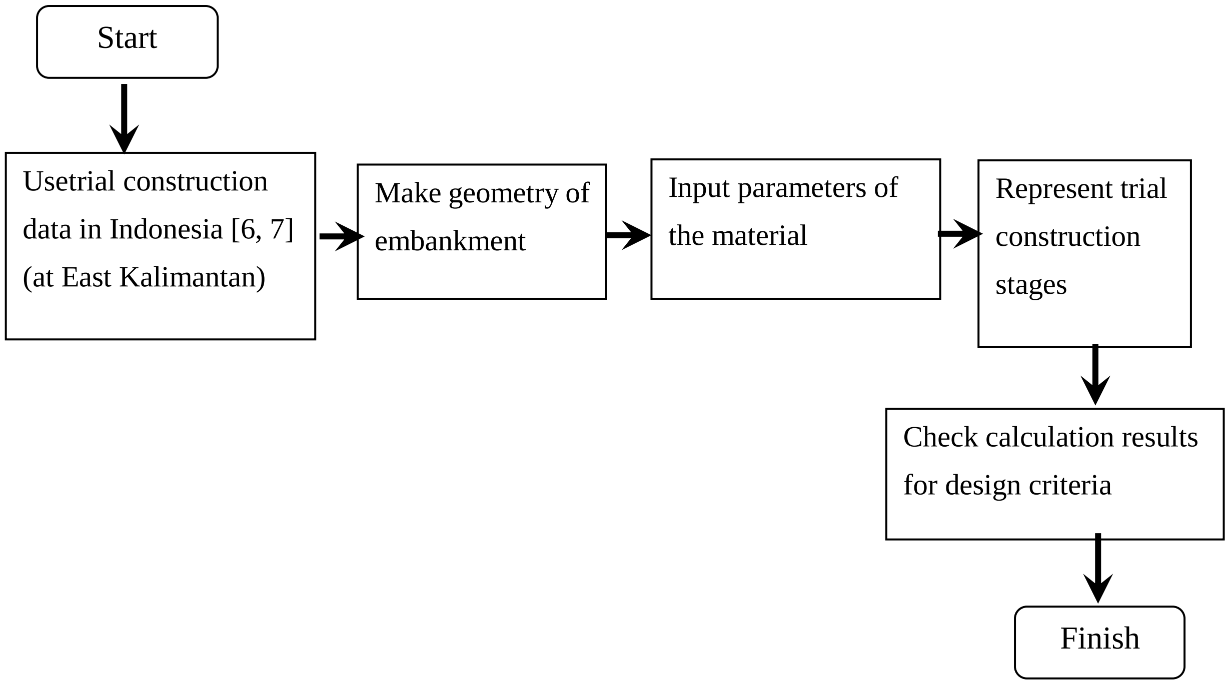


Figure 2.17 Outline of calculation steps using FEA for embankment

2.5.2.1 Trial construction dataset

To prepare the parameters of the soil models for simulation in the FEA, the dataset of the trial construction of the embankment at East Kalimantan (see Figure 2.7) is adopted. The obtained properties of the soft clay and embankment are listed in Tables 2.4 and 2.5 [4, 6, 7].

Table 2.4 The parameters of soft clay ground [6, 7]

Depth (m)	Unit weight (kN/m^3)		Cohesion c_u (kN/m^2)	Angle of friction ϕ_s (°)	Permeability (m/day)		Remark
	Unsaturated γ_{unsat}	Saturated γ_{sat}			k_x	k_y	
0 – 4	12	14.5	10	5	1.38E-03	6.89E-04	Soft clay-1
4 – 6	12	14.5	12	8	1.38E-03	6.89E-04	Soft clay-2
6 – 12	13	15	20	12	1.38E-03	6.89E-04	Soft clay-3
12 – 18	15	16	25	14	1.38E-03	6.89E-04	Soft clay-4
18 – 25	16	18	30	16.5	1.38E-03	6.89E-04	Silty sand
> 25	16	18	30	16.5	1.38E-03	6.89E-04	Hard layer

Table 2.5 The soil parameters of the embankment [4, 7]

Unit weight (kN/m^3)		Cohesion c_u (kN/m^2)	Friction ϕ_b(°)	Permeability	
Unsaturated, γ_{unsat}	Saturated, γ_{sat}			k_x (m/day)	k_y (m/day)
19	20	1	33	2	1

For this trial embankment construction, it was reported that timber pile clusters were installed in the soft clay before laying the geo-textile (see Figure 2.7). The axial stiffness of the timber pile clusters $E_{pc}A_{pc}$ will be explained below.

The local government conducted a trial construction at East Kalimantan by working with embankments 20 m long and 30 m wide for Case 1 and Case 2. The trial embankments of the two cases were monitored for 98 days and vertical deformation was reported at the centreline of the ground beneath the embankment [6,7]. The details of the two cases are shown in Figures 2.18 and 2.19.

(1) Case 1 is an embankment on soft clay reinforced by geo-textile.

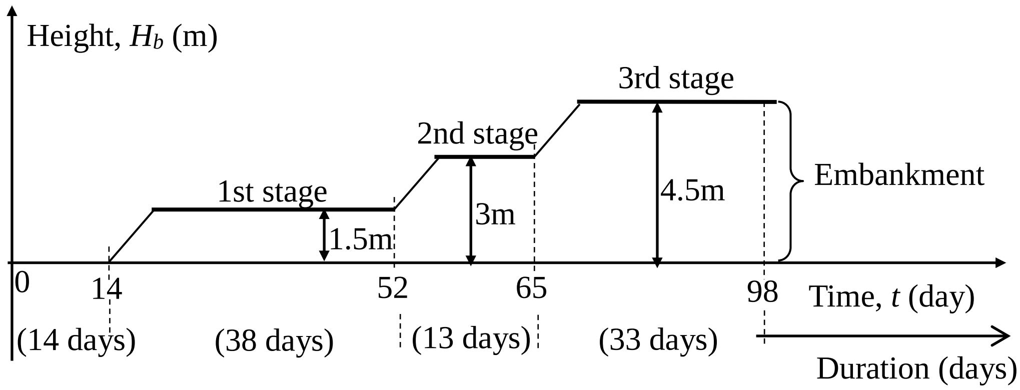


Figure 2.18 Construction stages of trial embankment for Case 1 [6, 7]

(2) For Case 2, an embankment on soft clay reinforced with geo-textile and supported by timber pile clusters is shown in Figure 2.19.

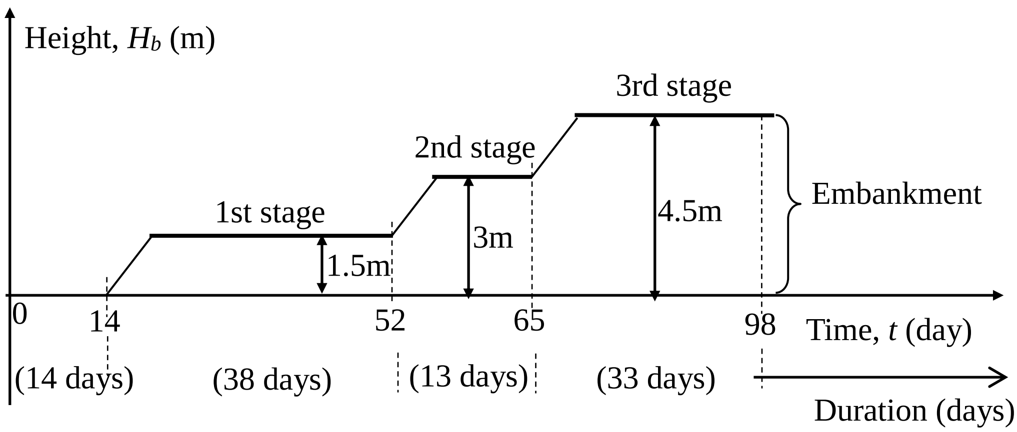


Figure 2.19 Construction stages of trial embankment for Case 2 [6, 7]

The reinforcement parameters of the traditional system in the finite element simulation are listed in Table 2.6 [7].

Table 2.6 The parameters of traditional reinforcement in the trial construction [6, 7]

Trial construction	Geo-textile T_{gt} (kN/m)	Timber pile cluster		Remark
		Diameter d_e (cm)	Length H_1 (m)	
Case 1	55	-	-	Without piles
Case 2	55	25 (d =10cm)	6	With timber piles

Table 2.6 shows the obtained diameter equivalent of timber pile clusters using Eq.2.2 (see Figure 2.7(b).

2.5.2.2 Making the geometry of the embankment

In order to make the FEA simulation of the design embankment on the soft clay ground, the geometry of the embankment is modelled as shown in Figure 2.20.

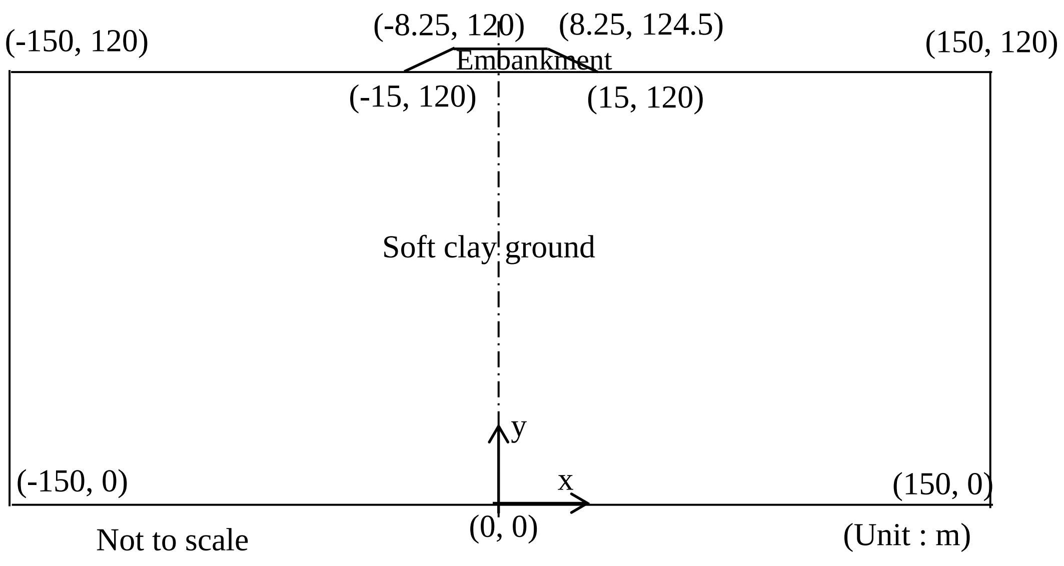


Figure 2.20 Geometry of embankment model on soft clay ground

The geometry of the embankment models on soft clay ground created by the two-dimensional symmetric construction with coordinate system (x,y) for the two cases is shown respectively in Figure 2.21 and Figure 2.22.

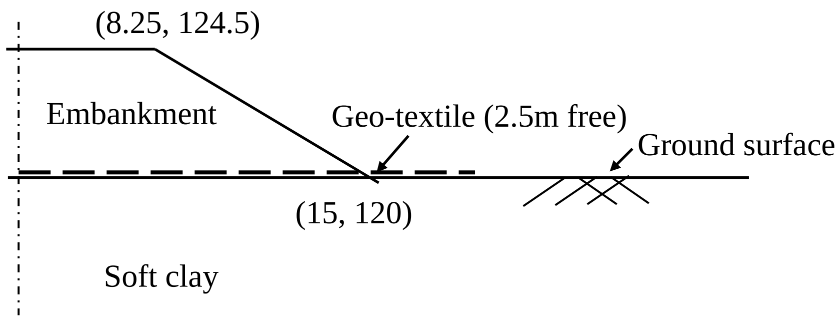


Figure 2.21 Modelled embankment construction for Case 1

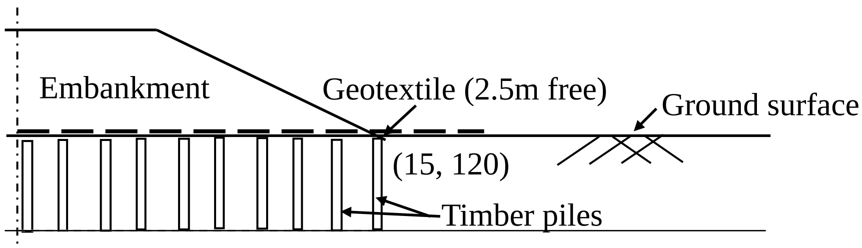


Figure 2.22 Modelled embankment construction with timber piles for Case 2

Figure 2.21 shows the boundary conditions of the embankment, where x and y are fixed on the bottom boundary and x is fixed on the side boundary for the geo-textile laid on the ground surface, and both of the toes are opened with length 2.5 m for the two cases.

2.5.2.3 Input soil parameters of material model

In order to make FEA simulations of the design criteria for the embankments, the soil parameters obtained from the trial construction dataare adopted (see Table 2.3 and Table 2.4). The parameters of the soft clay ground and embankment were reported for the Mohr–Coulomb failure criterion (MC) [6, 7]. Thus, this simulation is represented by two soil models as explained respectively.

(1) Modified Cam-Clay model for soft-to-medium clay layers

The Modified Cam-Clay (MCC) model is selected for normally consolidated soft clays. The material for the Modified Cam-Clay model consists of five parameters: the Cam-Clay compression index λ, Cam-Clay swelling index κ, critical state line M, Poisson's ratio υ_{ur} and initial void ratio e_0 [22].

The compression index λ is defined as [22]

$$\lambda = 0.0058PI \tag{2.25}$$

The plasticity index PI (in percent) for all of the soil is predicted by [23]

$$PI = 102.3 \times \frac{C_c}{(1+e_0)} + 13.34 \tag{2.26}$$

where C_c is the consolidation compression index of the soil.

The swelling index κ can be calculated by the next equation [22]:

$$3.0 \leq \left(\frac{\lambda}{\kappa}\right) \leq 8.0 \tag{2.27}$$

For the initial compression stress states, the slope of the critical state line M is defined as [22, 23]

$$M = \frac{6\sin\phi_s}{(3-\sin\phi_s)} \tag{2.28}$$

where ϕ_s is the angle of the internal friction of the soil from the triaxial compression test.

In soft clays, the angle of dilatancy parameter ψ for the MCC model is almost zero [18].

The results obtained from the soil sample for the input parameters of soft clay ground are listed in Table 2.7.

Table 2.7. Input soft clay ground parameters for MCC model used in FEA

Soil layers		Soft clay				Medium clay
Depth (m)	Unit	0 – 4	4 – 6	6 – 12	12 – 18	18 – 25
Material type		Drained	Drained	Drained	Drained	Drained
Soil unit weight: [6, 7]						
Unsaturated γ_{unsat}	(kN/m^3)	12	12	13	15	16
Saturated γ_{sat}	(kN/m^3)	14.5	14.5	15	16	18
Permeability:						
Permeability k_x	(m/day)	1.38E-03	1.38E-03	1.38E-03	1.38E-03	1.38E-03
Permeability k_y	(m/day)	6.89E-04	6.89E-04	6.89E-04	6.89E-04	6.89E-04
Initial void ratio e_{init}	-	2.2	2.2	2	1.8	1.5
Coef. consol. C_c	-	0.9	0.9	0.85	0.6	0.4
Plasticity *PI*	(%)	42.11	42.11	42.33	35.26	29.7
Compression λ [22]	-	0.525	0.525	0.496	0.384	0.290
Swelling κ	-	0.088	0.088	0.082	0.069	0.057
Poisson's ratio of unloading-reloading υ_{ur} [18]	-	0.15	0.15	0.15	0.15	0.15
Slope of critical state line M [22]	-	0.179	0.292	0.447	0.772	1.199
Cohesion c [6, 7]	(kN/m^2)	10	12	20	25	30
Internal friction ϕ	(^{o})	5	8	12	14	16.5
Dilatancy ψ [18]	(^{o})	0	0	0	0	0

(2) Mohr–Coulomb model for hard layer and embankment

For elastic perfectly-plastic soil behaviour, the Mohr–Coulomb (MC) model is used for the hard layer and embankment. The MC model involves five parameters: Young's modulus E, Poisson's ratio υ, angle of friction ϕ, cohesion c and angle of dilatancy ψ.

The results obtained for the hard layer and embankment for the input soil parameters are listed in Table 2.8.

Table 2.8 Input parameters of hard soil and embankment for MC model used in FEA

Soil description		Hard layer	Embankment
Dimension	Unit	Depth of 25 m to 120 m	Height of 4.5 m
Material type		Drained	Drained
Soil unit weight [6, 7]:			
Saturated γ_{unsat}	(kN/m^3)	16	19
Unsaturated γ_{sat}	(kN/m^3)	18	20
Permeability k_x	(m/day)	2	2
Permeability k_y	(m/day)	1	1
Soil Young's modulus E	(kN/m^2)	8.000	10.000
Soil Poisson's ratio υ	-	0.35	0.35
Soil cohesion c	(kN/m^2)	1	1
Soil angle of friction ϕ	(^{o})	30	33
Soil dilatancy ψ [18]	(^{o})	0	3

2.5.2.4 Input parameters of material models for reinforcement

There are two kinds of reinforcement for the traditional reinforcement method in the trial embankment: geo-textile reinforcement for Case 1 and geo-textile combined with timber pile clusters for Case 2 (see Figure 2.7) [6, 7]. These materials are explained below.

(1) Geo-textile reinforcement

In the model with geo-textile (see Figure 2.10), the tensile strength T_{gt} and strain ε_{gt} of the geo-textile are associated to determine the parameter of the material's axial stiffness $E_{gt}A_{gt}$ [20]:

$$E_{gt}A_{gt} = \left(100 \times T_{gt}\right) / \, \varepsilon_{gt} \qquad (2.29)$$

where A_{gt} is the area of the geo-textile.

(2) Timber pile cluster

The model of timber piles is used to reinforce the soft ground for the embankment (see Figure 2.11). In this section, the timber pile cluster is modelled as an elasto-plastic material in the FEA.

There are four parameters:axial stiffness $E_{pc}A_{pc}$, spacing of piles s, maximum force of pile compression F_{comp}, and maximum force of pile tension F_{tens}. These parameters are explained respectively below.

(i) The parameter of the timber pile cluster with axial stiffness $E_{pc}A_{pc}$ is defined with the spring contant of timber pile k [21]:

$$k = \frac{E_{pc}A_{pc}}{H_1} \tag{2.30}$$

Then the axial stiffness of the timber pile cluster $E_{pc}A_{pc}$ is predicted as [21]

$$E_{pc}A_{pc} = kH_1 \tag{2.31}$$

where H_1 is the length of the timber pile cluster.

The constant spring of the timber pile cluster k driven soil layer is defined in Figure 2.16. The constant spring k can be predicted as in Equation (2.22), in which δ_p is the allowed deformation for piling.

The allowable force F_p which is loaded on the timber pile cluster will be mobilised by allowing a vertical deformation of pile δ_p to the amount of [21]

$$\delta_p = \frac{d_e}{10} \tag{2.32}$$

where d_e is the calculated diameter of the timber pile cluster driven in soft clay.

Then the allowable load capacity of the timber pile cluster driven in soft clayF_{pc} can be defined as

$$F_{pc} = \frac{P_u}{F_0} \tag{2.33}$$

where F_o is the safety factor for estimating the allowable load capacity of the floating timber pile cluster driven in soft clay ground.

The ultimate load-bearing capacity of timber pile cluster P_u driven in soft clay is calculated in Appendix B [11].

(ii) The spacing of the piles installed for the road embankment follows the published guidelines (see Figure 1.7).

(iii) The maximum force of pile compression F_{comp} is assumed to be equal to the ultimate bearing capacity of timber pile cluster P_u (see Equation B.1 in Appendix B) [11].

(iv) The maximum force of pile tension F_{tens} is used,

The ultimate load-bearing capacity of timber pile cluster T_{ug} driven in soft clay is calculated in Appendix C [11].

2.5.2.5 Setting the calculation scheme for the embankment

In the calculation scheme of the FEA, the initial conditions are calculated with the initial pore water pressure, which comes from the water level at the ground surface. For drainage conditions, the ground surface is drained and other boundaries of the two vertical sides are closed.

The calculation steps start by creating a model of the ground, laying geo-textile on the ground surface, installing piles into the ground, filling the embankment and calculating the stress and settlement by consolidation (see Figures 2.16 and 2.17).

The calculation for determining the settlement by consolidation for Case 1 and Case 2 in the FEA is set up as follows.

(1) For Case 1, making the ground, laying the geo-textile, selecting the consolidation analysis and constructing the embankment as stage loading (see Figure 2.18).

(2) For Case 2, making the ground, installing the timber pile clusters, laying the geo-textile, selecting the consolidation analysis and constructing the embankment as stage loading (see Figure 2.19).

The bearing capacity of the soil in undrained conditions for Case 1 and Case 2 is predicted by the soil stresses beneath the embankment, using a plastic analysis in the FEA.

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Chapter 3. Design Criteria of Reinforcement of Thick Soft Clay Foundation using Traditional Construction Method in Indonesia

3.1 Introduction

Flooding and sea waves are two frequently occurring natural hazards in many Indonesian islands. More often than not, these hazards cause severe damage to the residential areas located along the coasts and river dikes. It is impossible to curb the occurrence of these natural hazards, but it is possible to implement appropriate technologies to mitigate their severe impacts.

The problems with river dikes in Indonesia commonly arise from the two factors of the natural and technical situations. A notable example of a natural situation is seen in Sumatra Island,which has almost all the typical problems in Indonesia, including thick soft soil, floods, earthquakes and wave attacks from the sea or rivers.

This chapter will address two cases of river dikes in Indonesia, at Tembilahan River and Siak River. Before reconstructing a soil dike 300m long and 53m wide using a concrete plate bridge on concrete piles (see Figure 1.2), the Tembilahan River dike had previously collapsed due to weak foundation soil and triggered by heavy rainfall. The configuration of its concrete pile construction wasset with a pile diameter d of 60cm and pile spacing s of 2 m and 5 m perpendicular and parallel to the river dike direction, respectively (see Figure 1.3) [1].

The Siak River dike was also reconstructed by filling the 600 m long embankment to overcome flood hazards and wave attacks from the surrounding river (see Figure 1.4) [2]. The local government conducted CPT and SPT tests to investigate the thickness of the soft soilH_s, and found H_s values of 21m and 12m at the Tembilahan and Siak River dikes respectively. The results of these investigations are summarised in Table 3.1.

Table 3.1 The results of CPT and SPT tests [1, 2]

River case	CPT, q_{nc} (kg/cm^2)	N-SPT	Depth H_s (m)	Soil consistency
Tembilahan	1 ~ 3	1 ~ 3	21	Soft soil
Siak	1 ~ 5	1 ~ 4	12	Soft soil

where q_{nc} is the uncorrected cone resistance, and N-SPT is the number of blows.

Table 3.1 shows the level of soft soils for the *N-SPT* at Tembilahan River and Siak River. The local engineer investigated soil samples for both sites and reported the soft soil properties: the undrained cohesion of soil c_u showed as less than 25kPa, the plasticity index *PI* was more than 52% for Tembilahan River (see Table 1.1) [1] and more than 29.5% for the Siak River dikes (see Table 1.4) [2]. Therefore those sites are found to be soft clays down to depths of 21 m for Tembilahan River and 12 m for Siak River [1~3].

In order to increase the bearing capacity of the soft soil, a traditional method of reinforcement called *Cerucuk* made from timber or bamboo piles was used by Indonesian local people based on the guidelines [4, 5]. The timber piles were installed into the soft soil for reinforcement prior to construction of the dikeat Siak River, as explained in Chapter 1 (see Figure 1.4).

However, as concerns the technical situation, the design of the Siak River dike with the implementation of a timber pile installation has never previously existed in the Indonesian design code.

The construction process of the traditional reinforcement system for the dikes involves (a) cutting the ground surface (GS) for site preparation, (b) installing timber piles, (c) laying geo-grid on top of the timber piles, (d) spreading gravel material and compacting it as a mattress for distributing the load pressure, and (e) filling the embankment (dike) [5].

For the typically designed dike construction of Siak River on soft clay (see Figure

1.4), finite element analysis (FEA) and the calculation results of the safety factors *Fs* for the slope of the dike were reported [2].

3.2 Research Objectives

This research aims to identify the geotechnical solutions for stable performance of the river dike on soft soils using traditional reinforcement.This involves the following: (1) installing timber piles in soft clay, (2) laying geo-grid beneath a gravel layer, and (3) spreading gravel over the geo-grid as a mattress beneath the embankment.

The objective of the research is to present an empirical calculation method based on geotechnics for the mattress construction performance using the traditional reinforcement system. It is shown in Figure 3.1.

Figure 3.1 shows the flow chart of the considered design based on the geotechnical rules for the required design criteria, with the ultimate bearing capacity for the mattress overlying soft clay and supported by timber piles. The empirical calculation method will be presented.

To evaluate the ultimate bearing capacity performance of the mattress, the load spreading within the mattress from the surface of theground was calculated approximately by using the value of the slope of load spreading. This ultimate bearing capacity of the mattress overlying soft clay, both unreinforced and reinforced, was studied for a small width footing [6, 7].

The Ministry of Public Works has published guidelines on how to construct an embankment on a mattress overlying soft soil supported by timber piles [4, 5]. However, the guidelines are too difficult for Indonesian local engineers to applly, because the design scheme is not detailed enough to calculate the required performance.

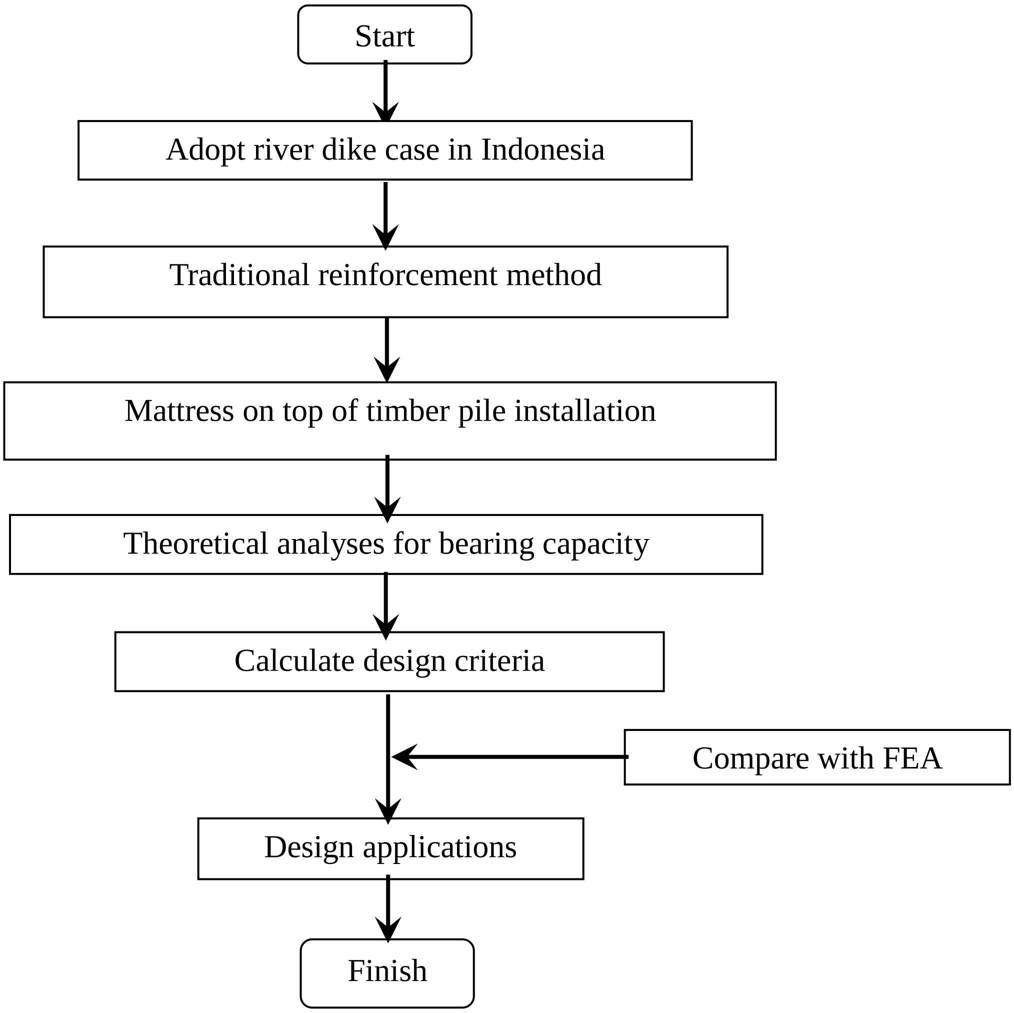


Figure 3.1 Flow chart for research objective

Therefore, several researcher shave studied empirical analysis methods for determining the ultimate bearing capacity of sand overlying soft clay reinforced by geo-textile, which was conducted for a small footing width of two truck tyres [8 ~ 11].

In the real construction for the mattress, a static load pressure is loaded on the mattress supported by timber piles. To express the two point loads of P_{0r} for both sides, the load pressure is considered from a truck with four tyres on the rear axle [12, 13].

The point load of P_{0r} can be defined through two tyres P_{0r} of $1/3W_T$, in which W_T is assumed to be the fully weighted truck [14]. The detailed expression of point load P_{0r} was shown in Chapter 2 (see Figure 2.8).

The design criterion of the traditional construction method will be provided by using several empirical formulae, as explained below.

3.3 Theoretical Analyses

3.3.1 Distribution of load pressure

This empirical calculation method is used to provide the stability criterion for a mattress overlying soft clay supported by timber piles. The load pressure from the truck is distributed through the mattress to the ground surface. The expression of this term is shown in Figure 3.2 [15].

Figure 3.2 shows that the total pressure p_0' distributed at the ground surface is required to be more than or equal to the allowable bearing capacity of soft clay with reinforcement q_{ar}, which is defined as [13, 15, 16]

$$p_0' = q_{ar} \tag{3.1}$$

$$p_0' = p_0 \frac{(2B_0 \times L_0)}{(2B'_0 \times L'_0)} + \left(D_m - \Delta_a\right)\gamma_m \tag{3.2}$$

$$q_{ar} = \frac{q_{ur}}{Fs} \tag{3.3}$$

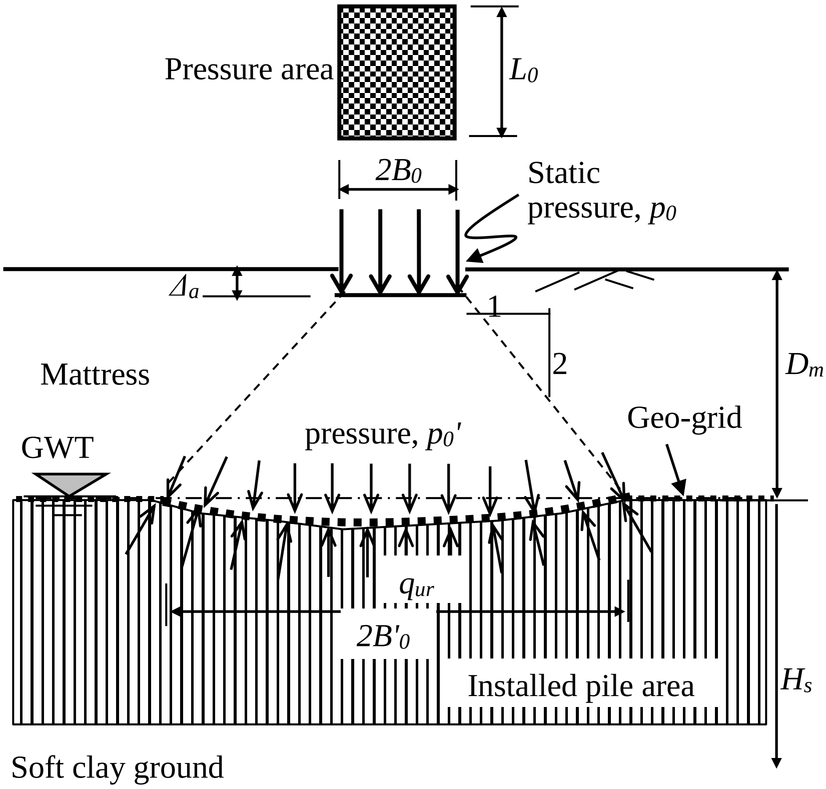


Figure 3.2 Load pressure from truck applied on the mattress overlying soft clay

where p_0' is the load pressure distributed to the ground surface and Δ_a is the vertical deformation allowed under loading pressure [5].

Here, the load pressure distributed by the angle 1:2 to the ground surface (See Fig.3.2). Which can be refered from the National Standardisation Agency of Indonesia. The width of load spreading at the ground surface $2B'_0$ is calculated by [12, 13]

$$2B'_0 = 2B_0 + (D_m - \Delta_a) \tag{3.4}$$

The length of load spreading at ground surface L'_0 is given by

$$L'_0 = L_0 + (D_m - \Delta_a) \tag{3.5}$$

where γ_m is the unit weight of the mattress, $2B_0$ is the width of the load pressure at the mattress surface, L_0 is the length of the load pressure at the mattress surface (see Figure 3.2) and Fs is the factor of safety [15, 16].

To calculate the ultimate bearing capacity of the mattress construction overlying soft clay supported by timber piles, the mechanism of the mattress construction supported by timber piles is shown in detail in Figure 3.3.

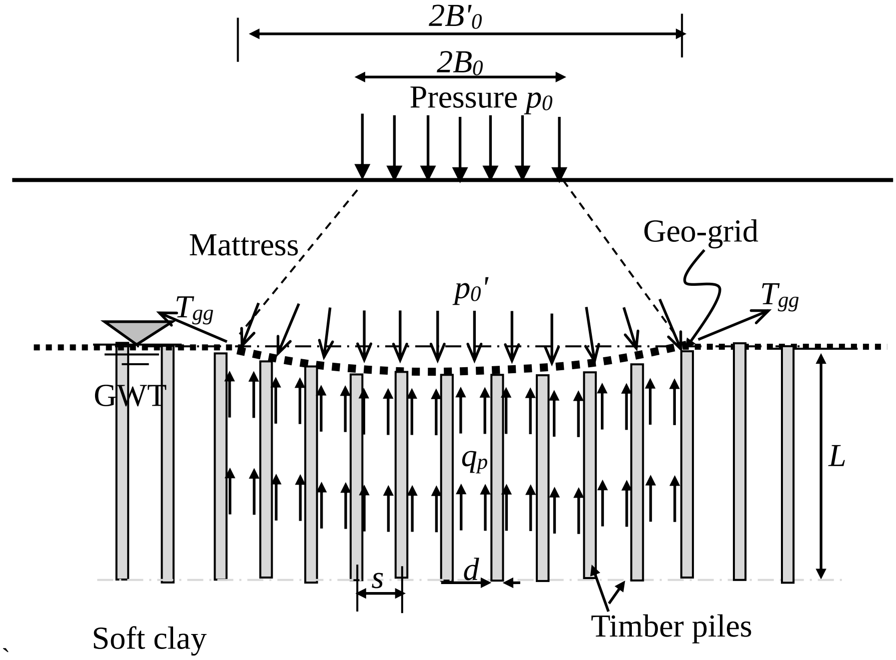


Figure 3.3 Load pressure distribution p_0' on geo-grid and timber piles

The ultimate bearing capacity of the mattress overlying soft clay supported by timber piles q_{ur} leads toa simple formula to calculate the stability criterion:

$$q_{ur} = q_{gg} + q_p \tag{3.6}$$

where q_{gg} is the tensile bearing capacity of the geo-grid, q_p is the friction of timber piles driven in soft clays, which is considered the soil's effective vertical stress [17, 18], L is the length of the piles embedded in the soil, d is the diameter of the piles, s is the spacing between piles.

The ultimate bearing capacity of the soft clay with reinforcement can be predicted by the relationship between the bearing capacity of the mattress q_{ur} and its vertical deformation due to loading pressure p_0 as in Figure 3.4.

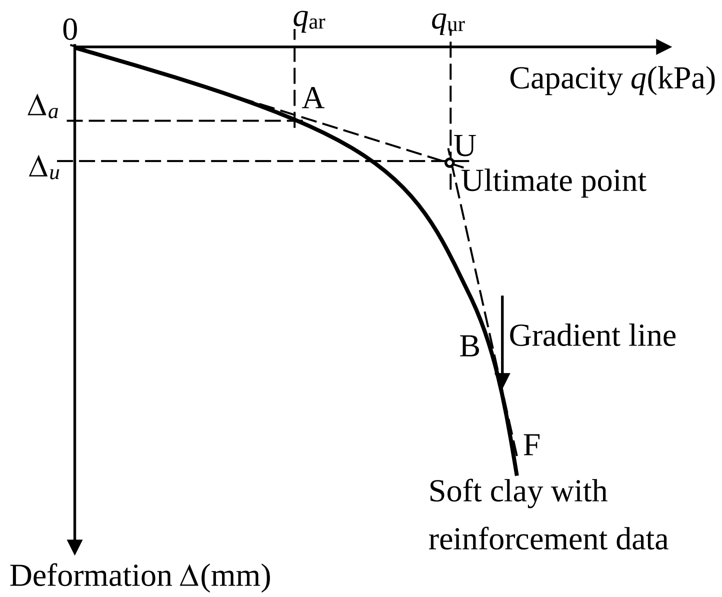


Figure 3.4 Correlation between bearing capacity of mattress and vertical deformation

Figure 3.4 shows the ultimate bearing capacity of the mattress overlying soft clay supported by timber piles q_{ur}. It is obtained by the intersection of two lines at point U: the gradient line of the elastic line of point 0 – point A and the gradient line of the failure line of point B – point F.

Therefore, the allowable bearing capacity of mattress q_{ur} and the allowable vertical deformation Δ_a are found in Equation (3.3).

3.3.2 The ultimate bearing capacity of reinforced ground

A calculation of the tensile capacity of the geo-grid beneath the mattress is derived from the assumption about the allowed deformation at the top of mattress Δ_a and its equal deformation beneath the mattress. The expression of this term is shown in Figure 3.5.

The approximate calculation of the tensile capacity of the geo-grid and the frictional capacity of the piles driven in soft clay are explained as follows.

3.3.2.1 Tensile capacity of geo-grid

There are three considered categories for calculating the criterion of geo-grid laid beneath the mattress: (a) normal stress reinforced by the geo-grid, (b) interfacial shear stress along the geo-grid, and (c) shear stress of soft clays affected by the geo-grid.

Figure 3.5 shows the deformation for a sand layer with geo-textile, which is applied for geo-grid at the ground surface for the width footing $2B'_0$ [10, 11]. In part of the geo-grid reinforcement, the tensile bearing capacity of the geo-grid q_{gg} for width spreading footing $2B'_0$ can be obtained by

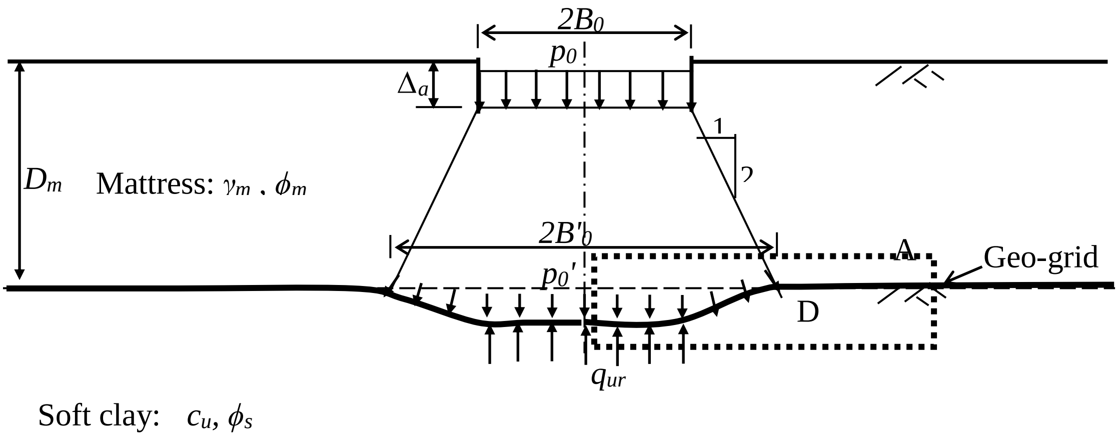
$$q_{gg} = \frac{1}{2B'_0}\left(T_{gg} \sin\theta_0 + \Delta_a \left[p_m + \frac{2B_0}{2B'_0} p_0 \right] \tan\psi \right) \tag{3.7}$$

in which the angle of inclination θ_0 at point D is defined as[6, 7]

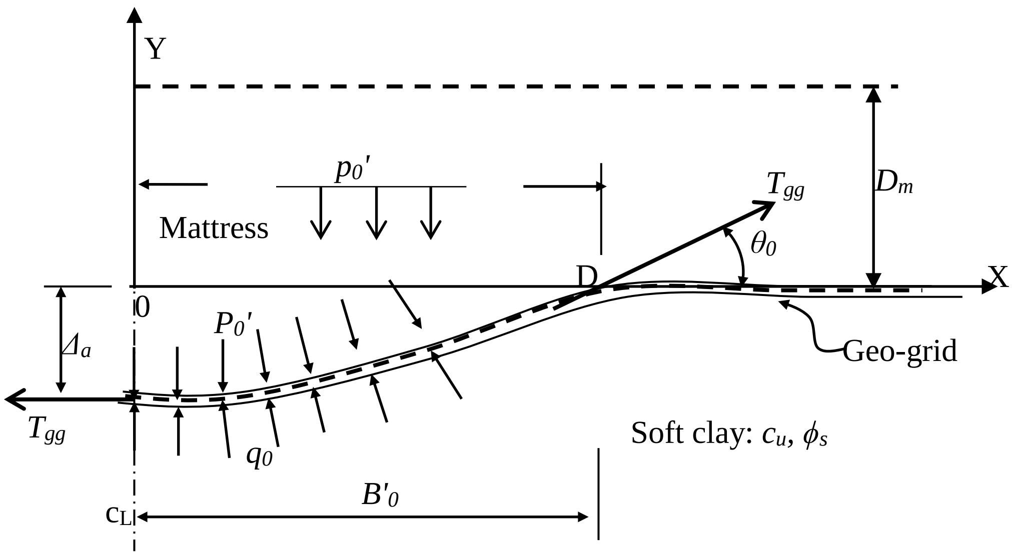
$$\theta_0 = \tan^{-1}\left(\frac{4\alpha_\Delta}{1-(2\alpha_\Delta)^2} \right) \tag{3.8}$$

The interface friction ofthe mattress with the geo-grid ψ is calculated by

$$\psi = \tan^{-1}\left(\frac{D_m\left[K_{am} - K_{pm}\right] + \dfrac{p_0}{p_m}\left[\eta K_{am} - \tan\delta\right]}{\left[1 + \dfrac{p_0}{p_m}(B_0 / B'_0)\right] 2B'_0} \right) \tag{3.9}$$



a. Mechanism of ultimate bearing capacity of mattress overlying soft clay



b. Detail-deformed shape and stress acting on the geo-grid beneath the mattress

Figure 3.5 Expressions of mattress overlying soft clay

The dimensionless factor η of half-width strip footing B_0 is considered an active pressure of the mattress and is calculated by

$$\eta = 2\ln\left(\frac{B'_0}{B_0}\right) \tag{3.10}$$

The vertical deformation factor α_Δ is defined as

$$\alpha_{\Delta} = \frac{\Delta_a}{2B'_0} \tag{3.11}$$

where θ_0 is the angle of inclination at point D (^{o}), δ is the angle roughness of the mattress to the tyre.

3.3.2.2 Bearing capacity of pile driven in soft clay

To predict the bearing capacity of the timber piles driven in soft clay, the research is focused on normally consolidated clay.

By considering the effective vertical stress of the soil, the friction of a timber pile in soft clay q_p for the width of footing $2B'_0$ can be defined as [18, 19]

$$q_p = \alpha_p \times \left(\frac{2B'_0}{s}\right) c_u \tag{3.12}$$

The adhesion coefficient of a timber pile driven in soft clay α_p is predicted by

$$\alpha_p = 0.55\left(\frac{L}{40d}\right)^{-0.2} \times \left(\frac{\sigma_{v0}'}{c_u}\right)^{0,3} \tag{3.13}$$

where d is the diameter of the timber pile.

The effective vertical stress σ_{v0}' along the pile of soft clay is predicted by

$$\sigma_{v0}' = L \times (\gamma_s - \gamma_w) \tag{3.14}$$

where γ_s is the unit weight of saturated soil, γ_w is the unit weight of water and L is the length of the pile embedded in the soil.

3.3.3 Loading pressure on the mattress

In static loading, the dimensionof load pressure on the mattress assumes that the contact area of the tyre is a rectangular footing. The intensity of load pressure p_0 at the mattress surface is predicted by [8, 14]

$$p_0 = \frac{P_{0r}}{2B_0 \times L_0} \tag{3.15}$$

where P_{0r} is the point load of two tyres of the truck's rear axle, $2B_0$ is the width of the contact area, and L_0 is the length of the contact area at the mattress surface.

3.4 Comparison with Finite Element Analysis

In order to compare the results of the proposed empirical calculation for mattress construction, the FEA is carried out by using the Mohr–Coulomb model to determine the soil stresses beneath the mattress.

The geometry of mattress construction on the soft clay is modelled for the FEA simulation (see Figure 2.14). To validate the ultimate bearing capacity of the mattress on the soft clays from the empirical calculation, the detailed expression of the half-width B_0 of the small strip footing on the soft clay surface is shown in Figure 3.6 [20].

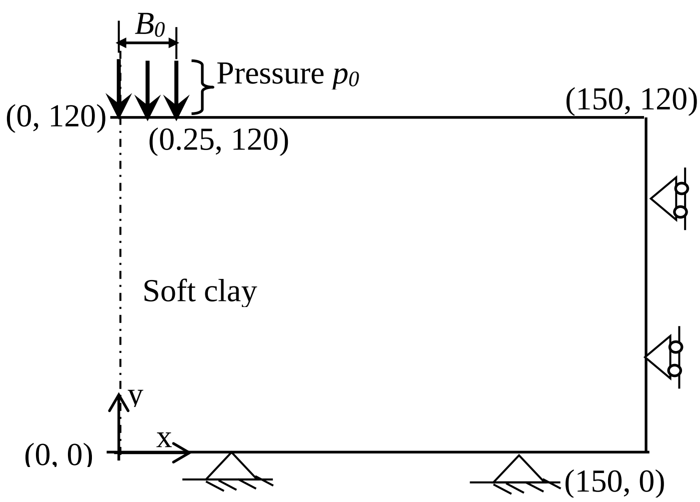


Figure 3.6 Geometry of half-width strip footing on the soft clay surface

Figure 3.6 shows the geometry of the mattress model created by the two-dimensional symmetric coordinate system (x,y). The boundary conditions of the embankment are defined by x and y fixed on the bottom boundary and x fixed on the side boundary.

The geo-grid laid at the ground surface beneath the mattress was explained in Chapter 2 (see Figure 2.15) .

For determining the soil plastic behaviour in the FEA, the Mohr–Coulomb (MC) model is prepared for all of the soft clay layers to calculate the plastic analysis. The MC model involves five parameters: the Young's modulus of soil E_s, Poisson's ratio υ_s, angle of friction ϕ_s, cohesion c and angle of dilatancy ψ [21]. The parameters of the soft clays are listed in Table 3.2.

Table 3.2 The parameters of soft clays for MC model used in the FEA [1, 2]

Soil layers		Soft clay		Medium clay	Stiff layer
Depth (m)	Unit	0 – 6	6 – 21	22 – 30	> 30
Material type		Undrained	Undrained	Undrained	Undrained
-Unsaturated γ_{unsat}	(kN/m^3)	13.6	14.1	15.2	13
-Saturated γ_{sat}	(kN/m^3)	14.8	16	16.8	14.8
Permeability k_x	(m/day)	9E-05	9E-05	9E-05	9E-05
Permeability k_y	(m/day)	9E-05	9E-05	9E-05	9E-05
Cohesion c	(kN/m^2)	10	18	25	5
Internal friction ϕ	(°)	3	3	10	30
Young's modulus E_s	(kN/m^2)	2,500	2,500	3,000	3,500
Poisson's ratio υ_s	-	0.35	0.35	0.35	0.30
Dilatancy ψ	(°)	0	0	0	0

The axial stiffness parameter of the geo-grid $E_{gg}A_{gg}$ is defined in Equation (2.17). In application, the spring constant of the timber pile k is calculated by using Equations (2.18)–(2.21) in Chapter 2.

In setting the scheme of the FEA, initial conditions are calculated with the initial pore water pressure, which comes from the water level at the ground surface. For drainage conditions, the ground surface is drained and the other boundaries of the two vertical sides are undrained.

The steps in the calculation scheme follow the construction stages shown in Figures 2.14 and 2.15. A plastic analysis is carried out in the FEA calculation scheme by making the ground model, placing the geo-grid, constructing the mattress, selecting plastic analysis and the mattress loaded by pressure p_0 of small strip footing width $2B_0$ (see Figure 2.14). The next step involves making the ground model with installing timber piles, placing the geo-grid, constructing a mattress and conducting a plastic analysis of mattress by pressure p_0 as the strip footing (see Figure 2.15). The soil parameters of gravel are listed in Table 3.3.

Table 3.3 The parameters of gravel for MC model used in the FEA [6, 7]

Gravel	Unit	Value
-Unsaturated γ_{unsat}	(kN/m^3)	19
-Saturated γ_{sat}	(kN/m^3)	20.5
Permeability k_x	(m/day)	1
Permeability k_y	(m/day)	1
Cohesion c	(kN/m^2)	1
Internal friction ϕ	(^{o})	46
Young's modulus E	(kN/m^2)	4,000
Poisson's ratio υ	-	0.35
Dilatancy ψ	(^{o})	16

3.5 Design Applications

In this section, two design application methods for reinforced soft clay using the traditional method, namely the empirical method and FEA are introduced.

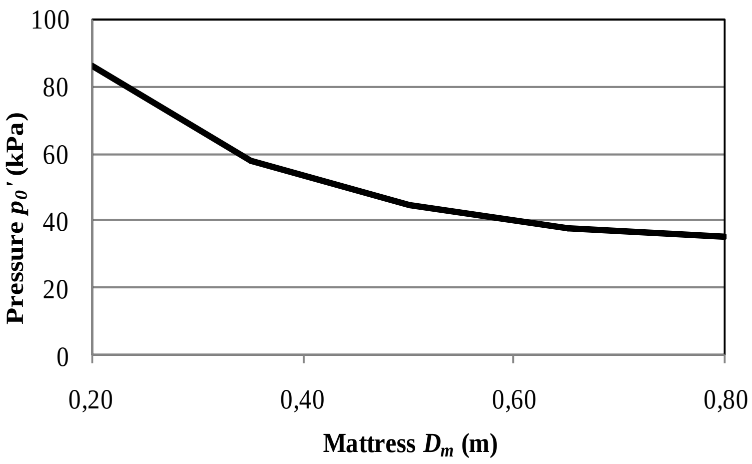
In the empirical method, the proposed calculation criteria for reinforced soft clay were examined down to the depth of 21 m at the Tembilahan River dike. The parameters of the soft clay lead to the cohesion c_u of 18 kPa and the submerged unit weight γ_s of 14.8 kN/m^3 [1].

The dimensions and parameters of the mattress are obtained as γ_m = 20.5kN/m^3, D_m = 0.2~0.8 m, N_c = 5.14. These results were accounted into the calculation of the factor of safety *Fs* = 2.0 [7, 15]. The timber pilesare installed in a square pattern with variable parameters, including spacing *s* (*s* = 3*d*, 5*d*, 7*d*), diameter of pile *d* (8 cm and 10 cm), length of pile *L* (3 m and 4.5 m) and tensile strength of geo-grid T_{gg} of 24 kN/m [2, 4].

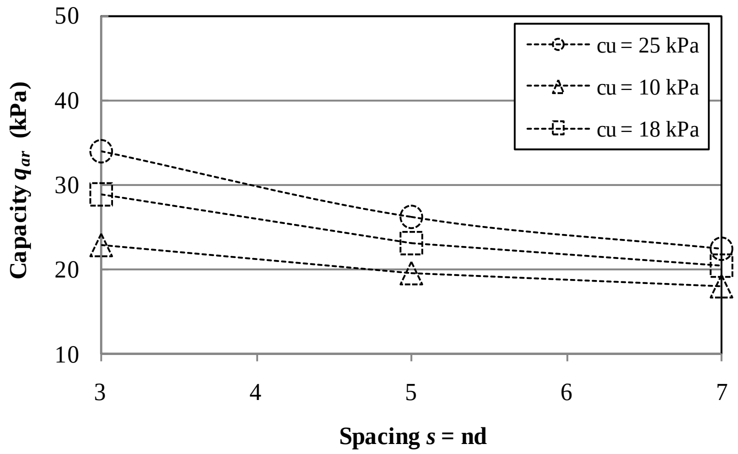
The load pressure of the small footing p_0 was calculated as 205.05 kPa, which applied a point load P_{0r} of 26.7 kN with an area $2B_0$ 0.51 m wide and L_0 0.255 m long [2, 14].

The load pressure distribution p_0' is obtained by calculation with a pile diameter *d* of 8 cm and length (*L* = 3 m and 4.5 m), with varying spacing *s* as shown in Figure 3.6 and Figure 3.7 respectively. Then, timber piles with diameter *d* of 10 cm and varied spacing *s* and length of piles (*L* = 3 m and 4.5 m) are shown in Figures 3.8 and 3.9 respectively.

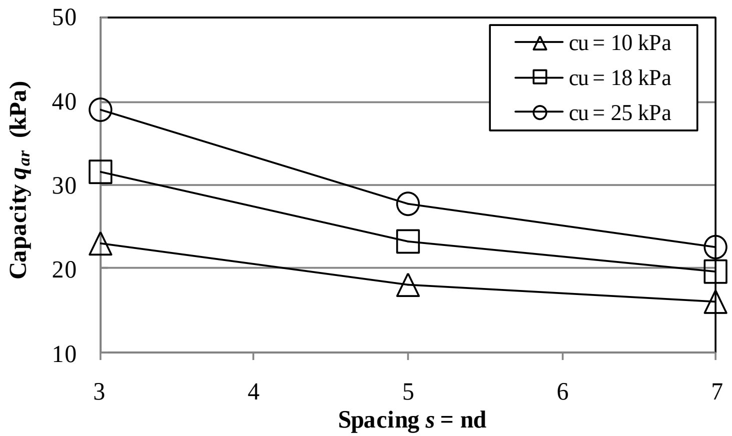
Figure 3.6 shows the criterion of the allowable bearing capacities q_{ar}. These were available when the calculated pressure is less than the distributed loading pressure p_0' with cohesion c_u of 18 kPa and 25 kPa in all the thickness of the mattress D_m. However, there are exception with the cohesion c_u of 25 kPa and mattress D_m of 0.50~0.80 m on variation spacings *s* of 3*d*~7*d* and length *L* of 3.0m.



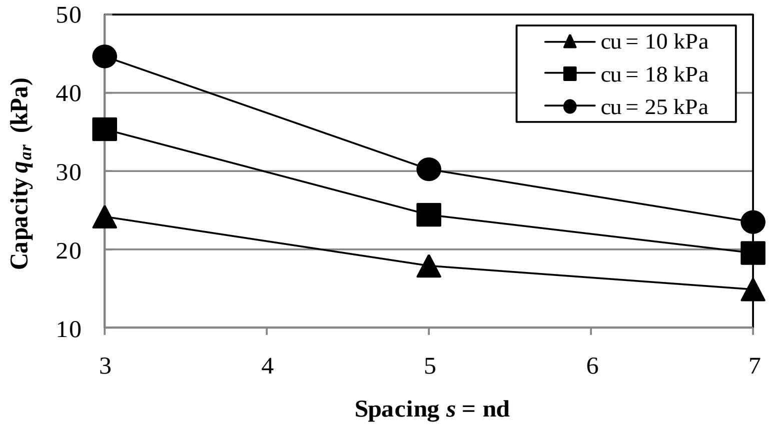
a. Pressure *p0'* vs mattress *Dm*



. b. Bearing capacity q_{ar} vs spacing s = nd (D_m= 0.20m)

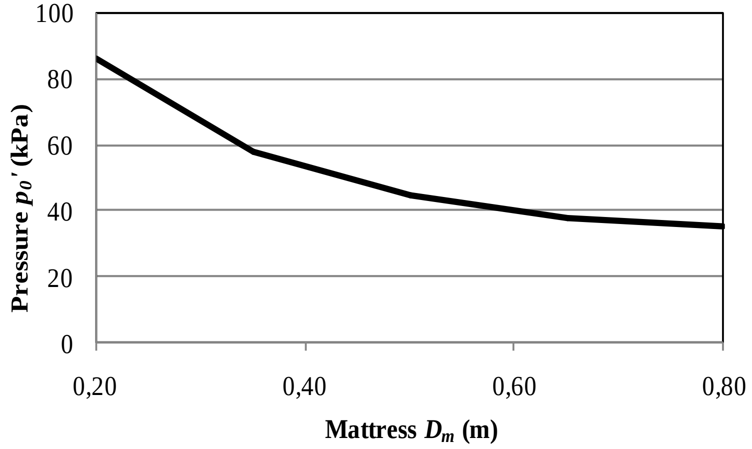


c. Bearing capacity q_{ar} vs spacing s = nd (D_m = 0.50m)

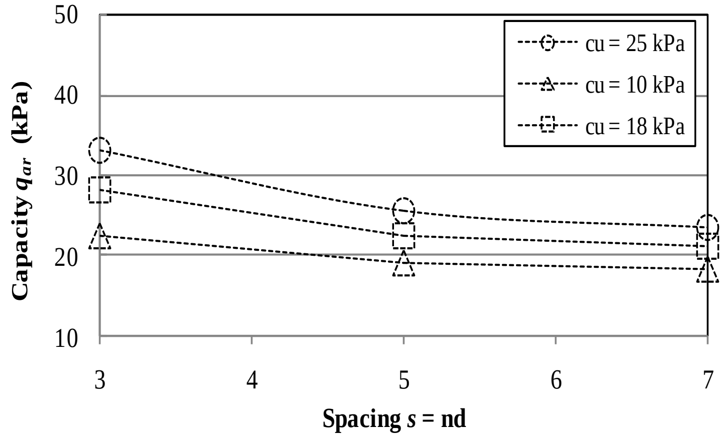


d. Bearing capacity q_{ar} vs spacing s = nd (D_m= 0.80m)

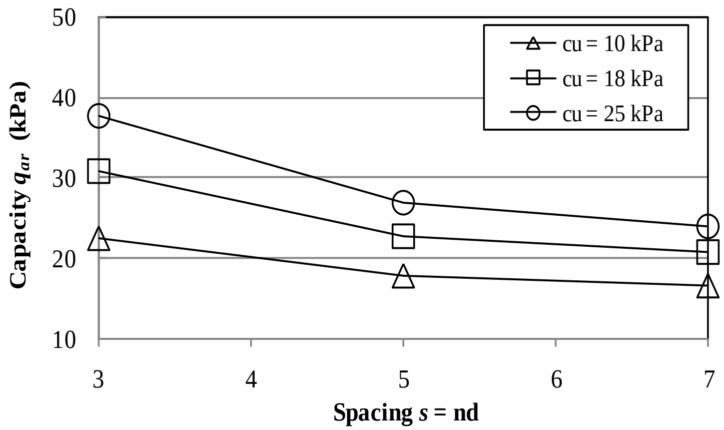
Figure 3.6 Results calculated by proposed method (for d = 8 cm, L = 3 m, n = 3, 5, 7)



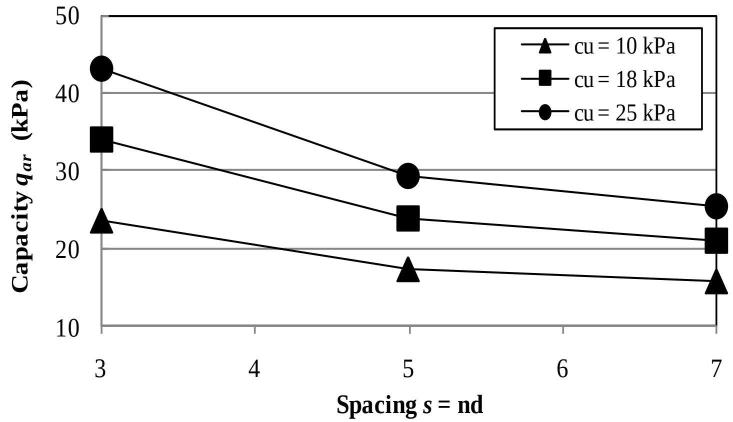
a. Pressure p_0' vs mattress D_m



b. Bearing capacity q_{ar} vs spacing s = nd (D_m = 0.20m)

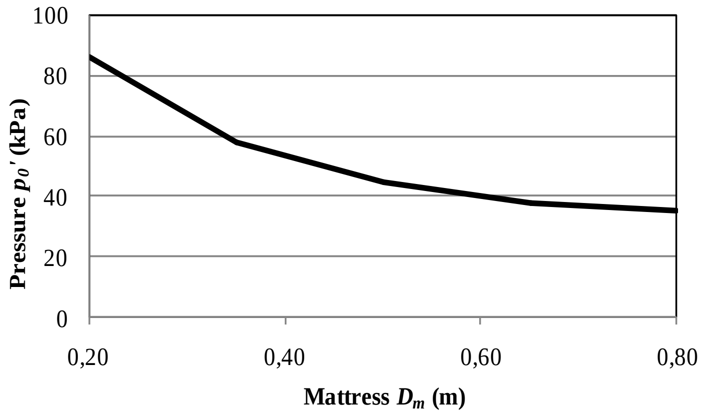


c. Bearing capacity q_{ar} vs spacing s = nd (D_m = 0.50 m)

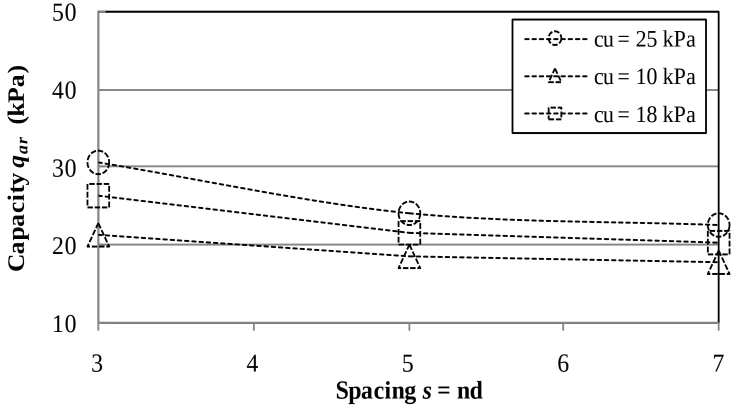


d. Bearing capacity q_{ar} vs spacing s = nd (D_m = 0.80 m)

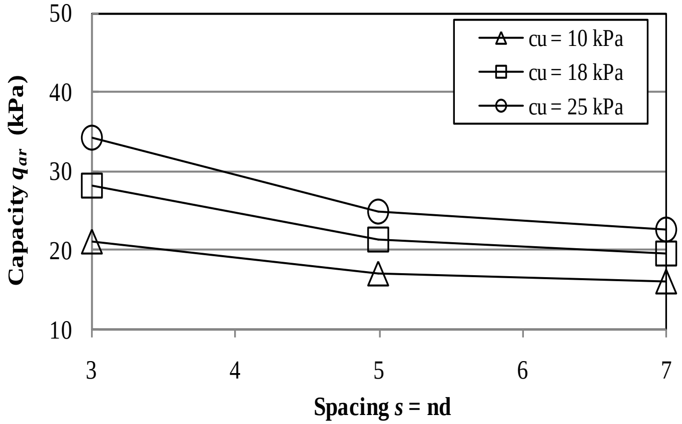
Figure 3.7 Results calculated by proposed method (for d = 8 cm, L = 4.5 m, n = 3, 5, 7)



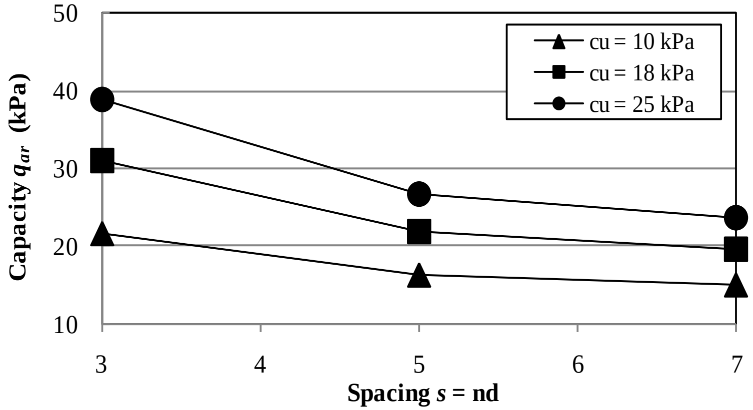
a. Pressure p_0' vs mattress D_m



b. Bearing capacity q_{ar} vs spacing s = nd (D_m = 0.20m)

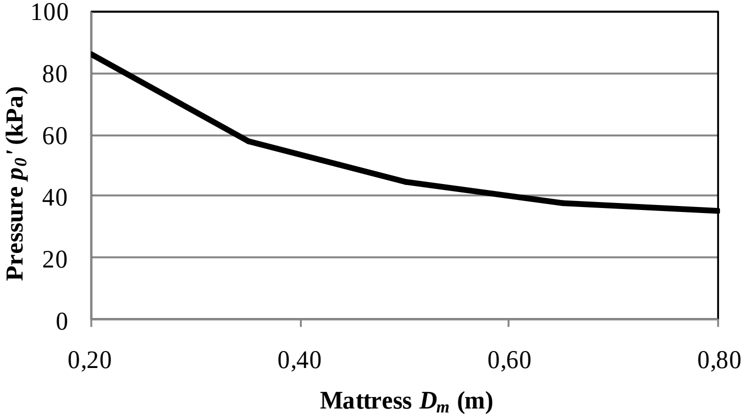


c. Bearing capacity q_{ar} vs spacing s = nd (D_m = 0.50 m)

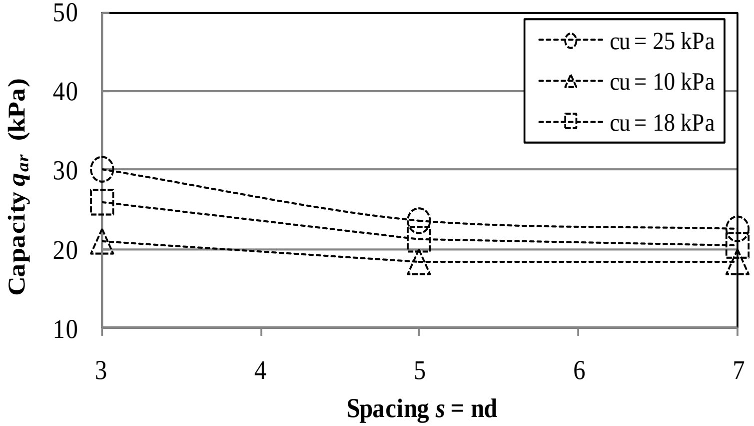


d. Bearing capacity q_{ar} vs spacing s = nd (D_m = 0.80 m)

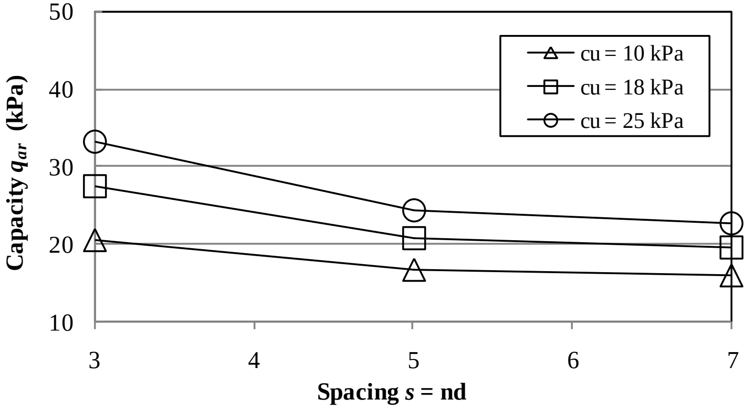
Figure 3.8 Results calculated by proposed method (for d = 10 cm, L = 3.0 m, n = 3, 5, 7)



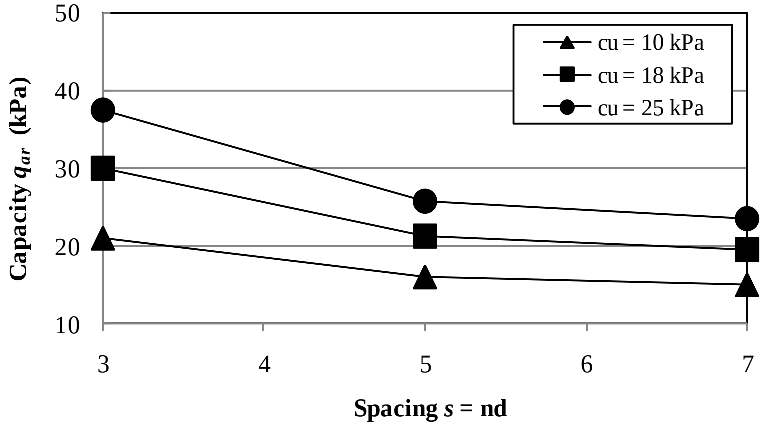
a. Pressure p_0' vs mattress D_m



b. Bearing capacity q_{ar} vs spacing s = nd (D_m = 0.20m)



c. Bearing capacity q_{ar} vs spacing s = nd (D_m = 0.50 m)



d. Bearing capacity q_{ar} vs spacing s = nd (D_m = 0.80 m)

Figure 3.9 Results calculated by proposed method (for d = 10 cm, L = 4.5 m, n = 3, 5, 7)

When the length of the piles was increased (L = 4.5 m), the allowable bearing capacities q_{ar} increased to 43.19 kPa in cohesion c_u of 25 kPa and with spacing of piles of 3d in Figure 3.7.

The criterion of allowable bearing capacity q_{ar} of 38.84 kPa more shows than the load distributed pressure p_0' using only for mattress D_m of 0.80 m on the cohesion c_u of 25 kPa that was improved by the timber piles (with spacing s = 3d, d = 10 cm, L = 3 m). This is shown in Figure 3.8 and Figure 3.9. However, when the length of the pile was increased to L of 4.5 m, the criterion of allowable bearing capacity q_{ar} was sufficient only with c_u of 25 kPa and D_m of 0.80 m.

The design criterion of the allowable bearing capacity of reinforced soft clay is summarised in Table 3.2.

Table 3.2 Summary of bearing capacity q_{ar} (c_u = 25 kPa and D_m = 0.80 m, p_0' = 35.17 kPa)

Set of reinforcement		Bearing capacity, q_{ar} (kPa)	Remark
Geo-grid on piles s = 3d, d = 8 cm	L = 3 m	44.77	$q_{ar} > p_0'$
	L = 4.5 m	43.19	$q_{ar} > p_0'$
Geo-grid on piles s = 5d, d = 8 cm	L = 3 m	30.25	$q_{ar} < p_0'$
	L = 4.5 m	29.29	$q_{ar} < p_0'$
Geo-grid on piles s = 3d, d = 10cm	L = 3 m	38.84	$q_{ar} > p_0'$
	L = 4.5 m	37.51	$q_{ar} > p_0'$
Geo-grid on piles s = 5d, d = 10cm	L = 3 m	26.68	$q_{ar} < p_0'$
	L = 4.5 m	25.89	$q_{ar} < p_0'$

In the calculation with FEA, plastic analysis was applied using the MC model. The parameters of the soft clay and mattress used in the FEA simulations are shown in Table 2.2 and Table 2.3. The mattress laid on top of the timber piles is applied only for the thickness D_m of 0.8 m. To present the material behaviours in the plastic analysis, the parameters of the geo-grid and timber pile are prepared as elasto-plastic materials. The result obtained for the geo-grid reinforcement were axial stiffness $E_{gg}A_{gg}$ of 4.8E+02 kN/m and maximum force N_p in plane applied by the strain ε_{gg} of 5% [3, 6].

For this application of the FEA model of timber piles installed in soft clay with length L of 3 m and 4.5 m, diameter d of 8cm, and spacing s of 50 cm, the results are listed in Table 3.3 [3].

Table 3.3 Parameters of timber pile reinforcement (for d = 8 cm, c_u = 18 kN/m^2)

FEA Simulation	Axial stiffness E_pA_p (kN/m)	Axial forces (kN)		Remark
		F_{comp}	F_{tens}	
1	4.26E+03	28	39	For length L = 3.0 m
2	6.04E+03	42	72	For length L = 4.5 m

In these simulations, construction of the reinforcement is prepared in several stages over a total of 38 days. Then the load pressure p_0 of 205.05 kN/m^2 with footing $2B_0$ of 51 cm is set up quickly at a zero time interval. The results of soil stresses σ beneath the mattress obtained for validation of the empirical method are listed in Table 3.4.

Table 3.4 Comparison of FEA and empirical method results (for c_u =18 kPa, D_m =0.80 m)

Reinforced soft ground	Comparison of results		
	Empirical: Bearing capacity q_{ur} (kN/m^2)	FEA	
		Soil stresses σ (kN/m^2)	Vertical deformation at the mattress (cm)
Timber pile d = 8 cm s = 50 cm, L = 3m	40.5	53.4	10.4
Timber pile d = 8 cm s = 50 cm, L = 4.5m	42.9	67.3	8.2

Table 3.4 shows the results obtained for the soil stresses when reinforced by geo-grid and timber piles in the FEA and empirical methods. For the elasto-plastic models with geo-grid and timber piles in the FEA, it may be affected to increase the bearing capacity in the clay.

3.4 Summary

It is common for the ground near to rivers to be thickly covered with soft clay. Thus, in order to protect against natural hazards, local people need, as a countermeasure, to construct robust embankments by river banks of soft clay with low bearing capacity. To solve the afore mentioned problem, local people have useda traditional reinforcement method using timber or bamboo piles. To establish this method, the Ministry of Public Works has published technical guidelines for the reinforcementof the soft clay before the embankment is built. The guidelines show the process of construction, including 1) cutting the ground for site preparation, 2) installing timber piles, 3) laying geo-grid on top of the timber piles, and 4) spreading and compacting a granular material on the geo-grid.

The guidelines assist engineers to construct embankments on soft ground but unfortunately do not show the details of a reasonable design based on geotechnical engineering. Hence, this paper aims to propose and discuss a criterion for the bearing capacity of reinforced soft clay, calculated by using several empirical calculations. Model cases were simulated for small footings with static loading on the mattress. The resulting calculation will help to determine the criterion for the allowable bearing capacity of reinforced soft clay with timber piles.

In the FEA simulation using the Mohr–Coulomb (MC) model and elasto-plastic material models for comparison of the proposed method, the results from the proposed method are shown to express reasonable behaviour.

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Chapter 4. Classical Design Approach of Indonesian Traditional Reinforced Embankment on Soft Clay Ground

4.1 Introduction

Floods and tidal waves are major natural disasters that occur frequently in Indonesia with a high degree and intensity. To reduce the damage induced by these disasters, people living in coastal areas or on riversides have constructed high river dikes on the soft ground. However, river dikes constructed in the traditional way are not problem-free.

The Ministry of Public Works recorded about 2,150 floods from October 1st, 2009 to February 28th, 2015. Of these, a total of about 1,566 floods took place in the eastern parts of Sumatra (307), northern parts of Java (1,020) and southern parts of Kalimantan (239), as shown in Chapter 1 (see Figure 1.1) [1, 2].

In this section, two cases of river dikes in Riau province are introduced. The first is the Tembilahan River dike located in Indragiri Hilir district, as shown in Figure 1.2, which shows a major access road constructed on the river dike. However, this collapsed over an area 300 m long and 53 m wide due to an annual flood. The failure was triggered by weak foundation soil [3, 4].

The second case is an example of a countermeasure. The local government has built a dike 4 m high and 600 m long along the edge of Siak River to prevent annual flooding. The dike is used as an access road for local people living along the river bank. In the design document, the soft ground of the river side was reinforced with timber piles before construction of the dike (see Figure 1.4) [4].

In order to investigate the properties of both dikes, standard penetration tests (SPT) were conducted at the two locations. It was found that the soft soil was 21 m and 12 m in depth at the Tembilahan River and Siak River respectively (see Table 1.2 and Table 1.3). Furthermore, field and laboratory tests were conducted for the two sites. The soil consistency tests and undrained compression tests were conducted using soil samples. The average plasticity index *PI* was found to be more than 40% and the undrained cohesion of soil c_u was less than 25 kN/m^2 (see Table 1.1 and Table 1.4) [3, 4].

To increase the bearing capacity of the soft ground Q_u, the idea of using bamboo or timber piles was studied by the Ministry of Public Works of Indonesia. They reported the results of full-scale plate loading tests (using ASTM-D-1194-1972) at three sites in West Java (Karawang, Cirebon, and Banjar). The Q_u value of soft ground with bamboo pile reinforcement was increased by about 315 % at the investigation site of Karawang, 242% at Cirebon and 215% at Banjar. However, the bearing capacities obtained can not be applied to other sites, because the report was limited to investigating the bearing capacity of bamboo piles in soft ground, the condition of which varies at each site [5].

A traditional reinforcement method using timber or bamboo piles has been widely used by installing them into soft ground to support an embankment. To understand the technical situation of this reinforcement system, the traditional construction method is first introduced [6].

The Ministry of Public Works of Indonesia has published guidelines for the construction of road embankments, as explained in Figures 1.6~1.10 [6, 7]. Figures 1.9 and 1.10 show the reinforcement in combination with geo-textile and a gravel layer. This combination of geo-textile and gravel layer is referred to as the mattress in the guidelines. The traditional reinforcement method for soft ground is also explained in the guidelines, but they only show the construction procedure, without a detailed explanation from the engineering point of view.

In the guidelines, several criteria for construction are defined for the required performance. An example is the settlement rate of the embankment by soil consolidation per year during construction for a rigid or flexible pavement for a road.

In the guidelines, road constructions on embankments are grouped into (i) rigid pavement constructions for higher importance roads, and (ii) flexible pavement constructions for lower importance roads. The settlement criterion requires less than 20 mm/year for higher importance roads and less than 30 mm/year for lower importance roads [7]. By loading from the embankment, the stresses within the soft ground usually remain in the range of the stability criterion, known as the factor of safety (*Fs*). The guidelines show that the road construction required a factor of safety *Fs* of 1.40 for higher importance roads and *F*s of 1.30 for lower importance roads [7].

In order to determine the appropriate reinforcement for soft ground, it is recommended in the guidelines to construct a full-scale trial embankment, and an outline of the design technique using a trial embankment for road construction is provided [8, 9]. However, the required performance of the river dikes and embankments is different. In the guidelines, two cases of trial models are briefly introduced. In the first, a road embankment construction was planned on peat soils usingthe preloading method without any reinforcement [8]. To determine the stability of the embankment, a stability analysis for a safety factor was conducted using limited equilibrium analysis. The conventional method was adopted to determine the suitable parameters for settlement (see Figures 2.1~2.5) [8].

In the second trial model, shallow reinforcement of soft ground using cement and timber piles for the road embankment construction was introduced [9]. Generally, a mixed soil–cement method has been used for road construction on embankments, with the height of the embankment up to 3 m without pile reinforcement [9].

However, in the guidelines, this type of construction requires the height of the embankment to be 3 m or more. Therefore, soil–cement mixing with various thicknesses and numbers of timber piles was applied for the required settlement and bearing capacity. When the monitored settlement rate is within the range of the guidelines, the engineer can continue construction while continuing monitoring to check on the settlement. Furthermore, according to the guidelines, finite element analysis (FEA) is required to simulate the settlement for the target embankment construction using the monitored data. Finally, a design ofembankment with traditional reinforcenent is obtained by a trial construction atthe site and using FEA. However, it is difficult for local engineers to conduct FEA because they do not have the necessary software and skills.

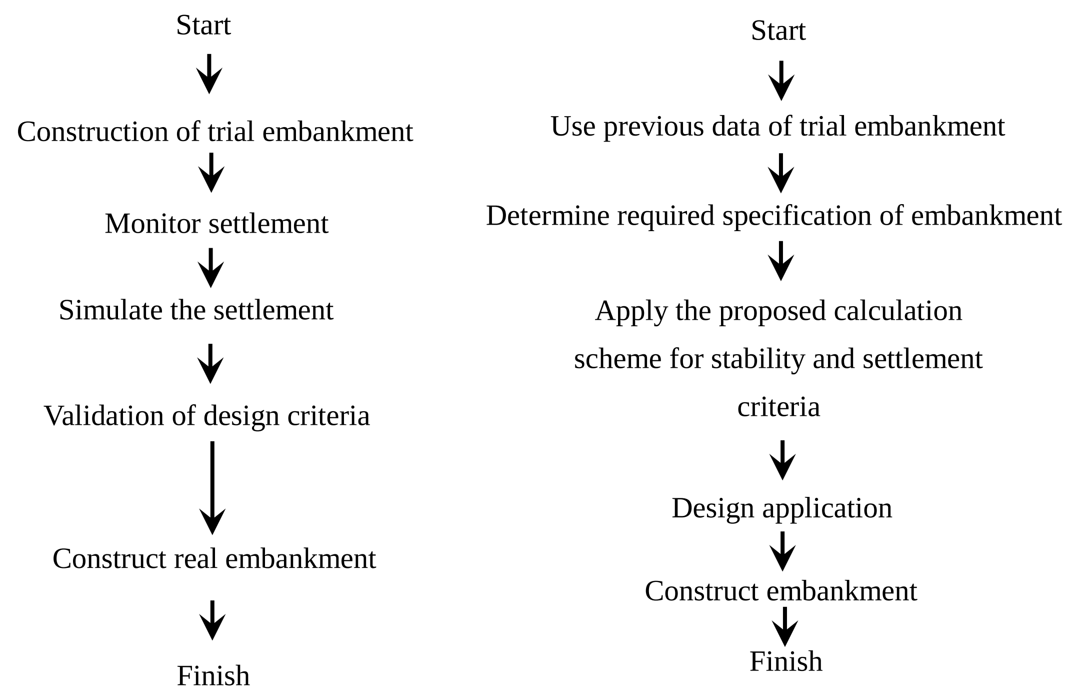
The traditional reinforcement method of soft ground is still popularly used by Indonesian local engineers and government. Therefore, an appropriate design for the stability and settlement of embankments is required based on a reasonable calculation scheme [10, 11].

4.2 Proposed Research Method

For the new calculation scheme, the dataset from a previous trial construction is used to evaluate the proposed method for design embankments. The flow chart of the proposed research is shown in Figure 4.1.

Figure 4.1(a) shows the conventional design method in the guidelines for constructing a road embankment on soft ground using traditional reinforcement [7]. Before starting on a real construction, engineers usually work to construct a trial embankment to monitor the settlement.

Figure 4(b) shows the flow of the proposed calculation scheme for the design of an embankment, derived from several empirical equations. This is a simple calculation method to offer local engineers in Indonesia.



(a) Guidelines from MPW (b) Proposed method

Figure 4.1 Flow chart of the proposed research

The data from trial embankments are utilised to calculate the required design specification using the proposed calculation scheme [7~9], which is explained as follows.

(i) First of all, determine the required specification of the embankment based on the guidelines, includingthe height (H_b), gradient of the slope (n), top width (B_a) and bottom width (B_b) of the embankment.

(ii) Apply soil properties to calculations from field and laboratory tests.

(iii) Select the properties of the reinforcement including length (L) (maximum 6m), diameter (d) (maximum 12cm), spacing of piles (s) (minimum 2.5d) and unit weight of timber (γ_p) (see Figure 1.7).

(iv) Select a the tensile strength of the geo-textile to be laid on top of the timber piles (see Figure 1.8).

(v) Set the thickness of the mattress as the same depth to which the ground is cut (see Figure 1.9). Then determine the pressure and capacity of the soft soil due to cutting the ground, including the unit weight of the softsoil (γ_s), depth (D_m) and internal friction (ϕ).

Based on this process, all engineers can conduct the proposed calculation scheme for designing the stability and settlement criteria of an embankment on soft clay ground. It is then possible to obtain criteria such as (i) the bearing capacity of the shallow footing, height of embankment and its factor of safety, and (ii) settlement of the embankment and the settlement rate. In the proposed scheme, the maximum design condition of the reinforcement can be obtained.

4.3 Proposed Design Criteria

4.3.1 Design criterion of stability

4.3.1.1 Bearing capacity of reinforced foundation soil

In order to establish a theory for the stability criterion, the data of the trial embankment built in Kalimantan is adopted [10]. In this trial construction on soft clay, the average undrained shear strength of soft clay-1c_{u1} with the installation piles was calculated as for normally consolidated clay. In the proposed calculation method, the position of the heads of piles without caps is assumed to be beneath the mattress layer, as explained in Chapter 2 (see Figure 2.7) [10, 11].

For design considerations, two cases of embankments are investigated:

(i) Case 1, an embankment on soft ground reinforced with geo-textile,

(ii) Case 2, an embankment on soft soil using the traditional reinforcement system, both as detailed in Table 4.1 [6, 7].

Table 4.1 Reinforcement system for foundation soil

Reinforcement system	Timber pile	Geo-textile	Mattress
Case 1	Without	With	Gravel
Case 2	With	With	Gravel

The vertical deformations at the centre beneath the embankments, constructed based on the guidelines in three stages for the two cases (see Figures 2.18 and 2.19), were monitored for 98 days.

Settlement plates were placed on the ground surface under the embankment to measure the trial embankment. Therefore, the difference between the reinforcement methods for soft ground can be distinguished as below.

1. Case 1, mattress (gravel on geo-textile), then fill embankment.
2. Case 2, timber piles, mattress laid on top of the timber piles (without caps), then full embankment.

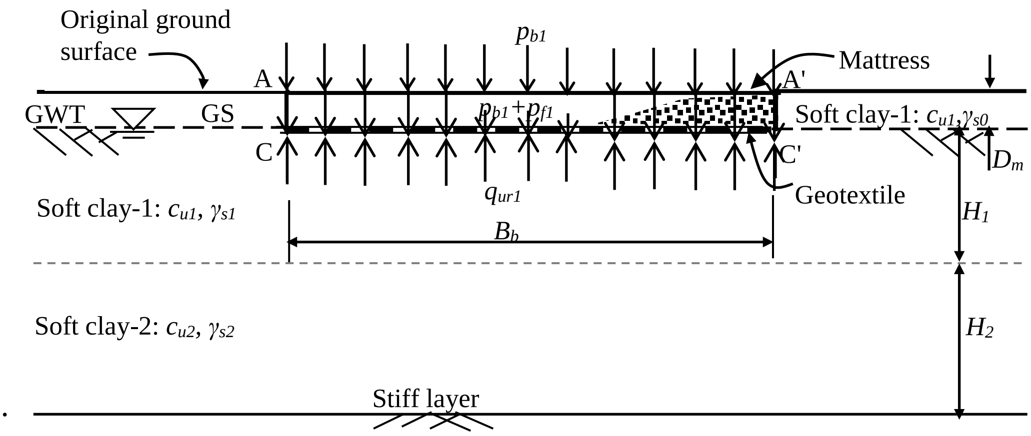
For Case 1, the loading pressure of embankment p_{b1} on the mattress is expressed in Figure 4.2. For the load of the embankment with the mattress as a foundation the required criterion is given by (Figure 4.2(a)) [12, 13]:

$$p_{b1} + p_{f1} \leq q_{ur1} \tag{4.1}$$

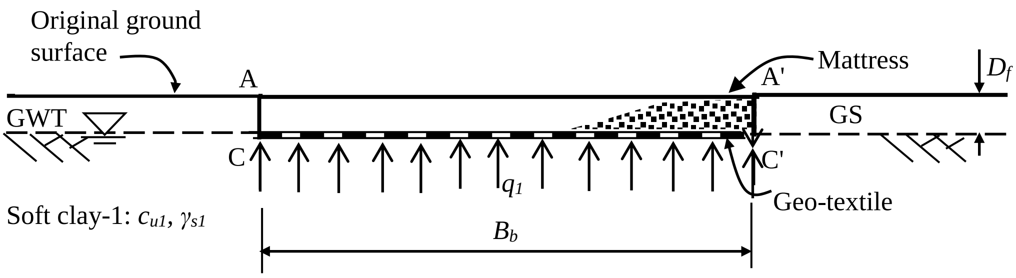
in which the mattress pressure as foundation p_{f1} is calculated as

$$p_{f1} = \gamma_m D_m \tag{4.2}$$

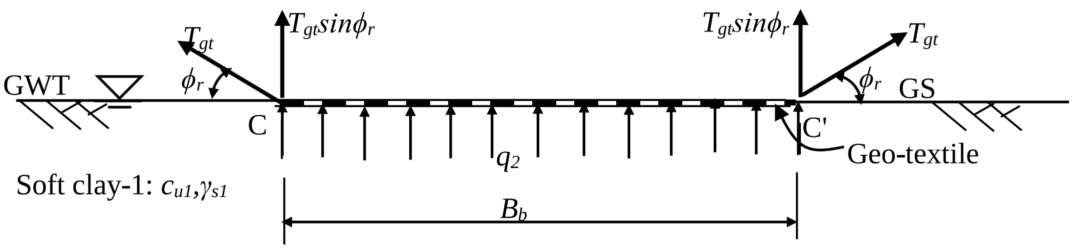
in which γ_m is the unit weight of the mattress, D_m is the thickness of the mattress and is assumed to be equal to the depth of ground cutting.



(a) Loading pressure represented by gravel layer with geo-textile



(b) Baring capacity of the soft clay ground



(c) Bearing capacity of geo-textile

Figure 4.2 A simple model of analytical solutions for foundation of gravel reinforced with geo-textile (mattress)

The material properties of the embankment are listed in Table 4.2.

Table 4.2 Material properties of embankment [10]

Soil fill layer	Unit weight γ_b (kN/m^3)	Internal friction ϕ_b (^{o})	Young's modulus E_b (MPa)
Embankment	19	33	10

The loading pressure from the filled embankment on the mattress p_{b1} is calculated by

$$p_{b1} = \gamma_b H_b \tag{4.3}$$

The ultimate bearing capacity of the mattress q_{ur1} for width of footing B_b (at points A-A') is defined as (see Figure 4.2(a)) [14]

$$q_{ur1} = q_1 + q_2 \tag{4.4}$$

in which q_1 is the bearing capacity of the soft clay ground and $q_2$is the bearing capacity of the geo-textile.

The bearing capacity of the soft clay ground q_1 with depth D_f for width of footing B_b in Figure 4.2(b) (at points A-A') is calculated by [15, 16]

$$q_1 = c_{u1} N_c + \gamma_{s0} D_f \tag{4.5}$$

where N_c is the factor of the bearing capacity for soft clay ground, found to be N_c= 5.14, and γ_{s0} is the unit weight of soft clay-1 above the ground surface (GS) [16].

In this case, the mattress with thickness D_m is assumed to deform δ_a due to loading from embankment. So, the depth of mattress D_f can be simplified as

$$D_f = D_m + \delta_a \tag{4.6}$$

where δ_a is the allowable vertical deformation for this Case 1.

Figure 4.2(c) shows a simple model for determining the tensile capacity of the geo-textile q_2 per unit meter square for width of footing B_b (at points C-C')[14, 17]:

$$q_2 = \frac{2T_{gt}\sin\phi_r}{B_b} \tag{4.7}$$

$$B_b = B_a + 2nH_b \tag{4.8}$$

where ϕ_r is the interface friction between the geo-textile and the mattress at C-C'.

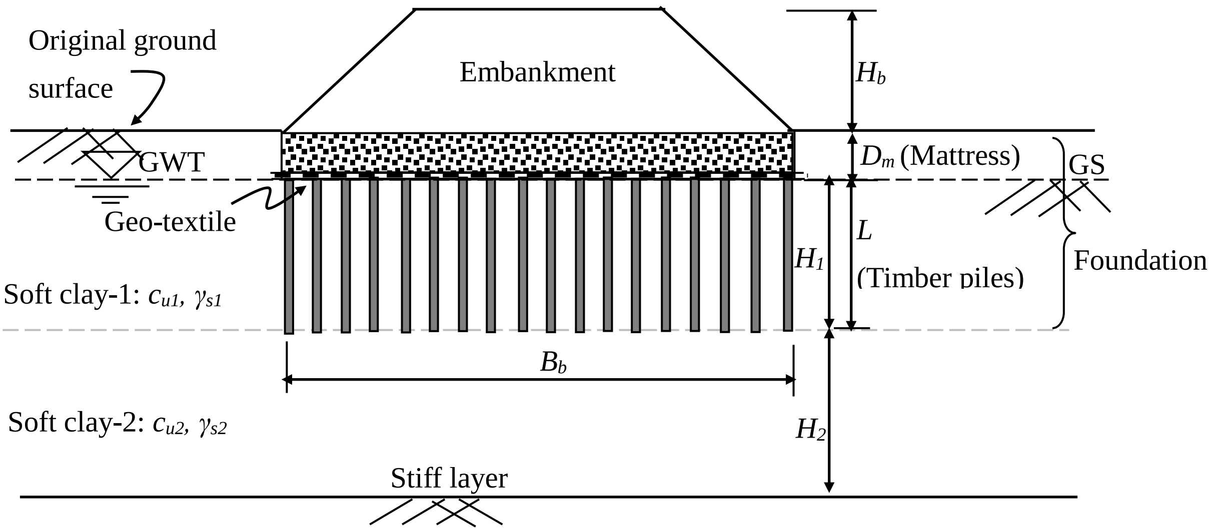
For Case 2, in order to calculate the bearing capacity of the soft clay with timber piles, the reinforced area is assumed as a rigid block. The traditional reinforcment system is expressed in Figure 4.3, in which there are some restrictions: (i) the soft clay beneath the timber pile installation is assumed as a uniform layer, (ii) the design does not consider the increasing shear strength of subsoil due to consolidation, and (iii) the tensile capacity of geo-textile q_2 on timber piles is not used in this case by no deformation ($q_{gt} \approx 0$).

Figure 4.3 (b) shows the embankment construction on the reinforced area. The required design criterion is expressed as [18, 19]

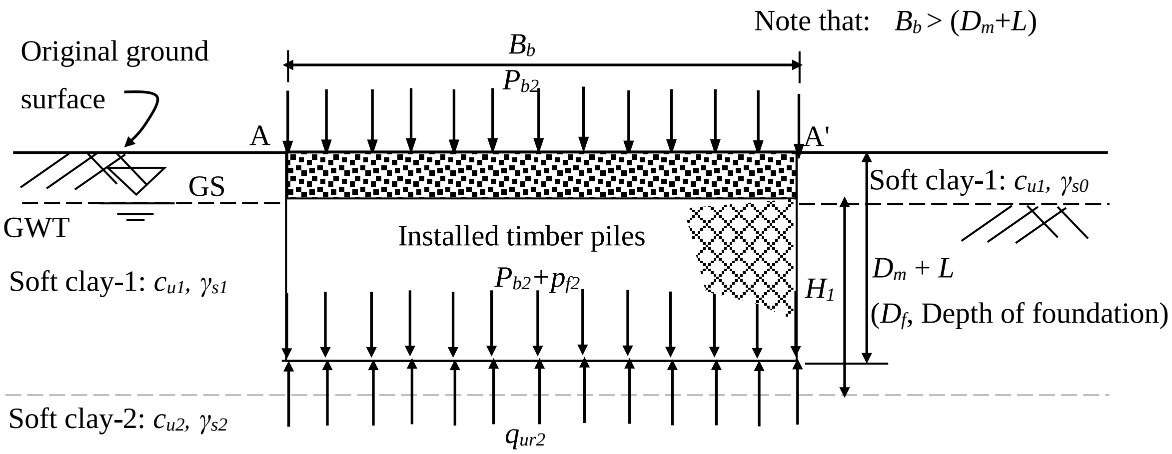
$$p_{b2} + p_{f2} \leq q_{ur2} \tag{4.9}$$

The load pressure of the embankment (p_{b2}) and the pressure of the foundation (p_{f2}) distributed to the ground surface are proposed by

$$p_{b2} + p_{f2} = \gamma_b H_b + \left(\gamma_m D_m + \gamma'_{sp} L\right) \tag{4.10}$$



(a) A traditional foundation system for soft clay with timber piles and mattress



(b) A simplified traditional foundation system as a shallow foundation

Figure 4.3 A traditional foundation with timber piles and simplified foundation for proposed calculations

Using Terzaghi–Peck's classical formula for the long strip footing width B_b, as the shallow foundation by $(D_m + L) / B_b < 1.0$ [15,16].

in which the (bulk) unit weight of soft clay with piles γ'_{sp} is calculated by (see Figure 4.3(b))

$$\gamma'_{sp} = \left[1 - n_p\left(\frac{\pi d_e^{\ 2}}{4s^2}\right)\right]\gamma'_{s1} + n_p\left(\frac{\pi d_e^{\ 2}}{4}\right)\gamma_p \tag{4.11}$$

where n_p is the number of piles driven per meter square ($n_p = 1.0$), γ_p is the unit weight of the timber and γ'_{s1} is the effective weight of soft clay-1.

The ultimate bearing capacity of the mattress supported by timber pile q_{ur2} at A-A' can be proposed by [14]

$$q_{ur2} = c_{u2}N_c + \gamma_{s0}D_m + \gamma'_{s1}H_1 \tag{4.12}$$

where N_c is the factor of the bearing capacity for the soft clay ground, γ_{s0} is the unit weight of soft clay-1 above the ground surface (GS), D_m is the depth of the mattress, and H_1 is the length of timber piles embedded in the clay.

4.3.1.2 Allowable height and safety factor

The allowable height at failureH_{ar} and safety factor Fs for the embankments of the two cases are explained as follows.

For Case 1, Equation (4.4) is substituted into Equation (4.3) and the allowable height H_{ar1} can then be calculated by [18, 20]

$$H_{ar1} = \frac{q_{ur1}}{Fs_0\gamma_b} \tag{4.13}$$

where Fs_0 is the required safety factor [7].

Therefore, after reinforcing with geo-textile beneath the embankment, the safety factor Fs_1 of the embankment is calculated by

$$Fs_1 = \frac{q_{ur1}}{p_{b1}} \tag{4.14}$$

In Equation (4.14), the height of embankment (H_b) is assumed to be equal to the allowable height of the embankment.

For Case 2, the allowable height of embankment H_{ar2} is defined as

$$H_{r2} = \frac{q_{ur2}}{Fs_0 \gamma_b} \tag{4.15}$$

The safety factor of the embankment supported by timber piles Fs_2 is predicted by

$$Fs_2 = \frac{q_{ur2}}{p_{b2}} \tag{4.16}$$

where the load pressure from the embankment p_{b2} ($p_{b1} = p_{b2}$) [7].

4.3.2 Settlement criterion

4.3.2.1 Total settlement

The total settlement of the ground surface beneath the embankment Δh_r for Case 1 (Δh_{r1}) and Case 2 (Δh_{r2}) will be presented below.

For Case 1, the total settlement of the ground surface beneath the embankment Δh_{r1} can be obtained by

$$\Delta h_{r1} = \delta h_{em} + \delta h_0 \tag{4.17}$$

The vertical deformations in Equation (4.17) are presented by using the empirical calculations as follows.

The deformation of the mattress δh_{em} is calculated by (4.18) [21].

$$\delta h_{em} = \frac{[B'\sigma_{bm1} I_0 (1-\upsilon_s)]}{E_s} \tag{4.18}$$

where I_0 is the influencing factor of the load distibution, B' is the width spreading of the loading pressure, σ_{bm1} is the vertical pressure, υ_s is the Poisson's ratio of the subsoil, and E_s is the Young's modulus of the subsoil.

The vertical pressure of the embankment σ_{bm1} on the ground surface can be predicted by [22]

$$\sigma_{bm1} = \gamma_b H_b \left(\frac{B}{B'}\right) + \gamma_m D_m \tag{4.19}$$

in which the width spreading B' due to the pressure of the embankment at the ground surface is defined as

$$B' = B + 2\tan\beta \times D_m \tag{4.20}$$

The average width of the embankment B is given by

$$B = B_a + nH_b \tag{4.21}$$

where B_a is the width at the top of the embankment, n is the gradient of the embankment slope, and H_b is the height of the embankment.

The Osterberg chart with a function f(a, b, z) for the influencing factor of load distribution I_0 is defined as[24]

$$I_0 = \frac{2}{\pi}\left[\left(\frac{(a+b)}{b}\right)(\alpha_1 + \alpha_2) - (\frac{a}{b})(\alpha_2)\right] \tag{4.22}$$

where α_1 and α_2 are load distribution factors of the embankment.

The load distribution parameters α_1 and α_2 are calculated respectively as

$$\alpha_1(rad) = \tan^{-1}\left(\frac{(a+b)}{z}\right) - \tan^{-1}\left(\frac{a}{z}\right) \tag{4.23}$$

$$\alpha_2(rad) = \tan^{-1}\left(\frac{a}{z}\right) \tag{4.24}$$

where a is the half-width of the embankment, b is the width slope of the embankment side, z is the depth of the soft clay, υ_s is the Poisson's ratio of the soil, β is the angle of loading distribution, and c_u is the soil undrained cohesion [23].

The Young's modulus of the soil E_s is defined as $210c_u$ [21].

The primary settlement by consolidation of the soft clay δh_0 beneath the mattress of the embankment is usually calculated as [8]

$$\delta h_0 = \frac{C_c}{1+e_0}\log_{10}\frac{\sigma'_{v01}+\Delta\sigma_{v1}}{\sigma'_{v01}}(H_2+H_1) \tag{4.25}$$

where C_c is the compression index, e_0 is the initial void ratio of the soil, H_1 is the compression of soft clay-1, and H_2 is the compression of soft clay-2.

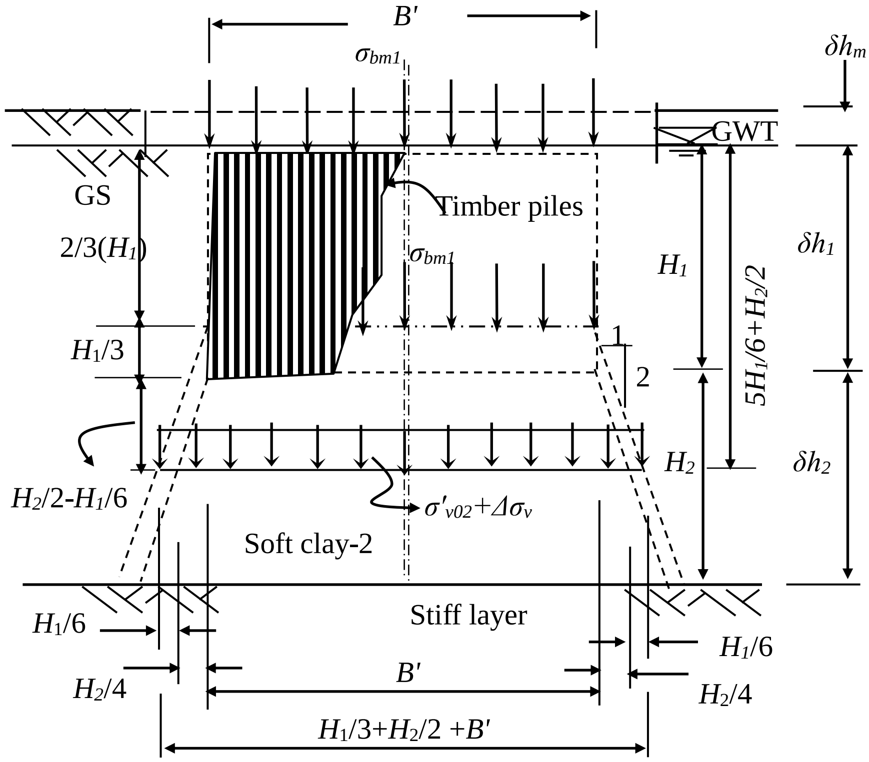
The design parameters of the primary settlement by consolidation of the soil are taken into account from laboratory test data.

The average effective overburden pressure due to the softclay σ'_{v01} for upward drainage is defined as

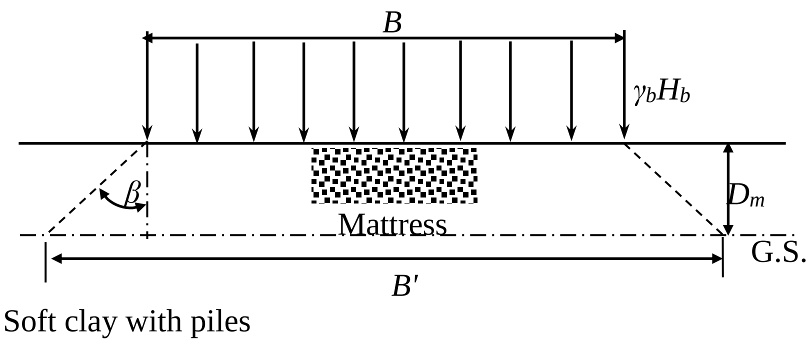
$$\sigma'_{v01} = \frac{(H_1+H_2)\gamma'}{2} \tag{4.26}$$

where γ' is the effective unit weight of the soft clay.

For Case 2, the definition of parameters for consideration in the design of the embankment on soft clay using the traditional reinforcement system is shown in Figure 4.4.



(a) Cross-section of embankment



(b) Load pressure distribution of the mattress

Figure 4.4 Definition of parameters for consideration in design of settlement [23]

From Figure 4.4, the total settlement of the ground surface Δh_{r2} beneath the embankment can be proposed by

$$\Delta h_{r2} = \delta h_m + \delta h_1 + \delta h_2 \tag{4.27}$$

where δh_m is defined as the deformation of the mattress layer ($\delta h_m \approx 0$), δh_1 is the elastic deformation of soil improved by the timber piles, and δh_2 is the primary consolidation settlement of soft clay without piles.

In Equation (4.27), the elastic deformation of the reinforced area δh_1 is predicted by

$$\delta h_1 = \frac{\sigma_{bm1} H_1}{E_{sp}} \tag{4.28}$$

The Young's modulus of the soil with timber piles E_{sp} is predicted as [21]

$$E_{sp} = a_p E_p + (1 - a_p) E_s \tag{4.29}$$

The ratio of the soil reinforcement area with pile a_p is given by

$$a_p = \frac{A_p}{s^2} \tag{4.30}$$

where A_p is the cross-section area of the pile ($A_p = \pi d^2/4$), E_p is the Young's modulus of the timber piles, and s is the spacing of the piles.

The primary settlement by consolidation of the soft clay beneath the piles installation area δh_2 can be predicted by [8]

$$\delta h_2 = \frac{C_c}{1+e_0} \log_{10} \frac{\sigma'_{v02} + \Delta\sigma_{v2}}{\sigma'_{v02}} \left(H_2 + \frac{H_1}{3} \right) \tag{4.31}$$

in which the total pressure at the middle depth of the soft soil is defined as $\sigma'_{v02} + \Delta\sigma_{v2}$.

The average effective overburden vertical pressure due to soft clays σ'_{v02} for upward drainage is defined as

$$\sigma'_{v02} = \gamma'_{s2}\left[H_1 + \frac{H_2}{2}\right] \tag{4.32}$$

The increasing vertical pressure $\Delta\sigma_{v2}$ due to the embankment at the middle depth of H_2 can be proposed as

$$\Delta\sigma_{v2} = \frac{B'(\sigma_{bm1} I_0)}{[H_1/3 + H_2/2 + B']} \tag{4.33}$$

where H_2 is the thickness of the soft clay beneath the timber pile installation in this case.

4. 3.2.2 Degree of consolidation

The rates of soil consolidation for the soft clay sector beneath the mattress in Case 1 and for only the soft clay sector beneath the installation pile in Case 2 are calculated. The rate of soil consolidation correlated time factor T_v for both cases is predicted by [8]

$$T_v = \frac{C_v t}{H_d^{\ 2}} \tag{4.34}$$

where C_v is the coefficient of consolidation of clays, taking its parameters from laboratory tests, and t is the consolidation time.

For Case 1, the thickness of the soil in single drainage H_d is given by

$$H_d = \frac{(H_1 + H_2)}{2} \tag{4.35}$$

where H_1 is the thickness of soft clay-1, and H_2 is the thickness of soft clay-2 in this case.

For Case 2, the thickness of the soft clay in single drainage H_d is given by (see Figure 4.4(a))

$$H_d = \frac{H_2}{2} \tag{4.36}$$

where the thickness of soft clay-1 is the same as the length of the timber piles embedded in it.

The calculation result of the time factor T_v in Equation (4.34) can be used to determine the degree of consolidation of soft clays U_z for all of the soils [8].

When the time factor T_v is less than 0.2, the degree of consolidation of soft clays U_z is calculated by

$$U_z = \sqrt{\frac{4T_v}{\pi}} \tag{4.37}$$

while, when the factor T_v is more than or equal to 0.2, U_z is calculated by

$$U_z = 1 - \left(\frac{8}{\pi^2}\right) \exp(-\frac{1}{4}\pi^2 T_v) \tag{4.38}$$

where the time factor T_v is given in decimal units.

4.3.2.3 The settlement rate

Therefore, the settlement rate of consolidated clays Δh_t considered for the two cases can be calculated respectively below.

For Case 1, the settlement rate of consolidated clays Δh_t is defined as

$$\Delta h_t = \delta h_m + \delta h_0 \times U_{z1} \quad (4.39)$$

For Case 2, the settlement rate of consolidated clays with timber piles Δh_{t*} is defined as

$$\Delta h_{t*} = \delta h_1 + \delta h_2 \times U_{z2} \quad (4.40)$$

where U_{z1} is the degree of consolidation considered for all of the clay layers in Case 1, and U_{z2} is the degree of consolidation considered for soft clays beneath the timber pile installation.

4.4 Comparison with Finite Element Analysis (FEA)

In order to compare the results of the proposed empirical calculation method, the FEA is employed as explained below.

4.4.1 Making geometryof the embankment

A representation of the geometrical model of the embankment on the ground was shown in Chapter 2 (see Figure 2.2). For simulations in FEA, the boundary conditions of the geometrical model of embankments on soft clay are set as shown in Figure 4.5.

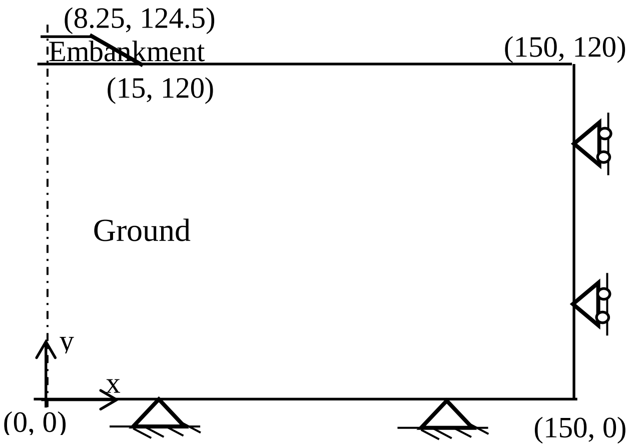
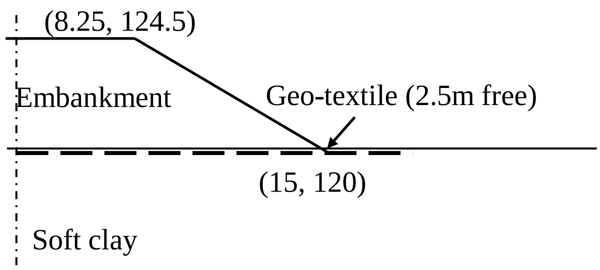
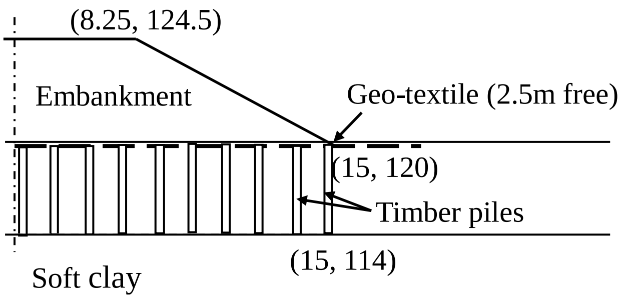


Figure 4.5 Half of geometrical model of embankment on soft clay

Figure 4.5 shows the two-dimensional geometry of the embankment model created by a symmetric construction with a coordinate system (x,y). In general, the units of length, force and time are based on the SI standard. The boundary conditions of the embankment are defined by x and y fixed on the bottom boundary, and x fixed on the side boundary.



(a) Embankment model of Case1



(b) Embankment model of Case 2

Figure 4.6 Modelled embankment construction in the two cases

Figure 4.6 shows the geo-textile sheet laid at the ground surface. The both tips of geo-textile on the toes are free with length on 2.5 m for the two cases. The coordinate points of the timber pile installation are connected by a line to enable mesh generation for modelling the anchor.

4.4.2 Parameters of the material model

In order to conduct investigations for laboratory tests, soil samples were taken from the site of a trial embankment. From these soil sampling results, soft soil layers were identified as shown in Table 4.3. The parameters of the soil material of the embankment are shown in Table 4.4.

Table 4.3. Soil properties and strength parameters used for soft clay in FEA

Material [10]		Parameters used in FEA					
Soft clays:		Soft clay				Medium	Hard
Depth (m)	Unit	0 – 4	4 – 6	6 – 12	12 – 18	18 – 25	25 – 120
For plastic analysis:							
Mohr–Coulomb model		MC	MC	MC	MC	MC	MC
Type undrained for all soil layers							
For consolidation analysis:							
Modified Cam-Clay model		MCC	MCC	MCC	MCC	MCC	MC
Type drained for all soil layers							
Soil unit weight:							
Unsaturated γ_{unsat}	(kN/m^3)	12	12	13	15	16	16.5
Saturated γ_{sat}	(kN/m^3)	14.5	14.5	15	16	18	20
Cohesion c_u	(°)	10	12	20	25	30	1
Friction ϕ_s	(°)	5	8	12	14	16.5	30
Dilatancy ψ	(°)	0	0	0	0	0	0
Permeability k_x	(m/day)	1.38E-03	1.38E-03	1.38E-03	1.38E-03	1.38E-03	1.0
Permeability k_y	(m/day)	6.89E-04	6.89E-04	6.89E-04	6.89E-04	6.89E-04	2.0
Stiffness E_s	(kN/m^2)	2,100	2,250	4,200	5,250	6,300	8,000
Poisson's ratio υ_s	-	0.35	0.35	0.35	0.35	0.35	0.35
Init. v. ratio, e_{init}	-	2.2	2.2	2	1.8	1.5	-
Coef. consol C_c	-	0.9	0.9	0.85	0.6	0.4	-
Plasticity *PI*	(%)	42.11	42.11	42.33	35.26	29.7	-
Compression λ	-	0.525	0.525	0.500	0.384	0.290	-
Swelling κ	-	0.087	0.087	0.082	0.069	0.057	-
Poisson's ratio υ_{ur}	-	0.15	0.15	0.15	0.15	0.15	0.15
Slope of critical state line M	-	0.179	0.292	0.447	0.772	1.199	-

Table 4.4 Soil properties and strength parameters used for embankment in FEA

Material [10]	Unit	Soil parameters used in FEA	
		Plastic analysis	Consolidation analysis
Model		MC	MC
Soil type		Drained	Drained
Bulk density: Unsaturated γ_{unsat}	(kN/m^3)	19	19
Saturated γ_{sat}	(kN/m^3)	20	20
Cohesion c_b	(kN/m^2)	1	1
Friction ϕ_b	(°)	33	33
Dilatancy ψ	(°)	3	3
Permeability k_x	(m/day):	2	2
Permeability k_y	(m/day):	1	1
Soil stiffness E_b	(kN/m^2)	10,000	10,000
Soil Poisson's ratio υ_s	-	0.35	0.35

Table 4.3 shows the parameters of the soft clay materials. The plasticity index *PI* for all of the soil layers is predicted by [25]

$$PI = 102.3 \times \left(\frac{C_c}{(1+e_0)} \right) + 13.34 \tag{4.41}$$

In the FEA simulation, the parameters of soft clay with a hard layer and embankment were prepared for the Mohr–Coulomb failure criterion.

For calculating the plastic behaviour, the Modified Cam-Clay material model is used for normally consolidated clay.

The parameters of the material model are calculated, including the compression index λ and swelling index κ, which are assumed to apply plane strain with the Mohr–Coulomb criteria. The swelling index κ of the soil can be interpreted for initial prediction as [26]

$$3.0 \le (\lambda / \kappa) \le 8.0 \tag{4.42}$$

The slope of the critical state line *M* in the plane strain model is defined as [26]

$$M = \frac{6 \sin \phi_s}{(3 - \sin \phi_s)} \tag{4.43}$$

where ϕ_s is the internal friction of soil, as obtained from the triaxial compression test.

Using a mechanism of transferred load for the timber pile cluster from the embankment, the constant spring k is calculated as [27]

$$k = \frac{F_{pc}}{\delta} \tag{4.44}$$

in which δ is the deformation of the timber pile cluster $\delta = d_e/10$, and d_e is the diameter of the timber pile cluster [27].

The capacity force of a pile F_{pc} is defined as

$$F_{pc} = P_{all} \tag{4.45}$$

The allowable bearing capacity of the timber pile cluster P_{all} in the soft clay is defined as [19]

$$P_{all} = \frac{1}{F_0}\left\{(f_s c_u)A_s + (9c_b)A_p\right\} \tag{4.46}$$

The axial stiffness of the timber pile cluster driven in soft clay $E_{pc}A_{pc}$ is calculated as

$$E_{pc}A_{pc} = k \times H_1 \tag{4.47}$$

where f_s is the friction factor between the soil and pile [19], c_u is the soil cohesion along the pile, c_p is the soil cohesion at the timber pile tip, A_s is the area of timber pile friction, A_p is the area of the cross-section of the timber pile tip, and Fo is the safety factor of the timber pile cluster.

4.4.3 Setting calculation scheme for FEA

In the calculation scheme, the initial conditions are calculated with the initial pore water pressure,which comes from the water level at the ground surface. The calculation steps are started by creating a model of the ground (see Figure 4.5), drawing a model of the geotextile at the ground, installing piles into the ground, drawing a model of the embankment and calculating the consolidation (see Figure 4.6). The calculation scheme then continues to the next step.

The diameter of a timber pile (d) is defined by input parameters such as the axial stiffness, compression force, and tension force in the FEA. The load-bearing capacity of pile driven soft clay is calculated by the friction of the pile area circumference P_s in Equation (A.1) and the load end bearing capacity of pile tip P_b in Equation (A.2) (see AppendixA).

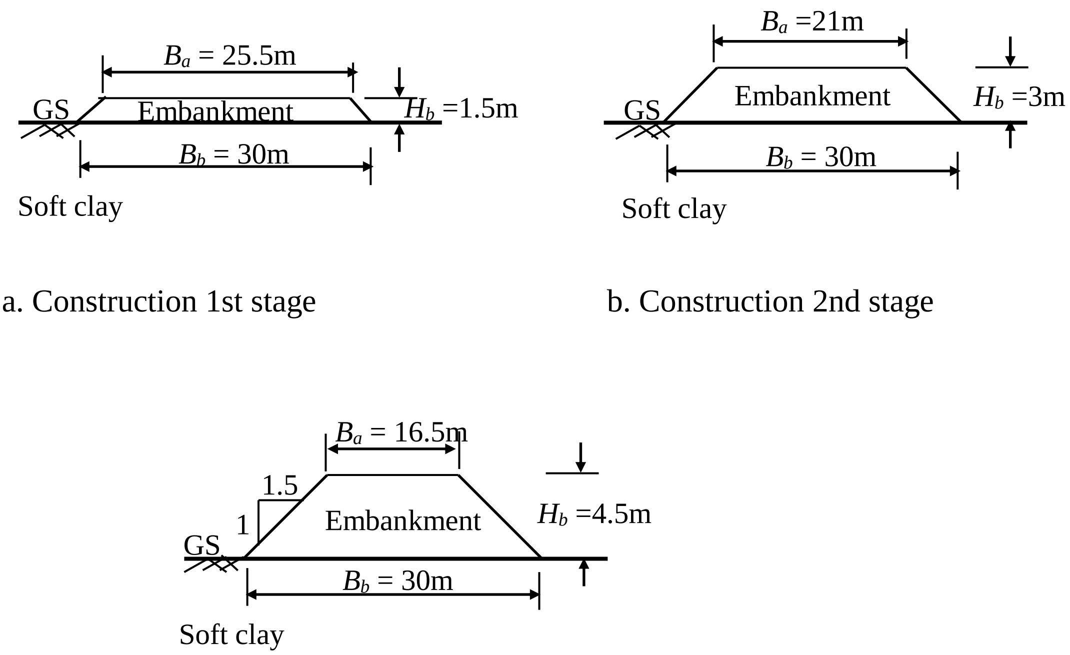
The load end bearing capacity of pile P_b driven in soft clay is calculated from the soil cohesion (c_b) and area of the cross-section of the timber at the pile tip (A_b). However, the bearing capacity of pile tip P_b by the timber pile cluster with equivalent diameter d_{eb} of $\sqrt{3}\times d$ is given small account (see Figure 2.7). Therefore, the load-bearing capacity of the timber pile cluster driven in soft clay can be used in the same way as a single timber pile in the two-dimensional FEA.

4.5 Results and Discussion

The dataset of the trial construction of an embankment at East Kalimantan is applied by using parameters and dimensions such as the width at crest B_a of 16.5 m, height H_b of 4.5m, and B_b of 30 m.This trial construction was without a mattress layer (D_m = 0), and with a slope n of 1.5 (see Figure 2.7), with the following trial construction stages (see Figures 2.18 and 2.19).

The ground layer is revealed to have the thickness of soft clays H_s of 18 m, which consists of the soil properties of the soft clay-1 down to a depth z of 6 m, resulting in c_{u1} of 11kN/m^2, and γ_{s1} of 14.5kN/m^3. Then, on the basis of soft clay-2 at depth z of 6~12 m, c_{u2} of 16 kN/m^2 is revealed [10, 11]. In the empirical method, the timber pile clusters were installed in the soft clay with spacing s of 1.0 m, diameter for friction d_{ef} of 25 cm (d = 10 cm) (see Figure4.5(b)), length L of 6 m, and unit weight γ_p of 1.1 kN/m^3. The geo-textile has an applied tensile strength T_{gt} of 55 kN/m, elongation to failure 5%, and the friction of the mattress ϕ_m of 35^o

[10,11]. The cross-sections of the trial embankments for the two cases are shown in Figure 4.7.



a. Construction 1st stage

b. Construction 2nd stage

c. Construction 3rd stage

Figure 4.7 Cross-section of trial embankment in three stages for the two cases [10, 11]

The input parameters used in the FEA include the axial stiffness of the geo-textile $E_{gt}A_{gt}$ of 1.1E+03 kN/m in an elastic material. The axial stiffness of the timber pile cluster $E_{pc}A_{pc}$ is 2.55 E+04 kN/m and F_p is 42 kN in elasto-plastic material (where d_{ef} = 0.25 m, d_{eb} = 0.17 m (see Appendix A), H_1 = 6 m, f_s = 1.0, c_{u1} = 10 kN/m^2, c_{u2} = 12 kN/m^2, c_b = 20 kN/m^2, A_s = 4.71 m^2, A_b = 0.014 m^2, F_0 = 2.0, and k = 4,242 kN/m).

4.5.1 Stability criterion

The stability criteria calculated with the proposed method for the two cases are obtained as below.

For Case 1, the design parameters and dimensions for the embankment use the mattress friction ϕ_r of 30.6^o. Using the obtained design parameters and dimensions, the result is an ultimate bearing capacity q_{ur1} of 58.41 kN/m^2 (where q_1 = 56.54 kN/m^2, q_2 = 1.87 kN/m^2).

The load pressure distributed at the ground surface p_{b1} is 85.5 kN/m^2. The height of the embankment H_{r1} was found to be 2.37 m (where Fs_0 = 1.30).

For Case 2, the design parameters and dimensions mentioned above for the stability criterion lead to the capacity q_{ur2} of 89.94 kN/m^2, and the total pressure of the embankment p_{b2} and foundation p_{f2} is given as 112.55 kN/m^2 (in which $\gamma'_{s1}$of 4.69 kN/m^3, $\gamma'_{sp}$of 4.52 kN/m^3, and L = 6 m). Thus, the design height of the embankment H_{r2} is predicted to be 3.64 m.

FEA was used to simulate the model embankment construction for the two cases. Therefore, the elasto-plastic soil materials, geo-textile and timber pile clusters are prepared to present the soil behaviours in the plastic analysis. In this application of FEA, the strength parameters model of the timber pile cluster are calculated with the equivalent diameter for frictiond_{ef} of 25 cm, bearing capacity at the pile tip d_{eb} of 17.3 cm (d = 10cm), spacing $s$of 100 cm and length L of 6 m. The timber pile cluster can be assumed to be the same as an equivalent timber pile (see Appendix). The parameters of the geo-textile and the pile model for the timber pile cluster are listed in Table 4.9 [3].

Table 4.9 Input strength parameters of geo-textile and pile models for FEA

Reinforcement method	Geo-textile by axial stiffness $E_{gt}A_{gt}$ (kN/m)	Timber pile cluster d_{ef} = 25 cm, H_1 = 6m		
		Axial stiffness $E_{pc}A_{pc}$ (kN/m)	Axial forces (kN)	
			F_{comp}	F_{tens}
Case 1	1.1E+03	-	-	-
Case 2	1.1E+03	2.55E+04	106	280

In this plastic analysis, all the clays are treated as undrained for constructing the embankment with height H_b of 4.5m over 98 days. The soil stresses σ beneath the mattress obtained for validation of the empirical method are listed in Table 4.10, which shows that the total soil stresses σ beneath the embankment with height of 4.5 m are $\sigma_1$of 42 14 kN/m^2 for Case 1 and σ_2 of 78 kN/m^2 for Case 2. The results obtained are less than the ultimate bearing capacity of the soft clay reinforced by the traditional reinforcement.

Table 4.10 Comparison of FEA and empirical method results

Reinforcement method	Simulation results		Remark
	Empirical	FEA	
	Bearing capacity q_{ur} (kN/m^2)	Soil stresses σ (kN/m^2)	
Case 1	q_{ru1} = 58.4	σ_1 = 42	Reinforced geo-textile
Case 2	q_{ru2} = 89.9	σ_2 = 78	Reinforced geo-textile and timber pile clusters

The safety factors Fs_1 for Case 1 andFs_2 for Case 2 are given in Equations (4.13) and (4.15). An embankment with a height H_b of 4.50 m has Fs_1 of 0.68 and Fs_2 of 0.80 for Case 1 and Case 2, respectively.

4.5.2 Settlement criterion

The settlement criteria of the two cases are obtained by the empirical method as detailed below.

For Case 1, the elastic vertical deformation δ_{hm} of 0.494 m is calculated. The parameter sand dimensions of the trial embankment are σ_{bm1} = 85.5 kN/m^2, I_0 = 0.884, elastic modulus of soil E_s of 2,310 kN/m^2 and Poisson's ratio of soft clay υ_s of 0.35 [10]. The primary settlement of clay beneath the embankment δh_0 is found to be 2.26 m, which is calculated from the total settlement rate of the time period t= 98 days. This results in Δh_t of 0.96 m at the consolidation rate of soft clay U_{z1}= 19 %, and T_{v1}= 0.028.

For Case 2, the vertical deformation of soft clays with the timber pile cluster δh_1 is found to be 0.005 m, with E_{sp} = 1.00E+05 kN/m^2, E_p = 2.0E+06 kN/m^2, a_p = 2.35E-02 m^2, andA_p = 2.35E-02 m^2 [28]. The primary settlement by consolidation of the soft clay beneath the timber pile δh_2 is obtained as 1.16m based on the parameters C_c = 0.81, e_0 = 1.94, σ'_{v02} = 56.28 kN/m^2, $\Delta\sigma_{v2}$ = 56.21 kN/m^2, c_v = 2.8E-02 m^2/day, γ'_{s2} = 4.69 kN/m^3, H_2 = 12.0 m, U_{z2} = 31.2% andT_{v2} = 0.077 [29]. Using the parameters listed above, the total vertical settlement rate Δh_{t*} up to 98 days is found to be 0.37m. The results of the empirical method and FEA methods for the two cases are shown in Figures 4.8 and 4.9.

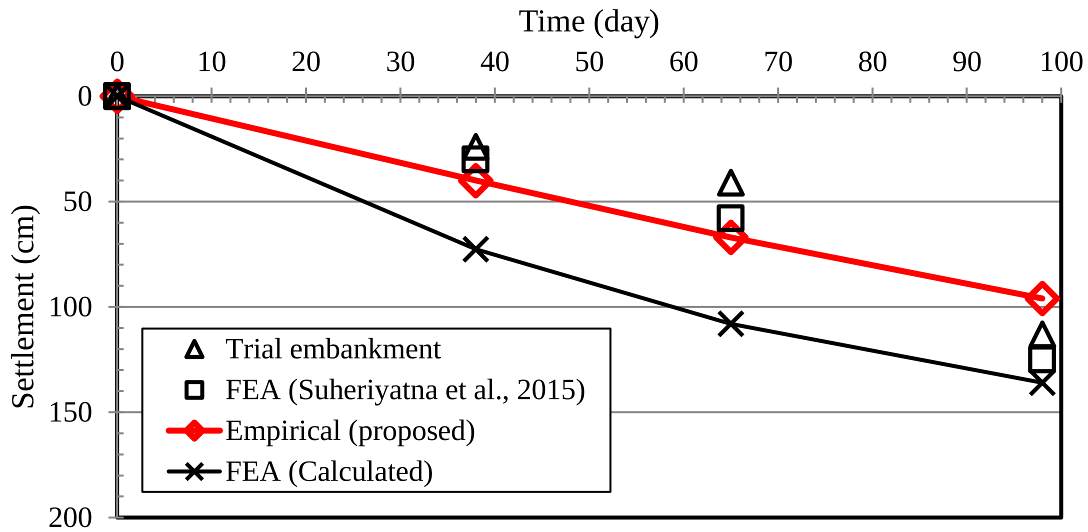


Figure 4.8 Calculated settlement by consolidation of soft clay reinforced by geo-textile up to 98 days

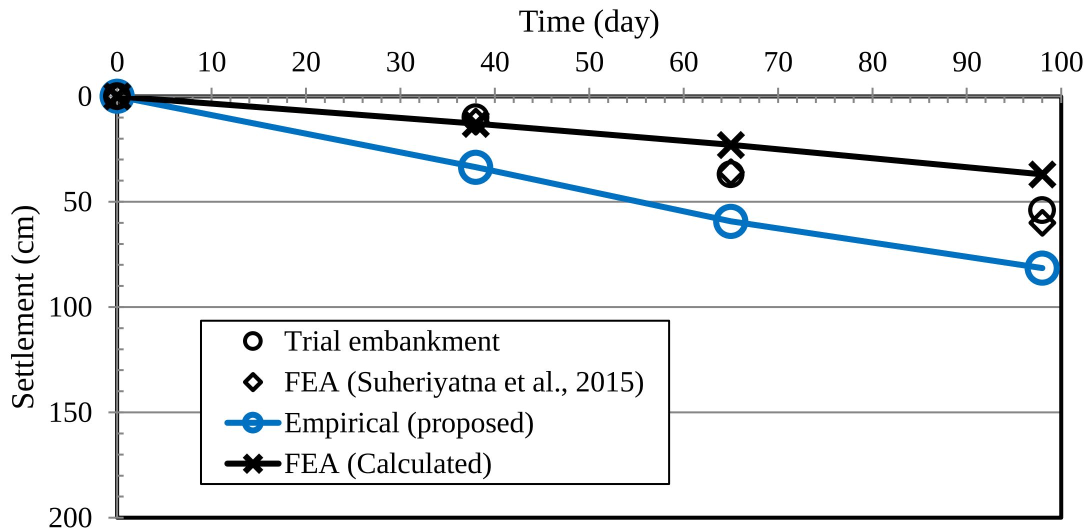


Figure 4.9 Calculated settlement of soft clay reinforced with geo-textile and timber pile cluster

The final settlement of the two embankments is found to be 0.94m at the consolidation degree U_{z2} of 90% for Case 2, and 1.73 m at the consolidation degree U_{z1} of 68% over 3 years for Case 1. During pile installation in submerged soft clays, there might be affect of dissipation the excessive water pressure around the pile shaft. The progress of the degree of consolidation with drainage leads to quicker consolidation than with general clay. The progress of the total settlement on the trial embankment is faster than in general clay.

In the empirical method, the degree of consolidation U_z on the soft clays beneath the pile area was considered for the rate of settlement. The increasing load pressure for the construction stage was also considered for the drainage.

In the FEA, the material properties of the soft clay and embankment for the two cases are listed in Table 4.3 and Table 4.4. In order to calculate the elastic and plastic behaviours of the soil, the Modified Cam-Clay model was chosen in the FEA with a drained condition. In the calculation scheme, the consolidation and the construction stage of loading are taken into account. From Figures 4.8 and 4.9, the calculation results of the settlement rate at 98 days using the empirical and FEA methods are summarised in Table 4.11.

In the report by the Indonesian Ministry of Public Works, the settlement at the ground beneath the embankments is 1.13 m at 98 days in Case 1 and 0.54 m in Case 2 [10].

Table 4.11 Summary of settlement rate of embankments at 98 days

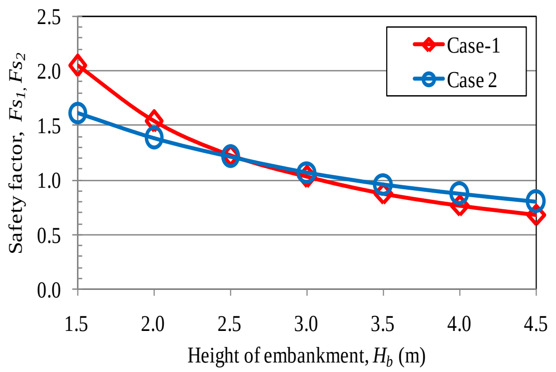
Reinforcement system	Total settlement Δh_t (m)			
	Reported [10]		Simulation results	
	Trial	FEA	Empirical	FEA
Case 1	1.13	1.25	0.96	1.36
Case 2	0.54	0.60	0.37	0.82

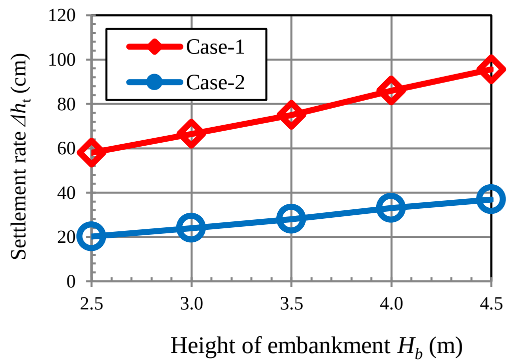
In the FEA simulation, the soil of the embankment is modelled as elastic material. The timber pile cluster is modelled as equivalent to a single pile model, with elasto-plastic material (see Table 4.9). The settlement rates for 98 days were obtained as 1.25 m for Case1 and 0.60 m for Case2 in the FEA of the report[10].

The calculation result with FEA for Case 1 is found to be 1.36m, which is larger than the settlement trial data, while the settlement for Case2 is 0.82m, which is also larger than the trial data.

In the results of the empirical calculations, the embankment constructions of the two cases for 98 days can be designed with the required criteria through the relationship between the height H_b and factor of safety Fs, as well as the relation with the settlement rate Δh_t, as shown in Figure 4.10.

Figure 4.10 shows the obtained relation of the required design criteria for the height of embankment using the traditional reinforcement system can be used to roughly predict the total settlement of the target embankment.





a. Relationship between height H_b and Fs b. Relationship between height H_b and Δh_t

Figure 4.10 Graphs of calculated results for embankments by proposed method

4.6. Summary

The classical calculation method using empirical consideration for traditional reinforcement of embankments is presented. The advantages of this proposed method are as follows:

(i) it uses a simple dataset such as soil parameters from laboratory soil tests, in-situ tests, timber pile parameters, and geo-synthetic parameters;

(ii) it is easy for local engineers to follow and to handle;

(iii) it may save time and construction cost because the engineer is not required to perform a trial construction;

(iv) it is easy to apply to design the embankment practically.

With this proposal, a reasonable design of construction with the required stability and settlement criteria is available to design an embankment on soft soil using the traditional reinforcement. In the case of ane embankment construction with height H_b of 4.50 m, the settlement can be predicted as 0.37 m with a 1.04 factor of safety *Fs* at 98 days. Therefore, the safety factors and allowable settlement can be evaluated for the target embankment.

In the FEA simulations for the trial embankments, the embankment was modelled as an elastic material. The timber pile cluster was modelled as a single pile, and the soft clay was modeled as elasto-plastic material. The total settlements were reported to be 1.25 m for the embankment on soft clay reinforced with geo-textile and timber piles and 0.60 m without timber piles [10].

The simulation results obtained for the vertical deformation are smaller than the trial embankment with timber piles. However, the vertical deformation obtained is larger than the monitored trial dataset for the embankment without timber piles. The effect of water dissipation around the timber piles on the consolidation and increment of shear strength is not considered in this study.

With this simulation, the mechanisms of traditional reinforcement in reinforced soft clay can be investigated using physical equations. A suitable empirical calculation method can be used to estimate the values of the parameters and the design.

Finally, in practical design, local engineers and the government may find that this proposed method provides good tools for reinforcement to prevent natural disasters. Construction data for the proposed scheme at other sites will be required in order to ensure greater accuracy for providing the design criteria for different embankments.

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Chapter 5. Conclusions

In this research, an empirical calculation scheme is used to design the traditional reinforcement system in Indonesia. Several conclusions can be drawn as follows.

The first study considers the allowable bearing capacity of the gravel layer on soft clay with geo-grid supported by timber piles due to static loading from the full weight of a truck. The results obtained lead us to conclude that piles installed with small spacinghave the effect of increasing the allowable bearing capacity. Selecting a pile diameter of 8cm and spacing of 3*d* is good for a construction with the allowable bearing capacity of reinforced soft clay as required in the guidelines.

The second study presents the proposed method using empirical calculation for the traditional reinforcement system for embankments on soft clay. The advantages of the proposed calculation method include the following:

1) it uses a simple dataset of soil parameters from laboratory soil tests, in-situ tests, and timber pile parameters;
2) it is easy for local engineers to follow and to handle;
3) it may save time and construction cost because the engineer does not have to perform a trial construction;
4) it is easy to apply to design an embankment practically.

With this proposal, a reasonable design of construction with therequired stability and settlement criteria is available for the design of an embankment on soft soil using traditional reinforcement. For an embankment constructed with a height H_b of 4.50m, the settlement can be predicted as 0.37 m with a 1.04 factor of safety *Fs* at 98 days. Therefore, the safety factor and the allowable settlement can be evaluated for the target embankment.

In the FEA simulations for trial embankments, the embankment was modelled as elastic and plastic materials. The timber pile cluster was modelled as equivalent to a single pile, and the soft claywas modelled as elasto-plastic material.The total settlements were reported to be 1.25 m for the embankment on soft clay reinforced in the traditional way and 0.60 m without timber piles [10].

The simulation results of the vertical deformation obtained were smaller than the monitored data from the trial ofan embankment with timber piles. However, the vertical deformation obtained was larger than with the dataset of the monitored trial of embankments without timber piles. The effect of water dissipation around the timber piles on the consolidation and increment of shear strength is not considered in this study.

Using the simulation, the mechanisms of traditional reinforcement for soft clay can be investigated by means of reasonable physical equations. A suitable empirical calculation method can be used to estimate the values of the parameters and the design.

Finally, in practical design, local engineers and the government may find that the proposed method provides good tools for reinforcement to prevent natural disasters. Construction data for the proposed scheme from other sites will be required to ensure greater accuracy in providing design criteria for embankments.

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Appendices

A. Calculation of equivalent diameter of timber pile cluster

The expression of the equivalent diameter of the timber pile cluster (three piles) d_e is shown in Figure A1.

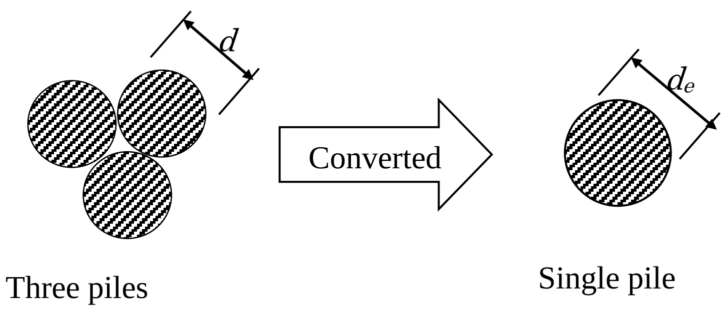


Figure A.1 Timber pile cluster converted into single timber pile for modelling in FEA

Figure A.1 shows that the equivalent diameter of the timber pile cluster is separated into two parts.

(i) For the friction of the timber pile cluster, the equivalent diameter of skin friction for the three piles d_{ef} can be defined as S_{ef}:

$$S_{ef} = 3 \times 2\pi r \tag{A.1a}$$

$$\pi d_{ef} = 3 \times \frac{5}{6}\left(2\pi \frac{d}{2}\right) \tag{A.1b}$$

$$\pi d_{ef} = \frac{5}{2}\pi d \tag{A.1c}$$

$$d_{ef} = 2.5d \tag{A.1d}$$

where d is the diameter of a single timber pile.

(ii) For the bearing capacity of the timber pile tip, the equivalent diameter at pile tip d_{eb} can be defined as follows:

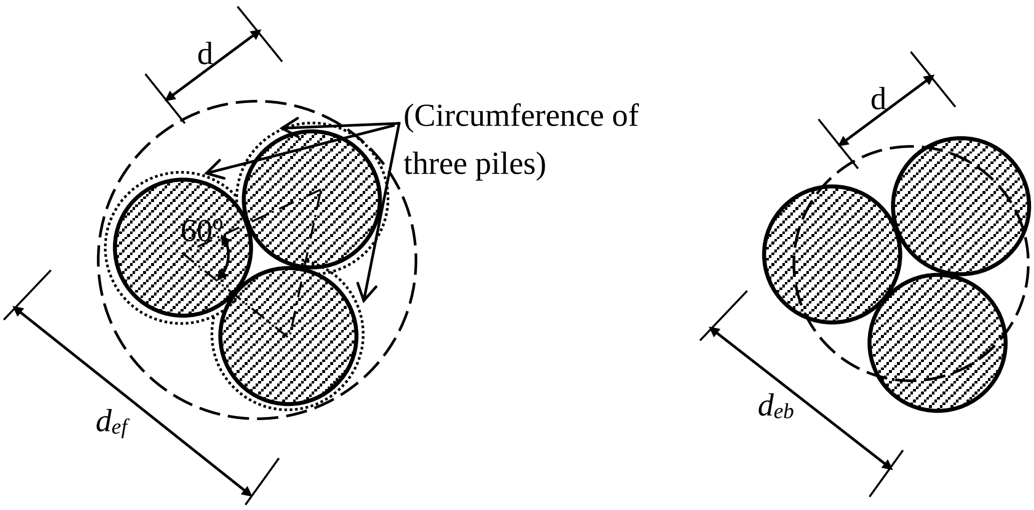
$$A_{eb} = 3A_p \tag{A.2a}$$

$$\frac{1}{4}\pi d_{eb}^{\ 2} = 3\left(\frac{1}{4}\pi d^2\right) \tag{A.2b}$$

$$d_{eb}^{\ 2} = 3d^2 \tag{A.2c}$$

$$d_{eb} = \sqrt{3} \times d \tag{A.2d}$$

where A_p is the area of the cross-section of a timber pile tip and d is the diameter of a single timber pile.

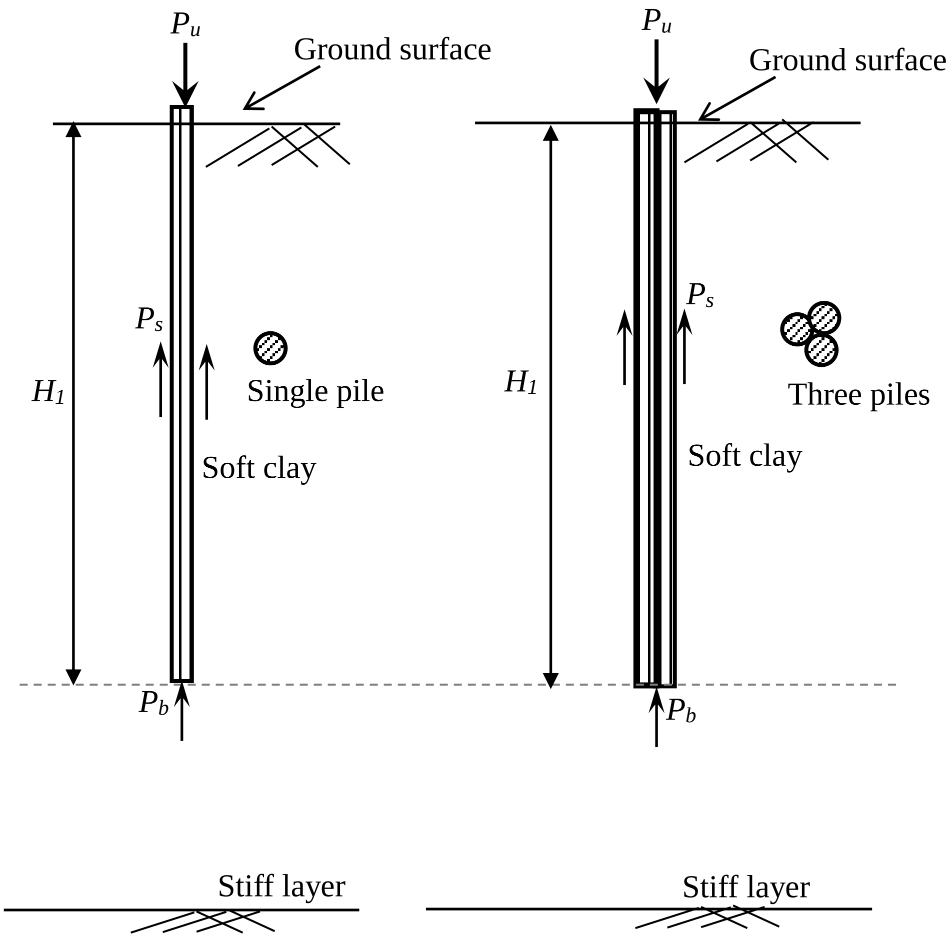


a. Equivalent diameter for pile friction d_{ef} b. Equivalent diameter for pile tip d_{eb}

Figure A.2 Definition of equivalent diameters of pile cluster for pile friction and pile tip capacities

B. Prediction of ultimate load-bearing capacity of timber pile

The expressions of the ultimate load-bearing capacity of the timber pile and timber pile cluster are shown in Figure B.1.



(a) Bearing capacity of single timber pile (b) Bearing capacity of timber pile cluster

Figure B.1 Mechanism of ultimate load-bearing capacity of the timber piles

In practical calculation, the ultimate load-bearing capacity of timber pile P_u is defined as

$$P_u = P_s + P_b \tag{B.1}$$

The friction capacity of timber pile P_s is calculated as

$$P_s = (f_s s_u) A_s \tag{B.2}$$

The area of the skin friction of atimber pile $A_s$and timber pile cluster A_{es} are defined respectively below.

For a single timber pile, it is defined as

$$A_s = \pi d \times H_1 \tag{B.3}$$

and for a timber pile cluster

$$A_{es} = \pi d_{ef} \times H_1 \tag{B.4}$$

where d_{ef} is the equivalent diameter for friction, f_s is the friction factor between the clays and pile and s_u is the undrained shear strength of clay along pile A_{es} (see Figure 1.5).

The friction factor f_s for the pile has a correlationwith the undrained shear strength s_u. It is reported by Kulhawy and Jackson (1989), after Coduto (1994).

The bearing capacity of timber pile tip P_b is defined as (Coduto, 1994)

$$P_b = (9c_p) A_b \tag{B.5}$$

The area of a cross-section of a pile tip A_b is

$$A_b = \frac{1}{4} \pi d_{eb}{}^2 \tag{B.6}$$

where c_p is the soil cohesion at the pile tip and d_{eb} is the equivalent diameter at the pile tip.

C. Prediction of ultimate capacity of tension for timber pile

The expressions of the ultimate capacity of tension for the timber pile and timber pile cluster are shown in Figure C.1.

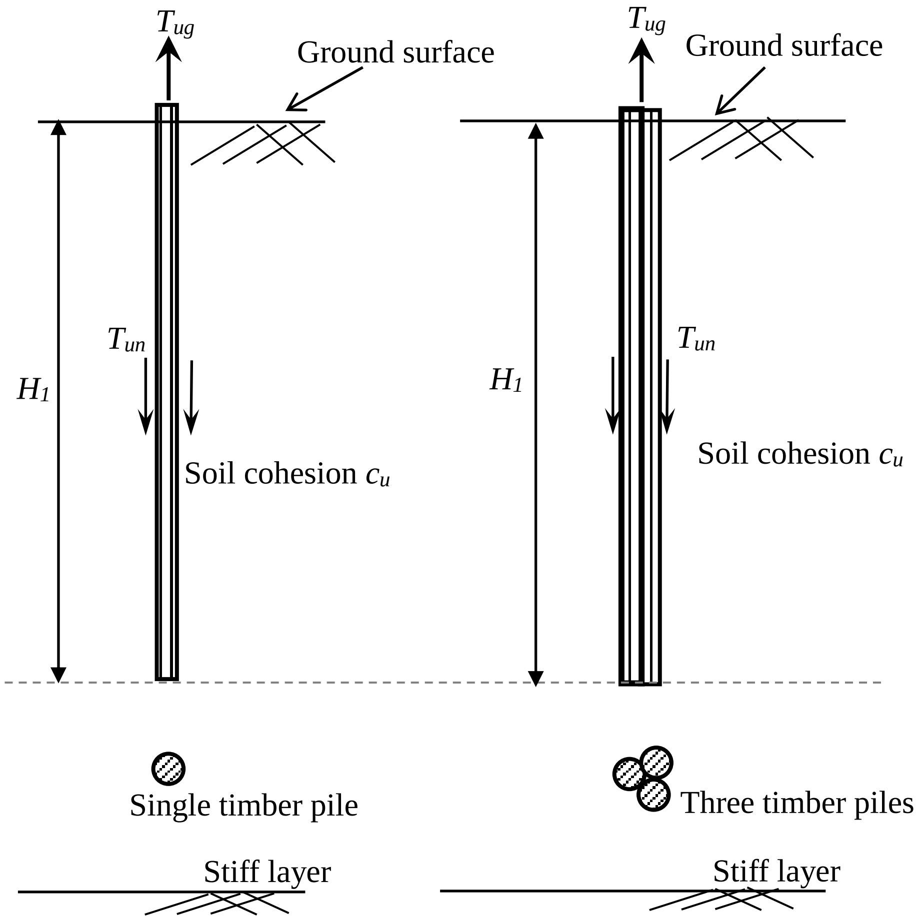


Figure C.1 Mechanism of ultimate tension capacity of timber piles

The ultimate tension bearing capacity of timber pile T_{ug} is calculated by

$$T_{ug} = T_{un} + W_p \tag{C.1}$$

The net ultimate tension bearing capacity of timber pile T_{un} is calculated as

$$T_{un} = H_1 \times p \times \alpha' \times c_u \tag{C.2}$$

where H_1 is the length of the pile, p is the perimeter of the pile section, α' is the adhesion coefficient at the soil–pile interface, c_u is the undrained cohesion of the soil.

The perimeter of the pile section p is defined as follows:

for a single timber pile it is defined as

$$p = \pi d \tag{C.3}$$

and for a timber pile cluster (three piles)

$$p_e = \pi d_{ef} \tag{C.4}$$

$$d_{ef} = 2.5d \tag{C.5}$$

where d_{ef} is the equivalent diameter of the circumference of the timber pile cluster for tension capacity in clay.

The adhesion coefficient at the soil–pile interface α' in the soft clays is considered for the undrained condition, and the undrained cohesion of soft clay c_u for less than c_u of 80 kPa is predicted as (Coduto, 1994)

$$\alpha' = 0.9 - 0.00625 c_u \tag{C.6}$$

The unit weight of timber pile W_p is usually predicted by

$$W_p = \gamma_p \times V_p \tag{C.7}$$

where γ_p is the unit weight of the pile embedded in soft clay and V_p is the volume of the timber pile.