Analysis of the Embankment Stability with Flat Concrete Sheet Pile (FCSP)

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Abstract. Problems of the embankment to be encountered above soft soil layer area are the stability of embankments such as: sliding/landslide, spreading, and sinking due settlement on soft soil layers excessive. The requirement of the safety factor of slope stability at embankment is relatively big, that is (SoF) ≥ 1.70 for approach bridge location and (SoF) ≥ 1.40 for approach bridge location. Safe Number on Model G1 Sheet pile Earthquake Mode is 1.356. So the minimum safe number for earthquake load (SoF) = 1.356 > 1.10 has been fulfilled. The program output produces a maximum moment value of 130.21 kN.m/m. The crack moment capacity of FCSP B 320.500 material is 66.5 kN.m/0.5m. When equivalent in the form of the moment of each meter length then the moment of resistance is 133 kN.m/m. Thus capacity resistance the moment of FCSP able to resist maximum moments, maximum moment value for 130.21 kN.m / m < moment of resistance is 133 kN.m m (safe). However, it is recommended to use minimal FCSP C 320.500 or greater for security reasons. The crack moment capacity of FCSP C 320.500 material is 72.4 kN.m/0.5m equal to 144.8 kN.m/m.

Keywords: Sheet Pile, Crack Moment Capacity, Maximum Moment, Moment of Resistance

1. Introduction

Soil embankment on road approach bridge (under bridge Kali-Beringin) at Km. 444+900 up to Km. 445+100 toll road Semarang–Batang East Java is located above existing river flow Kali-Beringin. The existence of soil embankment causing the river flow should be relocated or created open channels drainage so river discharge capacity is not disturbed. With limited space or right of way (ROW) to accommodate embankment legs and relocate Kali-Beringin, it is necessary to strengthen or add structure. And before the existing river flows are dumped with soil, in the existing river flows is installed sub-drain construction, in the form of stone stack wrapped with geotextile non-woven. The requirement of the safety factor of slope stability at embankment is relatively big, that is (SoF) ≥ 1.70 for approach bridge location and (SoF) ≥ 1.40 for nonapproach bridge location. To increase, the safety factor (SoF) slope stability at embankment can use reinforced sheet pile if available slopes are limited. In addition to the sheet pile, options are available using geotextile or ground cement intended to increase the shear strength of embankment. Requirements for subgrade bearing capacity (SoF) > 3 on embankment load, if based on data sondir / value of cone resistance (qc), where : cu = qc / 20, then get value relationship qc > 1.4xH-embankment (qc with unit of kg/cm2 and H-embankment in unit metre), the relationship is in theory and field practice quite accurate, in some of the projects that have already been implemented.
2. Technical review

2.1 Subgrade Bearing Capacity

Problems of the embankment to be encountered above soft soil layer area are the stability of embankments such as: sliding/landslide, spreading, and sinking due settlement on soft soil layers excessive. To technical review, the above, especially analysis of the embankment stability required a safety factor (SoF) to the collapse that occurred, the value of SoF (safety factor) should be taken minimal 1.50. To prevent spreading (uplift soil) around the embankment, then it is necessary to know the collapse of the soil bearing capacity due to embankment, the first most convenient is to take into account the height of the critical embankment (Hcr), which is show by the formulas (1) and (2) [1].

\[
H_{\text{critical}} = \frac{C_u \times N_c}{Y_{\text{embankment}}} \quad (1)
\]

\[
H_{\text{permit}} = \frac{H_{\text{critical}}}{\text{SoF}} \quad (2)
\]

\(H_{\text{cr}}\) : height of critical embankment (m)
\(H_{\text{permit}}\) : height without causing the collapse (m)
\(C_u\) : undrained cohesion \(C_u = q_c/(15-25)\) → conus resistance \(q_c\), kg/cm².
\(N_c\) : soil bearing capacity factor → 4.0-5.14
\(Y_{\text{emb}}\) : weight content soil embankment (t/m³)
\(\text{SoF}\) : safety factor (1.5-3)

When the height of planned embankment is smaller than the \(H_{\text{permit}}\) embankment, then embankment there will be no collapse or spread around the embankment, will but the process of soil subsidence under the embankment still occurs.

2.2 Embankment Stability Analysis

The embankment stability analysis will use the slice method, can be explained using figure 1, with AC being a circular arch as the surface of the landslide field of the experiment. Land above the plane landslide of the experiment is divided into several vertical slices, the width of each slice not always the same. Notice one thick unit perpendicular to the cross-section of the slope as the style drawing style that works on certain slices shown in figure 2. \(W_n\) is sliced weight. The forces \(N_r\) and \(T_r\) are the parallel components of the \(R\). \(P_n\) reaction and \(P_{n+1}\) are the forces acting on the sides of the slices [2]. Similarly, shear forces which work on the sides of the slices are \(T_n\) and \(T_{n+1}\), to ease the stress pore water is considered equal to zero. Style \(P_n\), \(P_{n+1}\), \(T_n\) and \(T_{n+1}\) is difficult to determine. But we can make the assumption that the resultant \(P_n\) and \(T_n\) is as large as the resultant \(P_{n+1}\) and \(T_{n+1}\) and also the working lines in a line [3]. For balance observation : \(N_r = W_n \cos \alpha_n\). The shear force of resistance can be expressed as follows as shown by the formula (3):

\[
\tau_l(\Delta L_n) = \frac{\tau_l(\Delta L_a)}{F_s} = \frac{1}{F_s} (c + \sigma \tan \theta) \Delta L_a
\]

(3)
The normal stressing $\alpha$ in the above equation as formula (3) is equal to show by the formula (4):

$$N_x = \frac{W_x \cos \sigma_x}{\Delta L}$$

(4)

To balance the style push to point 0 is equal to moment style of resistance to point 0, shown by the formula (5):

$$F_s = \frac{\sum_n (c_1 \Delta L_n + W \cos \alpha - \tan \phi \Delta L)}{\sum_n (W \sin \alpha \Delta L)}$$

(5)

When a part of the surface wall is saturated or there is groundwater level, then the formula above is stated as follows as shown by the formula (6):

$$F_s = \frac{\sum_n (c_1 \Delta L_n + W \cos \alpha - \tan \phi \Delta L)}{\sum_n (W \sin \alpha \Delta L)}$$

(6)

To accelerate the embankment stability analysis of slopes of high embankment the road is used Plaxis program. Simulation performed by Plaxis with phi/c reduction method to determine the factor of safety (FoS). This calculation is done with decreases the shear strength value (c and phi) of soil and subsequently, part of soil that has touched the mohr-columb collapse will experience plastic condition and plastic point form. At this point will experience high incremental strains compared to other points.
that have not experienced plastic. At a point where high incremental strains are generally ribbons or bands. This band on the slice method at the equilibrium limit is called a failure plane or field failure.

Criteria and parameters soil from the data sondir, laboratory test and field conditions as follows:

a) Parameter soil of embankment with CBR value > 6%.
b) The traffic load is taken q = 20 Kpa, equivalent q = 2 t/m2.

2.3 The location of Kali-Beringin Relocation

Here is the layout plan relocation of Kali-Beringin:

![Figure 3: Layout plan relocation of Kali-Beringin](image)

From figure 3 above shows that the old channel from Kali Beringin is passed by the plan toll road Batang-Semarang. In order to function from the river as natural drainage can be maintained, then the long flow channel is relocated to the right of the embankment road (the right side from Batang). Viewed from the geological aspect, the location of the test lies in the Damar Formation (Qtd). In general, the formation is composed of tuffan sandstone, conglomerate and volcanic breccia. At the initial depth of as thick as 5 - 10 meters early is the ground red (residual soil) derived from the Lappish of volcanic material. Based on the information, this location is a stable area and is not encountered geological structure marker (fracture/cesarean).

2.4 Geotechnical Investigation

At the location of the study has been conducted sondir test as much as 4 points on STA. 445+000 and 445+250 on the right and left side of the embankment. Inside drilling data Under Bridge Kali-Beringin structure is used for comparison. The result of the conus resistance value (qc) of the sondir and N-Spt data from boring can correlated to the consistency of clay soil layer as indicated on table 1. given by Begemann (1965) and Schmertmann (1969), as the following:

| Consistence of Clay Soil | Conus Resistance qc (kg/cm²) | N-Spt (blow/30 cm) | Cu (t/m²) |
|--------------------------|-------------------------------|-------------------|-----------|
| Very soft                | <3                            | < 2               | <1,25     |
| Soft                     | 3 – 8                         | 2 – 4             | 1,2 – 2,5 |
| Medium Stiff             | 8 – 20                        | 4 – 8             | 2,5 – 5,0 |
| Stiff                    | 20 – 40                       | 8 – 15            | 5,0 – 10,0|
| Very Stiff               | 40 – 80                       | 15 – 30           | 10,0 – 20,0|
| Hard                     | > 80                          | > 30              | > 20,0    |
Based on the classification of table 1, the results of the sondir readings can be interpreted. In general, based on resistance reading data conus of all four points indicate that at a depth of 2.50 meters began to be discovered good soil layers. Starting at that depth, the average value of qc has reached at least 30-40 kg/cm² with category stiff until very stiff. While at the initial depth between 0-2.50 meters are in the category medium stiff with a qc value of 5-20 kg/cm², shown on figure 4. With the condition of the field where embankment has been implemented, then in the analysis it is assumed that the initial layer of the medium stiff ignored because of the influence of the construction process.

Figure 4: Graphs of Sondir sta.445+000 and sta.445+250

2.5 Geotechnical Analysis

2.5.1. Idealization of Soil Modeling and Parameters

In performing the stability simulation of embankment, the most sensitive parameter is a strength shear soil parameter. With the availability of data in the form of data sondir, then it can be approached against the total shear strength (Undrained Shear Strength) with the following equation as shown on formula (7) [4]:

\[ C_u = \frac{q_c}{N_k} \]  

(7)

qc = conus resistance (kg/cm²)
Nk = coefficient of soil type

The total shear strength value can also be estimated based on the approach in table 1. The elastic modulus value is also required in the simulation. The elastic value of the modulus is not sensitive affect the value of SoF (safety of factor) obtained from the simulation. This condition occurs because the elastic modulus value does not have a big affect on the limit condition state, where SoF is calculated. The elastic value of the modulus affects the only deformation on the model before the plastic condition is achieved [5]. So the parameters for the simulation with Plaxis are as follows as show on table 2:
Table 2: Soil Parameters for Stability Simulation

| Parameters                | Model | Sub Grade | Soil Embankment | Units       |
|---------------------------|-------|-----------|-----------------|-------------|
| Type of Behavior          |       |           |                 |             |
| Soil Weight Unsaturated   |       |           |                 |             |
| Soil Weight Saturated     |       |           |                 |             |
| Young's Modulus           |       |           |                 |             |
| Poisson Ratio             |       |           |                 |             |
| Cohesion                  |       |           |                 |             |
| Friction Angle            |       |           |                 |             |
| Dilatancy Angle           |       |           |                 |             |
|                           |       |           |                 |             |
|                           | Model | MC        | MC              | -           |
|                           | Type  | Undrained | Undrained       | -           |
|                           | Yunsat| 16.00     | 17.00           | kN/m³      |
|                           | Ysat  | 18.00     | 19.00           | kN/m³      |
|                           | E_ref | 90.00     | 10.00           | kN/m²      |
|                           | ν     | 0.35      | 0.35            |             |
|                           | C ref | 60.00     | 90.00           | kN/m²      |
|                           | φ     | 1.00      | 10.00           | °           |
|                           | ψ     | 0.00      | 0.00            | °           |

2.5.2 Alternative Handling With Sheetpile FCSP

Alternative handling to reduce space of the embankment foot with using sheet pile concrete construction [6]. By using sheet pile concrete, then the slope of the embankment (V: H is 1:2) can be eliminated. The proposed FEM model as shown on figure 5:

Figure 5: FEM model for alternative solution Sheetpile at relocation of Kali Beringin (Model G1)

Following is the model G1 modeling results with FCSP reinforcement as shown on figure 6:

Figure 6. Incremental Strain on Model G1 Sheet pile.

And iteration curve of safe number on Model G1 Sheet pile is SoF = 1.781 as show on figure 7:

Figure 7: Iteration Curve of Safe Number on Model G1 Sheet pile (SoF = 1.781)

So the safe number (SoF) required for the area outside approach bridge is 1.40 for conditions without an earthquake. And the simulation results the safe number (SoF) of the G1 model is 1.781, then
still fulfilling minimum criteria. While, when reviewed with the earthquake load, then the simulation results can be seen as figure 8:

![Figure 8: Incremental Strain on Model G1 Sheet pile Earthquake Load Mode](image)

And iteration curve of the safe number on Model G1 Sheet pile Earthquake Mode is FoS = 1.356 as shown on figure 9:

![Figure 9: Iteration Curve of Safe Number on Model G1 Sheet pile Earthquake Mode (FoS = 1.356)](image)

So the minimum safe number for earthquake load SoF = 1.356 > 1.10 has been fulfilled. The magnitude of the maximum moment occurring can be as follows figure 10:

![Figure 10: Extreme Bending Moment on FCSP Model G1](image)

The program output produces a maximum moment value of 130.21 kN.m/m. The crack moment capacity of FCSP B 320.500 material is 66.5 kN.m/0.5m. When equivalent in the form of the moment of each meter length then the moment of resistance is 133 kN.m/m. Thus capacity resistance the moment of FCSP able to resist maximum moments, maximum moment value for 130.21 kN.m/m < moment of resistance is 133 kN.m/m (safe). However, it is recommended to use minimal FCSP C 320.500 or greater for security reasons. The crack moment capacity of FCSP C 320.500 material is 72.4 kN.m/0.5m equal to 144.8 kN.m/m.
3 Conclusions

The requirement of the safety factor of slope stability at embankment is relatively big, that is (SoF) \( \geq 1.70 \) for road approach bridge location and (SoF) \( \geq 1.40 \) for nonroad approach bridge location. To technical review the above, especially analysis of the embankment stability required a safety factor (SoF) to the collapse that occurred, the value of SoF (safety factor) should be taken minimal 1.50. The safe number required for the area outside the embankment on road approach bridge is 1.40 for conditions without an earthquake. Safe Number on Model G1 Sheet pile Earthquake Mode is 1.356. So the minimum safe number for earthquake load SoF = 1.356 > 1.10 has been fulfilled. The program output produces a maximum moment value of 130.21 kN.m/m. The crack moment capacity of FCSP B 320.500 material is 66.5 kN.m/0.5m. When equivalent in the form of the moment of each meter length then the moment of resistance is 133 kN.m/m. Thus capacity resistance the moment of FCSP able to resist maximum moments, maximum moment value for 130.21 kN.m/m < moment of resistance is 133 kN.m/m (safe). However, it is recommended to use minimal FCSP C 320.500 or greater for security reasons. The crack moment capacity of FCSP C 320.500 material is 72.4 kN.m/0.5m equal to 144.8 kN.m/m.

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