Forensic Analysis of Soil Improvement Methods in River Delta Area for Railway Mode

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ABSTRACT
In the context of the construction of a railway line, it is necessary to design an efficient and safe embankment construction. Therefore, the construction design needs to be studied using relevant data to obtain an embankment design that meets the needs so that the implementation process is not disrupted. This paper will present the results of the evaluation related to the condition of the existing embankment as well as alternative methods of soil improvement and embankment reinforcement, slope stability, settlement analysis and soil bearing capacity. Based on drilling data (SPT), soft soil is located up to a depth of 40 m and hard soil up to 60 m has not been found, plus additional information that the area is a river that has been filled and settlement has occurred about 32 cm. Analysis result of the existing state will continue to decrease to 2.7 m and add nonsolid embankment material. The first alternative is to dismantle the existing embankment, then the base of the embankment is repaired with geotextiles and embankment reconstruction according to standards, considering the addition of periodic ballast because it is still down by 1.78 m. The second alternative is the box culvert structure, considering the smaller load from the embankment. The settlement of consolidation that occurs is quite small at 0.126 m

***Keywords**: soil improvement, settlement of consolidation, slope stability, river delta,*

1. INTRODUCTION

The construction of the railway line in Makassar is the first stage of the construction of the Trans Sulawesi National Railway Network which is also listed in the National Strategic Project, whose construction began in South Sulawesi Province.. South Sulawesi Province is a province that has the largest estimated pattern of passenger and freight travel from other provinces on Sulawesi Island. The construction of the 142 km railway line connecting Makassar - Parepare has started since 2015 and is still ongoing. It is hoped that in 2024 all the development processes of the Makassar - Parepare railway infrastructure can be completed and ready to operate.

In the framework of the construction of the railway line, it is necessary to design an efficient and safe embankment construction. Therefore, the construction design needs to be studied using relevant data to obtain an embankment design that meets the needs so that the implementation process is not disrupted.

Railroads are planned in locations with varying soil conditions. Some of the results of field tests that have been carried out show that the soil conditions are soft clay. For the settlement that occurs still meets the specified requirements, it is necessary to have adequate reinforcement or construction methods.

In this paper, the evaluation results related to alternative methods of soil improvement and embankment reinforcement, embankment slope stability, settlement analysis and soil bearing capacity will be presented.

2. ANALYSIS METHODS

The analysis carried out is the safe number of slope stability at the required initial embankment height. The soil bearing capacity analysis is carried out to review the type of soil improvement required. The settlement that occurs is obtained by analysis using empirical methods and finite element-based software to determine the magnitude of the decrease that occurs.

A. TRAIN LOAD DISTRIBUTION

Railroad wheels provide both vertical and horizontal forces on the rails. According to Profilidis (2006), the schematic of the distribution of forces from the wheels on the subgrade is presented in Figure 1.

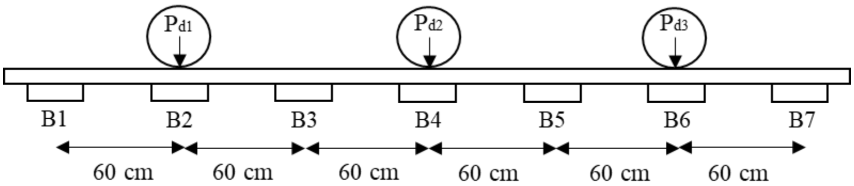

Figure 1. Schematic of loading train wheels against rails and bearings

Referring to SNI Geotechnical, subgrade serves to support the load transmitted by ballast to the subgrade, transmit the load to the layer below it, and provide a flat foundation at the position where the ballast will be placed. Here are some of the equations used to calculate the pressure under the ballast.

According to Clarke (1957) in Rosyidi [1], the pressure on the subgrade can be determined by the following equation.

$$\sigma_z = 2P_a \frac{BL}{(B+2z)(L+2z)} \quad \text{.......................................(1)}$$

with,
- σ_z = vertical pressure at depth z (kPa),
- P_a = average contact pressure between bearing and ballast (kPa),
- z = ballast thickness (m)
- B = sleeper width (kPa)
- L = sleeper length (m)

According to Schramm (1961) in Rosyidi [1], the pressure on the subgrade can be determined by the following equation.

$$\sigma_z = P_r \frac{1{,}5(L-g)B}{\{3(L-g)+B\}z\tan\theta} \quad \text{.......................................(2)}$$

with,
- σ_z = vertical pressure at depth z (kPa),
- P_r = average contact pressure between sleeper and ballast (kPa),
- z = ballast thickness (m)
- B = sleeper width (kPa)
- L = sleeper length (m)
- g = distance between sleepers (m)
- θ = ballast internal friction angle (^{o})

Rosyidi [1] used the Beam on Elastic Foundation (BoEF) and JNR analysis methods to calculate the pressure acting on the ballast in the following equation.

$$\sigma_2 = \frac{58\sigma_1}{10+d^{1.35}} \quad \text{.......................................(3)}$$

with,
- σ_2 = pressure acting on the subgrade (kg/cm^2),
- σ_1 = pressure under sleeper (kg/cm^2),
- z = ballast thickness (cm)

In addition, Rosyidi [1] takes into account the pressure acting on the subgrade using the AREA and Talbot analysis methods in the following equation.

$$\sigma_2 = \frac{58\sigma_1}{10+d^{1.35}} \quad \text{.......................................(4)}$$

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with, σ_2 = pressure acting on the subgrade (kg/cm^2),
σ_1 = pressure under sleeper (kg/cm^2),
d = ballast thickness (cm)

B. SOIL BEARING CAPACITY

The equation used is related to the soil properties and the shape of the shear plane when failure occurs. The bearing capacity failure analysis assumes that the soil is plastic. The equation used to calculate the bearing capacity of the soil improvement structure by considering it as a longitudinal strip foundation or plane strain is as follows [2]

$$q_{ult} = c.N_c + p_o.N_q + 0{,}5.\gamma_{soil}.B.N_\gamma \quad \text{(5)}$$

with, c = cohesion (kPa)
γ = soil volume weight (kN/m^3)
B = repair structure width (m)
P_o = overburden pressure (kPa)
N_c = taken 5.7 for the case of pure clay

For clay analysis, depth review is not considered, so that only the first term is used based on the above formula. In pure sandy soil, using the cone resistance value approach from the field sondir data, the bearing capacity value is approximated by the value of qc / 40.

C. STABILITY OF EMBANKMENT SLOPE

In general, there are two (2) factors that cause slope instability, namely internal and external factors. Internal factors are factors that originate from the body of the embankment slope such as slope-forming soil material, groundwater level, slope slope and cracks on the slope. While external factors are factors that come from outside such as rainwater infiltration, transportation loads, the presence of vegetation, slope creep, and earthquakes. For the calculation of embankment slope stability, using empirical calculations and the help of Geo Slope software.

The general form for calculating slope stability is to find the number of safety factor (SF) by comparing the moments that occur due to the acting forces [3]

$$SF = \frac{R.c.L_{AC}}{W.y} \quad \text{(6)}$$

with: SF = safety factor
W = The weight of the soil that will slide (kN)
L_{AC} = arch length (m)
c = cohesion (kN/m^2)
R = the radius of the circle of the landslide area under consideration (m)
y = Distance of center of gravity W to center of radius (m)

D. SETTLEMENT

The analysis of the decline in the construction of the rail road was carried out using empirical and numerical methods/software based on the finite element method. The simulation results are presented in the form of an analytical and numerical decline analysis output. Immediate settlement analysis on sandy soils [4]

$$S_i = \frac{H}{C} ln \frac{p_0' + \Delta p}{p_0'} \quad \text{(7)}$$

$$C = \frac{1{,}5 q_c}{p_0{}'} \quad \text{(8)}$$

$$I_z = \frac{1}{\pi}\left[\left\{\frac{a+b}{a}\right\}(\alpha_1 + \alpha_2) - \frac{b}{a}\alpha_2\right]$$

……………………………………………..(9)

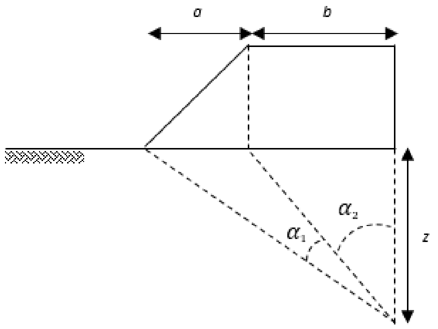

Figure 2. Illustration of the settlement influence factor

with,
- S_i = settlement immediately as thick as layer H
- H = the thickness of the settlement layer under review
- C = compression index
- q_c = static cone resistance
- p_0' = effective overburden pressure
- I_z = strain influence factor
- Δp = additional vertical stress in the center of the layer by additional embankment stress

Analisis penurunan segera pada tanah berlempung [5]

$$S_i = \mu_1 \mu_0 \frac{q_n B}{E} \quad \text{……………………………………………………(10)}$$

with,
- S_i = immediate settlement
- μ_1 = correction factor for excavation depth
- μ_2 = correction factor for limited thick soil layers
- B = uniform load width
- q_n = pressure due to net load

Settlement for normally consolidated clay ($p_c' = p_o'$) [6]

$$S_c = C_c \frac{H}{1+e_0} log \frac{p_1'}{p_o'} \quad \text{……………………………………………(11)}$$

For overconsolidated clay ($p_c' > p_o'$) The total primary consolidation settlement is expressed by an equation that depends on the value of p1',

Jika, p1' < pc'

$$S_c = C_r \frac{H}{1+e_0} log \frac{p_1'}{p_o'} \quad \text{…………………………………………(12)}$$

Jika, p1' > pc'

$$S_c = C_r \frac{H}{1+e_0} log \frac{p_c'}{p_o'} + C_c \frac{H}{1+e_0} log \frac{p_1'}{p_c'} \quad \text{………………………………(13)}$$

with,
- C_r = return compression index
- C_c = compression index
- H = soil layer thickness
- p_c' = preconsolidation pressure
- e_0 = initial pore number

Δp = additional stress due to foundation load
p_o' = effective overburden pressure first before loading
p_1' = $po' + \Delta p$

Casagrande [7] and Taylor [3] proposed the following equation for the relationship between U and Tv

- For U < 60% ; Tv = (0,25 p) U^2
- For U > 60% ; Tv = -0,933 log (1 – U) – 0,085

with, U = degree of consolidation
T_v = time factor

To find out the amount of time for consolidation, use the following equation.

$$t = \frac{{H_t}^2\, T_v}{C_v} \quad \text{...(14)}$$

with, H_t^2 = the thickness of the fisted soil layer
T_v = time factor
t = consolidation decline time

3. RESULT AND DISCUSSION

A. GEOTECHNICAL INTERPRETATION DATA

Geotechnical interpretation data is data that contains information about the index and engineering properties of the soil layer at the review site. This data is used as a reference in making finite element models as well as empirical calculations at the analysis stage. In this report, the soil layers at the site are presented in the soil stratigraphy in Figure 3 and Figure 4.

Figure 3. Land investigation site plan

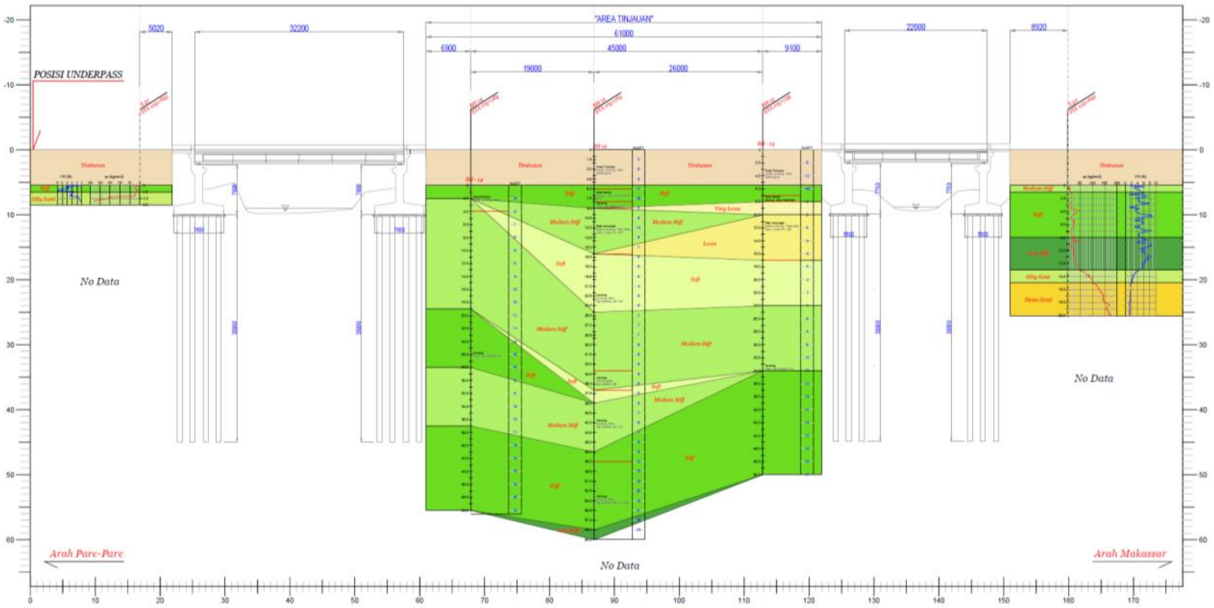

Figure 4. Soil stratigraphy

B. LOAD DATA

1) Train Load

Based on the data obtained, the axle load of the locomotive to be used has a load of 22.5 tons, the design speed is 120 km/hour with the width of the rail used is 1435 mm. The uniformly distributed load is generated based on the locomotive axle load and is described under ballast.

The dynamic factor value or dynamic load index is obtained from empirical experiments and vehicle or train speed parameters. The dynamic load index value is also determined from the quality of the instruments and components of the railroad used and the assumptions used in the design of the railroad structure [1]

Dynamic load factor

$$I_D = 1 + 5,21\frac{V_r}{D} = 1,820$$

Table 1. Distribution of axial load on ballast

	Axial load on ballast						
	B1	**B2**	**B3**	**B4**	**B5**	**B6**	**B7**
Consequences of P_{d1}	23%	40%	23%	7%	0%	0%	0%
Consequences of P_{d2}	0%	7%	23%	40%	23%	7%	0%
Consequences of P_{d3}	0%	0%	0%	7%	23%	40%	23%
Total	23%	47%	46%	54%	46%	47%	23%
Total (kN)	50.77	103.74	101.53	119.19	101.53	103.74	50.77

For the next load calculation, the maximum load is used, namely the load that is on the ballast just below the wheel.

Average contact pressure between sleeper and ballast P_a = 269.48 kPa

Based on the analysis using the Clarke, Schramm equation, the combination of BoEF and JNR, as well as the combination of AREA and Talbot, the load is obtained as shown in Table 2.

Table 2. Pressure value on subgrade

No	Method	Nilai Tekanan pada Tanah Dasar (kPa)
1	Schramm, 1961	63.35
2	AREA dan Talbot	63.59
3	BoEF dan JNR	43.37
4	Clarke, 1957	57.94
5	Boussinesq, 1885	42.97
	Average	54.24

2) Embankment Load

The body of the railroad is a layer of soil, either in its original condition or in the form of repair or in an artificial form that carries the load carried out by the top and bottom ballast layers. The body of the road on the embankment consists of subgrade in the form of embankment soil

Embankment volume weight γ_{embk} = 18 kN/m^3
Existing embankment height h_{embk} = 5.5 m
Earth pressure due to embankment $p_{o\ embk}$ = 99 kPa

C. REVIEW OF EXISTING CONDITION

Based on the results of the calculation of the carrying capacity of the soil in the existing embankment condition with a height of 5.5 m, the value of the bearing capacity of the soil with a safe value value above 3.00 for clay soil is at a depth of 42 m with a value of SF = 2.25. These results can mean that the existing embankment will continue to fall, considering the soft soil data up to 50 m and has not yet met hard soil. Plus, field information that formerly the area was an embankment river.

Information received from the embankment construction site has been built for about 2 years. Based on the measurement of the existing top rail periodically in the last 7 months, the decrease in changes is about 1 – 2 cm which tends to decrease over time. From the design of the top rail plan with the current top rail measurement it has dropped about 32 cm.

Analysis of the stability of the embankment slopes in the existing condition with an embankment height of 5.5 m still does not indicate the occurrence of damage, when viewed visually. The following provides an analysis of the safety figures on the slope of the existing embankment in Figure 5.

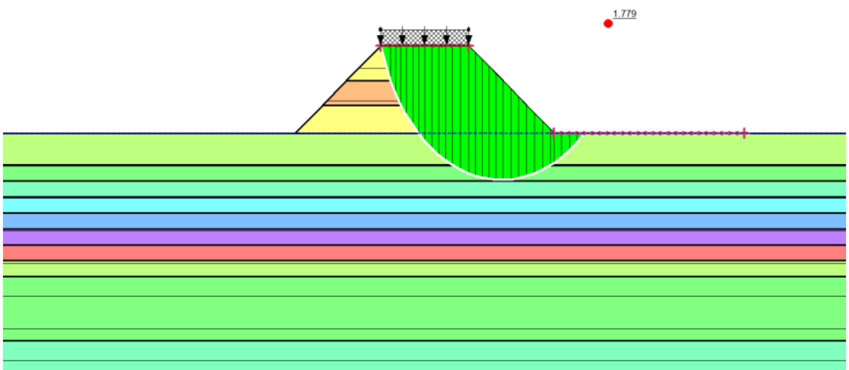

Figure 5. Geo Slope analysis of embankment stability is 5.5 m with SF_{STATIC} = 1.779 for the existing condition and $SF_{DYNAMIC}$ = 1.324 for the existing condition

In terms of land subsidence due to the dominant clay soil, then in terms of consolidation settlement, it is presented in Figure 6.

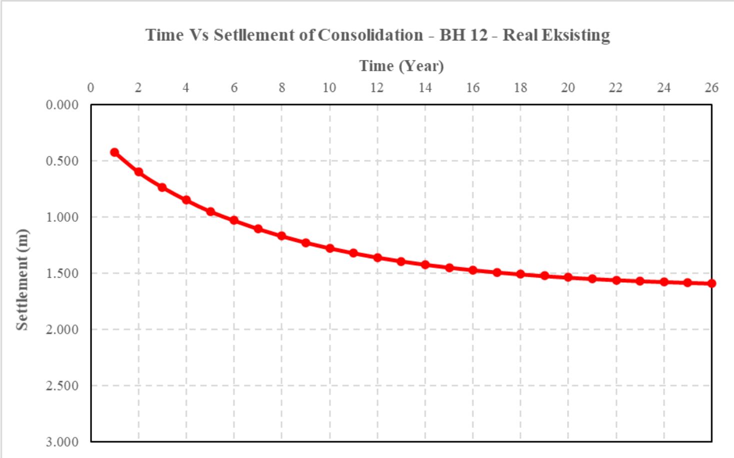

Figure 6. Graph of settlement against time analytical method with embankment height of 5.5 m existing condition on original soil

With the help of finite element-based analysis using Plaxis software, the settlement that occurs in the top embankment with the existing embankment soil condition is quite soft (N-SPT value between 3-5) is 2.7 m. This means that the consolidation settlement that occurs in the embankment alone is 2.7 m – 1.95 m = ~ 0.75 m. The following is a shading display for the consolidation of the existing condition, which is presented in Figure 7.

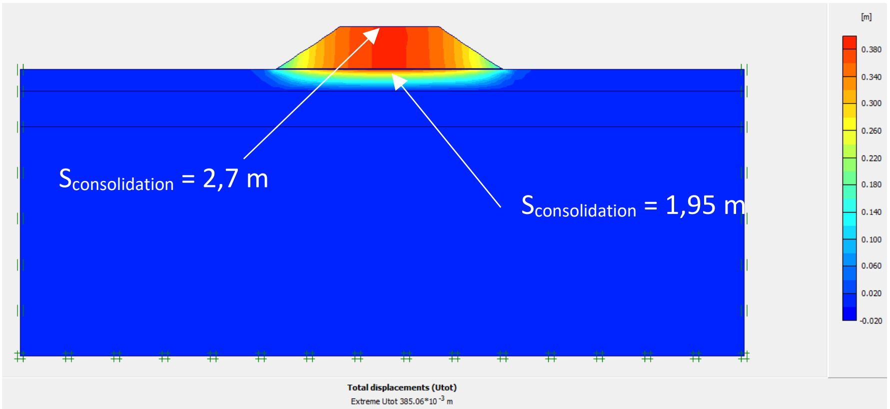

Figure 7. Shading consolidation settlement height of the embankment is 5.5 m in the existing condition of 2.7 m on the top of the embankment and 1.95 m on the original soil.

D. ALTERNATIVE DESAIN IMPROVEMENT

Embankment Reconstruction and Reinforcement

After seeing a high settlement of the embankment body and the poor soil condition of the embankment, then it was redesigned by dismantling the existing embankment, then replacing it with a fairly good standard embankment reinforced with a double geogrid and replacing it with a depth of 1 m / to meet bulls of rock before. Below is a graph of the consolidation decline and FEM output for the alternative by reconstructing the heap in Figure 8 and Figure 9.

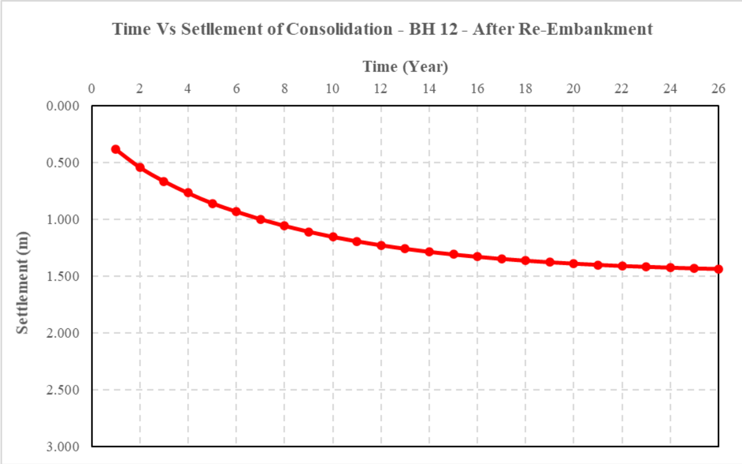

Figure 8. Graph of consolidation settlement against time of 5.5 m embankment analytical method after dismantling the embankment with 1 m replace repair and duplicate geogrid

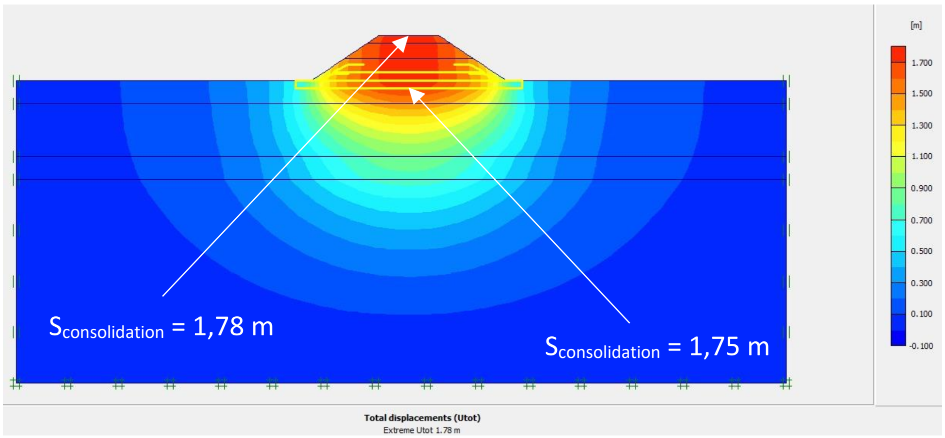

Figure 9. Shading for consolidation of embankment settlement is 5.5 m after dismantling the embankment with 1 m replacement repair and a duplicate geogrid of 1.78 m on the top of the embankment and 1.75 m on the original soil.

Replacing the Existing Soil Fill with a Box Culvert Structure

By considering several alternative treatments in this “river delta” area by considering the soft soil deep enough, it is studied and planned with several alternatives, including bridge structures or box culvert arrangements. Conditions to consider when using an alternative bridge structure, the foundation must be designed very deep, seeing that hard soil has not been found to a depth of 60 m. In addition, there is also a concern that the bridge load is quite large, it is feared that it will decrease again. Therefore, the alternative treatment with bridges seems less than ideal in the "river delta" area.

Another alternative treatment carried out in this "river delta" area needs to be designed with a lighter load than the soil pile in order to minimize or reduce the subsidence that occurs. One alternative is the arrangement of box culverts along the river delta area (L=~61 m). After calculating the general load can be reduced 33,429% of the load on the soil embankment. For more details, see Table 3.

Table 3. Dead load estimation using box culvert arrangement

No	Part of Structure	Length (m)	Width (m)	Hight (m)	$\gamma_{concrete}$ (kN/m^3)	$\gamma_{ballast}$ (kN/m^3)	n (bh)	Load (kN)	Footing Area (m^2)	Uniform Load (kPa)
1	Top Slab	61	6.2	0.6	24	-	-	5446.1	P = 61 m	5.511
2	Bottom Slab	61	16.2	0.7	24	-	-	16601.8	L = 16.2 m	16.800
4	Vertical Wall	0.5	6.2	4.7	24	-	18.0	6294.2		6.369
5	Railing Wall	61	0.2	0.9	24	-	2.0	527.0		0.533
6	Wing Wall	5	0.5	4.7	24	-	36.0	5076.0	988.2	5.137
7	Signal Box	61	0.2	0.6	24	-	2.0	351.4		0.356
8	Ballast	61	4	0.65		19	-	3013.4		3.049
	Total							**37309.9**		**37.755**

With the hope of reducing the consolidation decline that occurs, it is planned to use an alternative treatment with a box culvert structure. For the analysis scheme carried out in the form of modeling the existing embankment, after that it is seen how much consolidation decreases after 2 years of construction, then the process of unloading the embankment (unloading the embankment) and loading the box culvert (DL and LL). The following is the calculation of the consolidation of the analytical method and FEM presented in Figures 10 to 12.

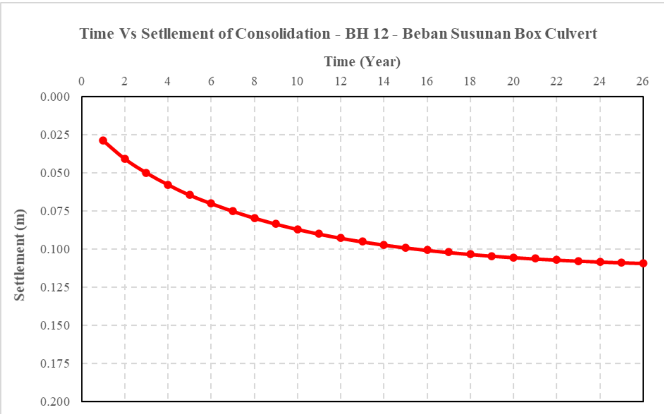

Figure 10. Graph of decrease against time analytical method for alternative conditions of handling with box culvert arrangement

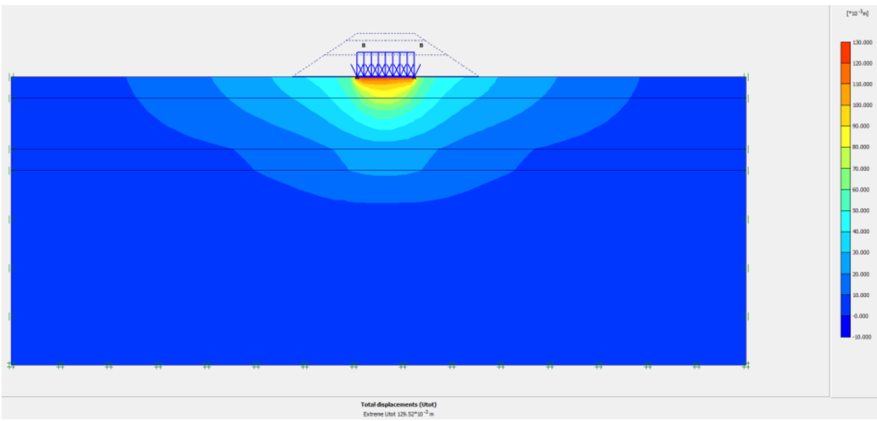

Figure 11. Shading of consolidation settlement for alternative handling conditions with a box culvert arrangement of 0.126 m.

Figure 12. Perspective illustration of box culvert arrangement

4. CONCLUSION

Information received from the embankment construction site has been around for about 2 years. Based on the measurement of the existing top rail periodically in March, June and September 2021, the decrease in changes is around 1 – 2 cm which tends to decrease over time. From the design of the top rail plan with the current top rail measurement it has dropped about 32 cm. Observation of the embankment body still shows intact condition / no damage has occurred.

Based on the analysis of the existing embankment, there is a potential for a decrease of 2.7 m on the top of the embankment and 1.95 m on the original soil. It can be interpreted that the consolidation decrease that occurs in the embankment body alone is ~ 0.75 m. This is also reinforced by the N-SPT value starting from the top of the embankment showing a low value at the embankment body.

Alternative The first repair is by dismantling the existing embankment, then the base of the embankment is repaired with geotextiles and embankment reconstruction according to standards, considering the addition of periodic ballast because it is still down by 1.78 m. The second alternative is a box culvert structure, considering the smaller load from the embankment. The decrease that occurs is quite small at 0.126 m.

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