Comparative Study of Conventional Cantilever Wall and Mechanically Stabilized Earth Wall for Slope Failure Remediation

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Abstract

Slope failure is caused by many factors, mainly high rainfall, especially in a tropical country like Indonesia. Many solutions have been expanded to do remediation of slope failure. However, establishing the best solution method is difficult because of many geotechnical challenges and different situations at the site. In addition, fast rate construction in line with a good quality of stability aspect is necessary to be attained. Therefore, a comparative study of two common solution methods, namely conventional cantilever wall and mechanically stabilized earth wall, was conducted to determine the best solution for slope failure remediation in the case studied. The analysis was performed by using a computer software program coded Geo5. A slope failure case in Bali was selected to investigate the performance of both studied methods in meeting all engineering criteria, including bearing capacity, internal stability (overturning and sliding), and global stability. The results show that mechanically stabilized earth wall gives more advantage in meeting all criteria with SF global stability of 1.40. On the other hand, conventional cantilever walls show a deficiency in SF global stability of 1.17 that will require pile foundation support or another reinforcement under the base plate, which will automatically increase construction time.

Keywords

Slope failure remediation, cantilever wall, mechanically stabilized earth wall, geo5

INTRODUCTION

The range of slope stabilization works may be categorized as surface protection and drainage, subsurface drainage, slope regarding, retaining structures, structural reinforcement, strengthening of slope-forming material, vegetation or bioengineering, removal of hazards, and special materials [1]. The causal factors triggering landslide are very diverse, including ground conditions (plastic weak material, sensitive material, collapsible material, weathered material, sheared material, jointed or fissured material, adversely oriented mass discontinuities, adversely oriented structural discontinuities, contrast in permeability and its effects on ground water contrast in stiffness), geomorphological processes (tectonic uplift, volcanic uplift, glacial rebound, fluvial erosion of the slope toe, wave erosion of the slope toe, glacial erosion of the slope toe, erosion of the lateral margins, subterranean erosion, deposition loading of the slope or its crest, and vegetation removal), physical processes (intensive short period rainfall, rapid melt of deep snow, prolonged high precipitation, rapid drawdown following floods, earthquake, volcanic eruption, breaching of crater lakes, thawing of permafrost, freeze and thaw weathering, and shrink and swell weathering of expansive soils), and man-made processes (excavation, drawdown of reservoirs, irrigation, vegetation removal, mining, and artificial vibration including traffic, pile driving, or heavy machinery) [2]. The intense high rainfall is judged as the most frequent causal factor of slope failure in Indonesia, a tropical country.

The remedial works of slope failure could be categorized in four groups, namely slope geometry, drainage, retaining structure, and internal slope reinforcement [2]. The slope geometry category is performed by adding counterweight berm or fill material to stability maintain area, removing material from landslide driving area and change with light weight fill, or reducing the slope angle [2]. The drainage category including surface drains for diversion of water, shallow or deep trench drains having filled with free draining geo-materials, vertical boreholes for self-draining, vertical wells for gravity draining, buttress counterforts of coarse-grained materials, sub vertical and sub horizontal boreholes, drainage tunnels, galleries drainage by siphoning, and vacuum dewatering [2]. The retaining structure category including gravity walls, gabion walls, passive piles, cast in situ reinforced concrete walls, reinforced earth retaining structures, buttress counterforts of coarse-grained materials, and rock fall attenuation or stopping systems [2]. While the internal slope reinforcement including rock bolts, anchors, micro piles, and soil nailing [2].

RESEARCH SIGNIFICANCE

Among those diverse slope remediation method, this research will evaluate and compare two types of retaining structures, namely conventional cantilever wall (part cast in situ reinforced concrete walls) and mechanically stabilized earth wall (part of reinforced earth retaining structures). The analysis was performed by using a computer software program coded Geo5. A slope failure case in Bali was selected to investigate the performance of both studied methods in meeting all engineering criteria, including bearing capacity, internal stability (overturning and sliding), and global stability.

METHODOLOGY

A. CANTILEVER WALL DESIGN METHOD

A cantilever wall is a retaining structure which is made from reinforced concrete, that why the dimensions of stem and base slab becomes relatively thin. This type of structure not only rely on self-weight to maintain the stability against sliding and overturning, but also rely on the total soil weight above its base slab. Cantilever wall is preferable to retain the soil until 8 m height [3]. Initial estimation of the cantilever wall dimensioning is shown in Figure 1. The designer can use this dimension guideline to determine the dimension of every cantilever wall part and adjust during the calculation process if needed. The aspects that should be considered in cantilever wall design are bearing capacity minimum 3.0, stability against overturning and sliding minimum 1.5, and global stability (slope stability) minimum 1.5 according to SNI 8460:2017.

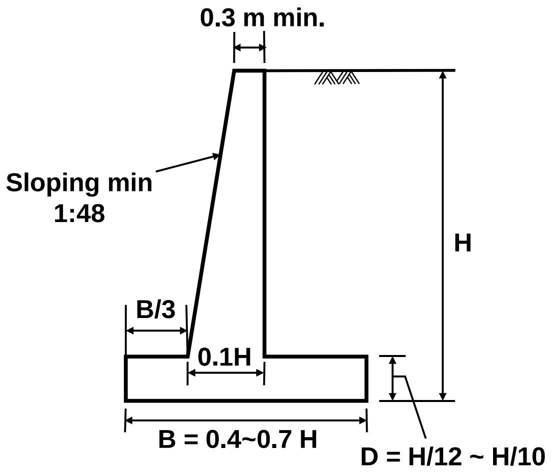


Figure 1 Dimension determination guidelines of cantilever wall [3]

Bearing capacity of cantilever wall is based on shallow foundation approach with the calculation method follow J. Brinch – Hansen theory (Eq. 1) [4]. The detailed calculation method for each aspect of the Eq. 1 is described in Table 1.

$$R_d = cN_c s_c d_c i_c b_c g_c + q_0 N_d s_d d_d i_d b_d g_d + \tfrac{b}{2}\gamma N_b s_b d_b i_b b_b g_b \quad (1)$$

Table 1 Detail equation for shallow foundation as J. Brinch – Hansen Theory [4]

Equation	No.
Bearing capacity factors	
$q_0 = \gamma_1 d$	(2)
$N_c = (N_d - 1)\cot g\varphi \rightarrow for: \varphi > 0$	(3)
$N_c = 2 + \pi \rightarrow for: \varphi = 0$	(4)
$N_d = \tan^2\left(45 + \frac{\varphi}{2}\right)e^{\pi \tan\varphi}$	(5)
$N_b = 1.5(N_d - 1)\tan\varphi$	(6)
Shape factors	
$s_c = 1 + 0.2\frac{b}{l}$	(7)
$s_d = 1 + \frac{b}{l}\sin\varphi$	(8)
$s_b = 1 - 0.3\frac{b}{l}$	(9)
Depth factors	
$d_c = 1 + 0.1\sqrt{\frac{d}{b}}$	(10)
$d_d = 1 + 0.1\sqrt{\frac{d}{b}\sin 2\varphi}$	(11)
$d_b = 1$	(12)
Inclination factors	
$i_c = i_d = i_b = (1 - \tan\delta)^2$	(13)
Base factors	
$b_c = b_d - \frac{1 - b_d}{N_c \tan\varphi}$	(14)
$b_d = (1 - \alpha\tan\varphi)^2$	(15)
$b_b = b_d$	(16)
$g_c = 1 - \frac{2\beta}{\pi + 2}$	(17)
$g_d = g_b = (1 - 0.5\tan\beta)^2$	(18)

Where c is cohesion, q_0 is equivalent uniform load accounting for the influence of foundation depth, d is depth of the footing bottom, γ_1 is unit weight of soil above the footing bottom, b is width of foundation, γ is unit weight of soil, (N_e, N_d, N_b) is coefficient of bearing capacity, (s_e, s_d, s_b) is coefficients of shape of the foundation, (d_e, d_d, d_b) is coefficients of influence of foundation depth, (i_e, i_d, i_b) is coefficients of influence of slope of the load, (b_e, b_d, b_b) is coefficients of influence of base slope, (g_e, g_d, g_b) is coefficients of influence of slope of the terrain, φ is angle of internal friction of soil, l is length of foundation, δ is angle of deviation of the resultant force from the vertical direction, β is slope of terrain, and α is slope of footing bot. Please see Figure 2 for the detailed position about β and α.

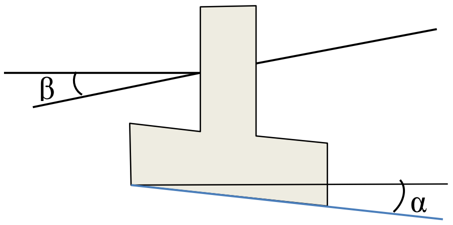


Figure 2 Detail position of notation angle β and α (after [4])

The internal stability of the conventional cantilever wall was checked by calculating overturning dan sliding stability by following Eq. 19 and Eq. 20, respectively.

$$\frac{M_{res}}{M_{ovr}} > SF_o \quad (19)$$

Where M_{ovr} is overturning moment, M_{res} is resisting moment, and SF_o is safety factor for overturning.

$$\frac{H_{res}}{H_{act}} = \frac{\left[\left(N \tan\varphi + c\left(b-2e\right)\right)/\mu + F_{res}\right]}{H_{act}} > SF_s \quad (20)$$

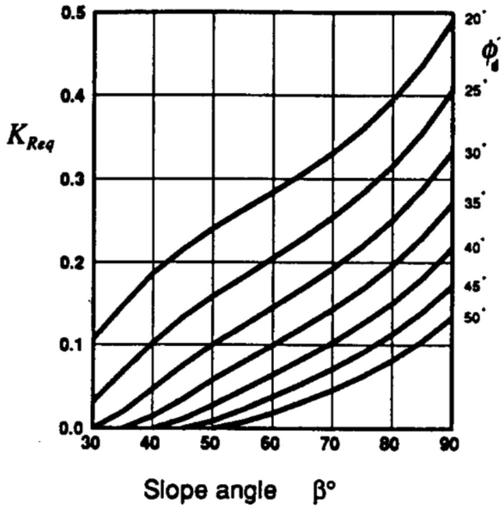
Where H_{res} is the resisting horizontal force, H_{act} is shearing force acting in the footing bottom, N is normal force acting in the footing bottom, F_{res} is resisting force (from georeinforcement and mesh overlap), SF_s is safety factor for the sliding resistance, μ is reduction coefficient of the contact base – soil, and e is eccentricity which follow the Eq. 21. The global stability is calculated by using limit equilibrium approach.

$$e = \frac{M_{ovr} - M_{res} + \frac{Nb}{2}}{N} \quad (21)$$

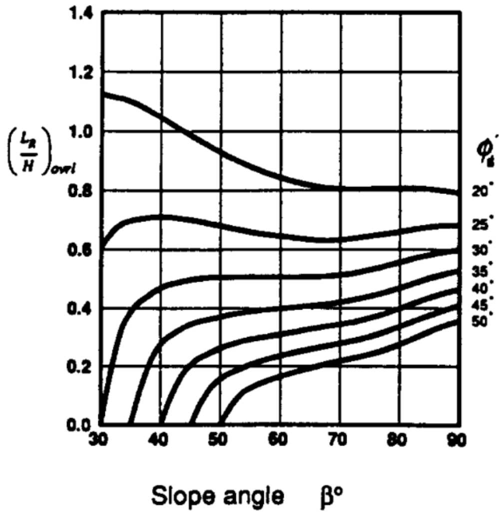
B. MECHANICALLY STABILIZED EARTH WALL DESIGN METHOD

MSE wall is a combination between facing (segmental precast concrete panels, steel plate, gabions, or geosynthetic), reinforcement (steel of geotextile reinforcement) and granular fill material which establishes an internally stable composite structure. The reinforcement is placed layer by layer inside the granular fill materials. MSE wall is an alternative to substitute conventional retaining wall (gravity and cantilever retaining wall) which is widely applied to road construction. Moreover, MSE wall also used to be applied to abutment, wing wall, and slope reinforcement with vary facing angle start from 3H:1V (3 horizontal: 1 vertical) until almost vertical (90 degrees). SNI 8460:2017 required the minimum factor of safety of MSE wall against lateral sliding is 1.5, overturning is 2.0, bearing capacity is 2.5, and global stability is 1.3. Among those varieties, this research will focus on MSE wall for slope reinforcement by using geotextile.

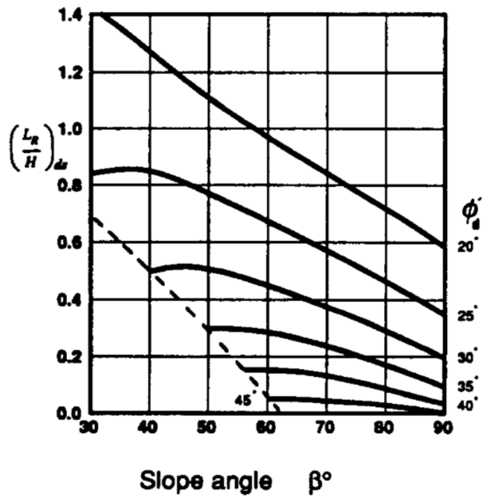
Jewell (1990) suggests the practical guidelines to design steep reinforced slopes by using various type of reinforcement materials including geotextile and polymer grids [5]. The mains parameter required is facing slope (β), internal friction angle of fill materials (φ'), and total height of the slope (H). Jewell (1990) provided three types of design charts depending on the pore water pressure coefficient (r_u) = 0, 0.25, and 0.50. The first chart provides the lateral earth pressure coefficient (K_{req}) from which the required reinforcement force for equilibrium is calculated, and the second chart give the minimum required reinforcement length L_R/H depending on which one govern between overall stability and direct sliding failure mechanism (see Figure 3). This research will use granular fill material, so the pore water pressure coefficient is assumed to be zero.



(a)



(b)



(c)

Figure 1. Chart for r_u = 0 with (a) Minimum required force, (b) minimum required length overall stability, and (c) minimum required length direct sliding [5]

Figure 4 Situation of the slope failure in Luwus, Bali

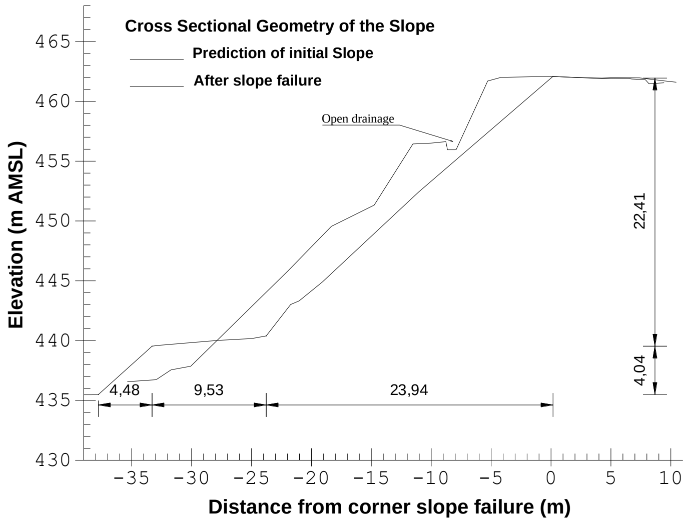


Figure 5 Typical cross section after slope failure

The first step is plotting the parameter into Figure 1 to determine K_{req} and L_R/H. A linear interpolation between the charts is sufficient. The second step is determining the tensile strength required and the value should be smaller than the long-term tensile strength of geotextile materials used ($R_{req} < Rt$).

$$R_{req} = K_{eq}\gamma\left(z + \frac{q}{\gamma}\right) \tag{22}$$

Where γ is the unit weight of fill materials, z is the depth below the slope crest, and q is the external uniform load. While the long-term tensile design strength of geo-reinforcement Rt is calculated from the Eq. 23.

$$R_t = \frac{T_{ult}}{RF_{CR} RF_D RF_{ID} FS_{UNC}} \tag{23}$$

Where R_t is long-term parameter for strength of reinforcement design, T_{ult} is ultimate tensile strength of geo-reinforcement, RF_{CR} is reduction coefficient of long-term deformation of reinforcement (determined based on geo-reinforcement lifetime), RF_D is reduction coefficient of the durability of reinforcement (determined based on soil pH), RF_{ID} is reduction coefficient of failure of reinforcement when inserting into the soil (determined based on the grain sizes of soil), and FS_{UNC} is overall coefficient of model uncertainty.

The resistance of reinforcement against pull-out from the earth body in Geo5 software is calculated from the geo-reinforcement and the normal force acting in the direction normal to its area.

$$T_p = 2LC_i\sigma_z \tan\varphi \tag{24}$$

Where T_p is bearing capacity against tearing. L is reinforcement length (from front face to its end), C_i is coefficient of interaction between soil and geo-reinforcement, and σ_z is vertical geostatic stress. The stability against lateral sliding, overturning, and global stability using same approach as used in conventional cantilever wall design.

C. THE CASE STUDY SITUATION

The location of road slope failure is in incline area, the left side in downhill slope while the right side is uphill slope (see Figure 4). There was gravity retaining structure on the left side with the effective height 5 m and open drainage in the bottom of it (Figure 5). The top level of the retaining structure is on the same elevation as the road. The vertical alignment of the road is around 3% with the Bedugul direction is higher. The slope failure investigation was conducted to figure out the cause. The rainfall in the last 3 days before the incident was high and the water flow exceeded the capacity of the drainage system, so it was impact on the erosion under the gravity retaining structure. The gradual erosion process triggered the instability of the gravity wall, so the failure unavoidable.

D. SOIL STRATIGRAPHY AND PARAMETERS

There is no soil data available for the case studied, so the soil data was adopted from the nearest project. There is slope failure case with similar slip surface which located 300 m apart from Luwus failure. The soil data resource has higher elevation, but it is assumed that both locations have identical soil stratigraphy. The soil layers, in general, can be divided into 4 layers with the relative zero elevation at the surface of the road pavement. The first layer consists of clayey silt soil with soft to firm consistency and thickness around 9 m. The second layer categorizes as sandy silt with stiff to very stiff consistency. This layer thickness is around 16 m from 9 m to 25 m depth. The third layer still has stiff to very stiff consistency, however, the soil type changes to sandy clay with the thickness around 10 m. The last layer is silty clay with stiff to very stiff consistency (see Figure 6).

Some undisturbed samples (UDS) were carried out during the drilling and tested in the soil laboratory, especially for the shear strength parameters including direct shear and triaxial test (see Table 2 for the detailed result). A geotechnical engineer needs to be careful while looking at the laboratory result. The understanding of soil mechanics, parameters correlation, and experience from previous project with the identical soil type is a must to customize the adequate parameters. Furthermore, the consistency of the soil parameters through three types of shear strength test needs to be examined.

Table 2 shows the shear strength parameters from direct shear test are the lowest compared to UU and CU triaxial test results. Meanwhile, the result from UU and CU shows a good consistency in both cohesion and internal friction angle parameters. Therefore, the combination of laboratory results and correlation are used to determine the design parameters for each soil layer as shown in Table 3.

RESULTS AND DISCUSSIONS

The analysis for slope failure remediation in Luwus Road is conducted in two approaches solutions, by conventional cantilever wall and mechanically stabilized earth wall (or reinforcement of slope using geosynthetic materials). Both solutions are performed using Geo5 software which adopts semi empirical solutions. The details of each analysis are presented in this chapter.

Table 2 Soil laboratory test results

Relative depth (*m*)	Unit Weight (KN/m^3)	Direct Shear		Triaxial UU[1]		Triaxial CU[2]	
		c (KN/m^2)	ϕ (°)	c (KN/m^2)	ϕ (°)	c (KN/m^2)	ϕ (°)
8.50	16.65	5	15.0	24.30	14.97	28.06	16.87
23.50	16.90	0	22.9	35.37	16.27	42.38	21.82
29.50	15.30	2	21.9	27.97	22.06	23.64	38.65
30.50	16.90	4.6	20.1	32.67	14.36	41.70	16.12
50.50	19.00	0	24.4	28.16	22.15	31.80	25.88
59.50	19.40	1.6	27.5	32.65	14.32	34.85	13.52

[1]Unconsolidated Undrained, [2]Consolidated Undrained

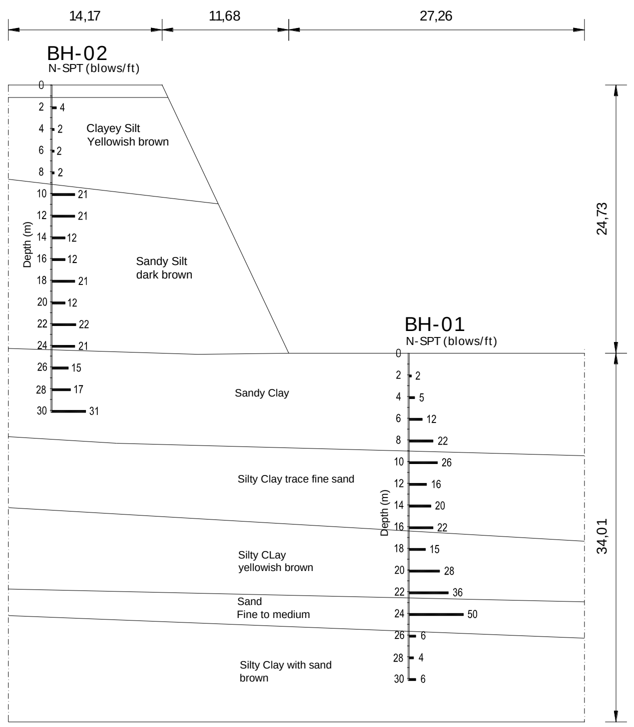


Figure 6 Performance measure BPKP's Intersection at Banda Aceh in existing condition

Table 3 Soil design parameters for analysis

End layer depth (*m*)	Soil Type	N design (blows/ft)	γ (KN/m^3)	γ_{sat} (KN/m^3)	*c'* (KN/m^2)	ϕ' (°)	δ (°)	*E* (MPa)	*v*
15	Clayey silt	15	16.5	18	15	28	18	7	0.33
24	Sandy silt	20	17	19	20	30	20	10	0.32
35	Sandy clay	23	17	19	35	32	21	15	0.32
50	Silty clay	30	19	20	20	35	23	20	0.31

A. CANTILEVER WALL ANALYSIS

The cantilever wall analysis was designed according to the dimensions guideline as shown in Figure 1. The effective height of the failure slope (Figure 5) is around 24 to 26 m. Overall height of the cantilever retaining structure, however, is not follow the height of the failure slope because the dimensions could be extra huge. Therefore, the height of the cantilever wall just accommodates 1/3 to 1/2 of overall height of the failure slope and in this research was chosen 10 m height, while the front face side is covered with compacted fill as shown in Figure 7a.

The internal stability of cantilever wall against overturning and sliding shows a satisfactory result, which means the SF value bigger than the minimum requirement. Overturning stability is the ratio between resisting moment and overturning moment (see Eq. 19). Resisting moment is summation of multiplication result between vertical force and point force application distance in horizontal axis, while overturning moment is summation of multiplication result between horizontal force and point force application distance in vertical axis. Table 4 shows the detail calculation of vertical dan horizontal force, while the point force application is shown in Figure 7a. The resisting moment (M_{res}) is 4883.17 kNm/m and the overturning moment (M_{ovr}) is 943.52 kNm/m, so the SF is 5.18 which means higher than minimum requirement 1.5. Sliding stability is calculated by following Eq. 20. The resisting horizontal force (H_{res}) is 606.31 kN/m and the active horizontal force (H_{act}) is 259.69 kN/m, so the SF is 2.33 which means higher than minimum requirement 1.5.

The bearing capacity check shows a satisfactory result too, unfortunately the global stability check is not satisfactory. The calculation of bearing capacity following the Eq. 1. The result of design bearing capacity (R_d) is 374.02 kN/m^2 and the maximum stress which transferred to soil (see Figure 7b) is 114.33 kN/m^2, so the SF for bearing capacity is 3.27 which means higher than minimum requirement 3.0. The factor of safety against the potential global slip failure is 1.17 which means lower than minimum requirement 1.5 (see Figure 8). The slip failure surface with circular approach is start in the back of traffic load and end in the slope toe. This surface of failure line indicates that even the bearing capacity of the cantilever wall is satisfactory, but additional reinforcement or anchored system like driven pile or bored pile is still required.

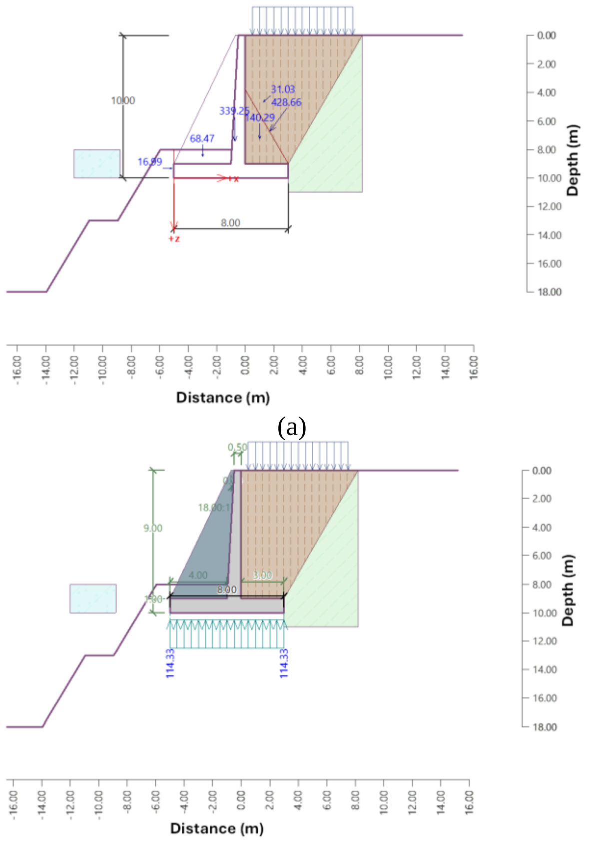


Figure 7 (a) Internal stability checking of conventional cantilever wall and (b) distribution of contact pressure under footing

Table 4 Detail of force acting on cantilever wall structure

Parameters	$F_{horizontal}$ (kN/m)*	Application Point hor. (m)	$F_{vertical}$ (kN/m)	Application point Ver. (m)	M_{ovr} (kNm/m)*	M_{res} (kNm/m)*
(a)	(b)	(c)	(d)	(e)	(b x c)	(d x e)
Weight - wall	0	-2.56	339.25	4.28	0	1451.99
Weight - soil	0	-1.50	68.47	2.01	0	137.62
Front Face resistance	-16.98	-0.67	0.49	-0.13	11.38	-0.06
Weight - earth wedge	0	-2.73	140.29	6.00	0	841.74
Active pressure	253.03	-3.28	346.01	6.72	-829.94	2325.19
Trafic load	23.64	-5.33	20.10	6.25	-126.00	125.62
TOTAL	259.69	-	914.61	-	-944.56	4882.10

** Minus (-) sign shows the direction of the force opposite to the convention*

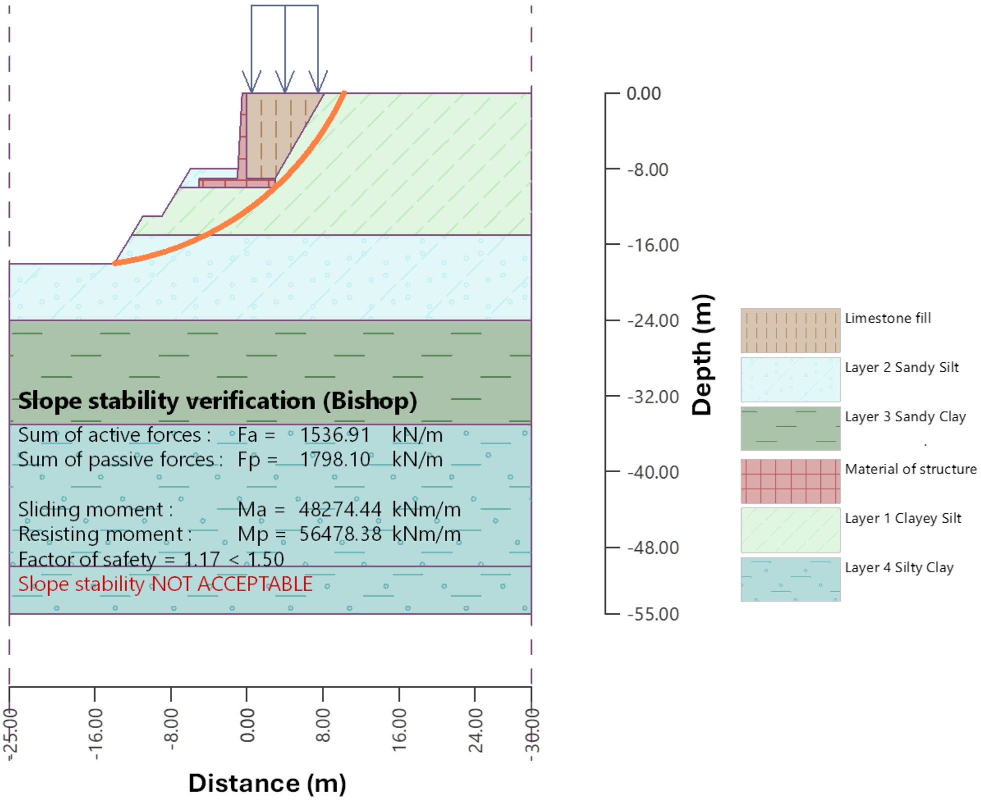


Figure 8 Global stability of conventional cantilever wall

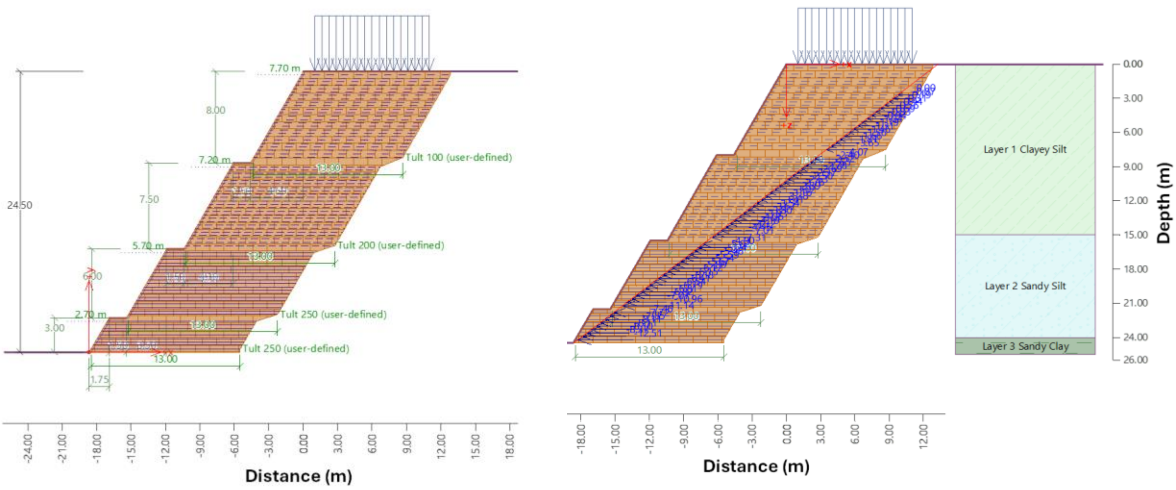
B. CANTILEVER WALL ANALYSIS

The MSE wall analysis was designed according to design chart of Jewell (1990). The first step is plotting the parameters β = 60° and φ = 30° to Figure 3a. The value of K_{req} = 0.14, and it is applied to Eq. 22 for some levels of depth to get the value of R_{req} as shown in Table 5. The tensile strength of geotextile material is defined by multiplies the R_{req} with some reduction coefficient according to Eq. 23, including RF_{CR} = 2.5, RF_D = 1.25, and RF_{ID} = 1.25. The chosen tensile strength of geotextile should comply the condition of $Rt > R_{req}$.

Table 5 Detail calculation of T_{ult} design

z (m)	R_{req} (Eq. 22) (KN/m)	T_{ult} req (Eq. 23) (KN/m)	T_{ult} Selected (KN/m)
8	21.56	84.22	100
15.5	40.46	158.05	200
21.5	55.58	217.11	250
24.5	63.14	246.64	250

The second step is plotting the parameters β = 60° and φ = 30° to Fig. 3b and 3c to obtain $\left(L_R/H\right)_{ovrl}$ = 0.5 and $\left(L_R/H\right)_{ds}$ =0.45. The total height of the failure slope will be rehabilitated (H) is 24.5 m, so the minimum required



(a) (b)

Figure 9 (a) Geometry of MSE wall in Geo5 and (b) result of internal stability analysis

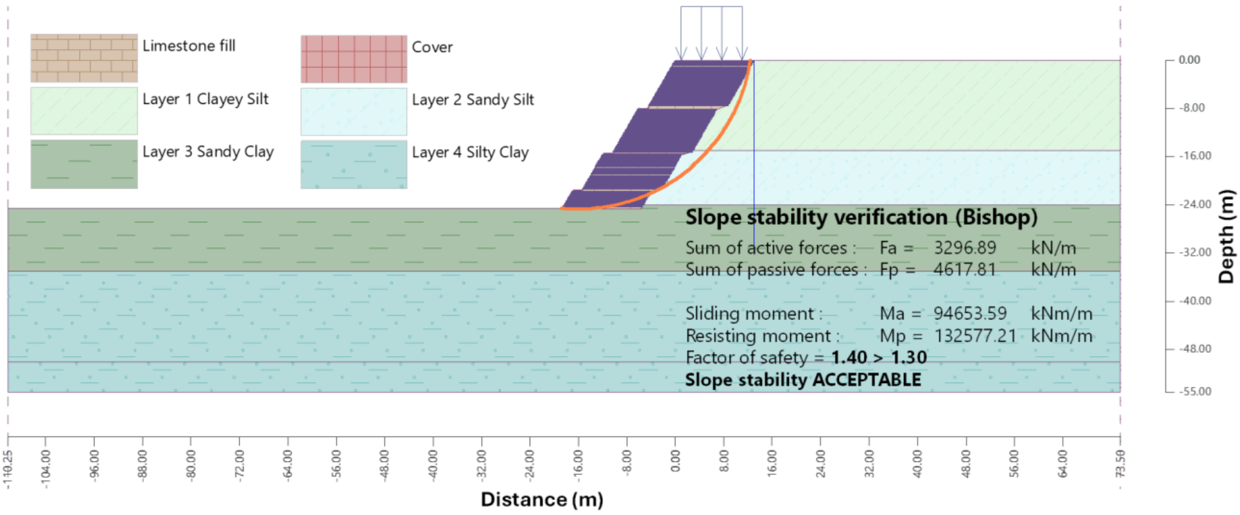


Figure 10 Slope stability analysis result of MSE wall

length or overall stability and direct sliding is 12.25 m and 11.025 m, respectively. Therefore, for the conservative approach, the length of geotextile inside the slope is 13 m.

The third step is doing modelling of this configuration in MSE wall module of Geo5 software (Figure 9a). The design verification is performed in the aspect of bearing capacity, internal stability, and global stability. The bearing capacity is done manually by using total stress analysis or undrained parameter. The undrained shear strength (S_u) of soil beneath the MSE wall is the third layer which have N-design value = 23, so by follow correlation $S_u = 6 \times N = 6 \times 23 = 138\ kN/m^2$ as suggested by some references [6] [7] [8]. Therefore, the critical height ($H_{critical}$) that could be supported by this soil layer is equal to

$$5.14 \times Su / \gamma \times SF = \frac{5.14x138kN / m^2}{18kN / m^3 \times 1.5} = 26.27m > 24.5m,$$

so it is satisfactory.

The internal stability of geotextile against pulls out stress due to the potential of slip surface is shown in Figure 9b. All the levels of geotextile are satisfactory the internal stability. While, global stability is performed by using limit equilibrium approach, same as the analysis of cantilever wall (Figure 10).

CONCLUSIONS

The mechanically stabilized earth wall gives more advantage in meeting all criteria, including bearing capacity, internal stability (overturning and sliding), and global stability. The SF global stability of MSE wall is 1.40 which is bigger than the minimum requirement 1.30 of SNI 8460:2017. On the other hand, conventional cantilever walls show a deficiency in SF global stability of 1.17 that will require pile foundation support or another reinforcement under the base plate, which will automatically increase construction time. The use of computer software will provide a more sophisticated analysis to examine various slope failure remediation methods to establish the most suitable solution for the specific sites. There is still expansive room to be researched in this area.

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