Numerical Analysis of Multi-Level Gravity Walls in Tupa Village, Bulango Utara-Bone Bolango District by Limit Equilibrium Methods

1st Indriati Martha Patuti
Department of Civil Engineering, Faculty of Engineeering
Universitas Negeri Gorontalo
Gorontalo, Indonesia
indri.m.patuti@ung.ac.id

2nd Ahmad Rifa'i*
Department of Civil and Environmental Engineering
Universitas Gadjah Mada
Yogyakarta, Indonesia
ahmad.rifai@ugm.ac.id
*Corresponding Author

3rd Kabul Basah Suryolelono
Department of Civil and Environmental Engineering
Universitas Gadjah Mada
Yogyakarta, Indonesia
kabulbasah@yahoo.com

4th Suprapto Siswosukarto
Department of Civil and Environmental Engineering
Universitas Gadjah Mada
Yogyakarta, Indonesia
suprapto_siswosukarto@ugm.ac.id

***Abstract*—Boundary equilibrium method is one of the methods applied in numerical analysis in the geotechnical field. Slope stability is determined by the values of safety factors. This is obtained from various methods, i.e. discussing the usual method or Fellenius method, Bishop method, Bishop Simple method, Simple Janbu Method, Spencer Method, and General Limit Equilibrium method, GLE / Morgenstern-Price method. Pore water pressure, soil shear strength, and slope geometry are factors that can affect slope stability. The slope stability analysis was carried out on the slope by multi-level gravity walls. The research location is at the natural slope in Tupa Village, North Bulango, Bone Bolango District. The natural slope had collapsed (FS<1.0). After reinforcing the slope with a multi-level gravity walls, the slope became stable. The safety against sliding, FS_s = 1.65 > 1.5, the safety against the overturning, FS_o = 3.06 > 2.0, the safety against the bearing capacity failure, FS = 6.48 > 3.0, and the safety factor for global slope (using GLE/Morgenstern-Price method), FS = 1.64 > 1.5. The results of the study show how to increase slope stability increases with increasing shear strength. However, more pore pressure can cause a significant decrease in safety factors. Furthermore, increasing the load on the slope can also reduce the value of the safety factor.**

Keywords—Limit Equilibrium Methods, Slope Stability, Safety Factors

I. INTRODUCTION

Bone Bolango District-Province of Gorontalo is generally a mountain area (43% of the total area). The occurrence of landslide in this type of area is frequent, especially in some villages/sub-districts, such as in Tupa Village, North Bulango Sub-district [1]. For road slope stability, single level gravity walls are usually used with an average slope height of 1-2 m, although in hilly areas with slope heights of more than 10 m as shown in Fig.1. The single level gravity walls are not efficient to prevent landslides that often occur in wet season. For the reason, multi-level retaining walls need to be designed. Researchers have conducted many studies on the design and analysis of retaining walls [2-8].

Fig.1. Gravity walls in Bone Bolango District

II. LITERATURE REVIEW

A. *Gravity Walls*

The stability of gravity walls depends on the geometry and the weight. The base width should be large enough to avoid tensile stresses within the wall. These stresses are caused by lateral earth pressure, which can also cause base slipping and toppling failure [9]. The following are the criteria for analyzing the stability of a retaining wall [9, 10]:

1. The factor of safety against sliding, $FS_s \geq 1.5$

$$FS_s = \frac{F_r}{F_s} \quad (1)$$

where:

FS_s : factor of safety against sliding,
F_s : resultant of forces tending to move the wall (earth pressures E_A or E_P),

F_r : resisting forces = $A\ (c' + \sigma'_v \tan \delta)$,
A : base wall area per lineal metre = $B \times 1$ (m^2),
c' and δ : strength parameters, soil cohesion, c' (kN/m^2) and wall base friction, $\delta = 1/3 - 2/3\varphi$ (°),
σ'_v : effective stress at the wall base (kN/m^2) = $\gamma_w.H$,
γ_w : effective unit weight of the wall (kN/m^3),
H : height of wall (m).

2. The factor of safety against overturning, $FS_o \geq 2.0$

$$FS_o = \frac{\sum M_r}{\sum M_s} = \frac{(W.a) + \alpha.E_p.b}{E_A.c} \tag{2}$$

where:
ΣM_r : resisting moment factor of safety against sliding,
ΣM_s : moment leading to overturning of forces tending to move the wall (earth pressures E_a or E_p),
W : weight of the wall or the soil (kN),
a : base wall area per lineal metre = $B \times 1$ (m^2),
α : reduction factor = 0 – 0.5, 1/3 is usually adopted in most cases,
E_A, E_p : active earth pressure, passive earth pressure (kN),
γ_w : effective unit weight of the wall (kN/m^3),
H : height of wall (m).

If the base of the wall is embedded into the soil, as in Fig. 2, the stability effect of the passive thrust could be considered in the analysis.

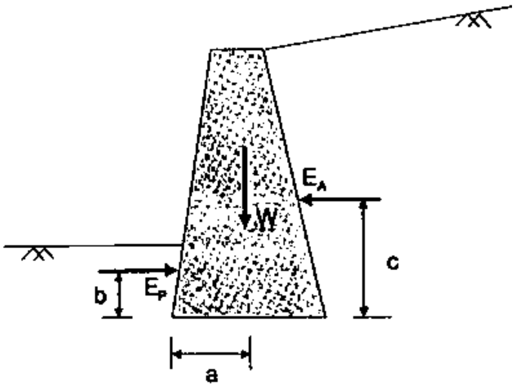

Fig. 2. Overturning failure stability check [9]

3. The factor of safety against bearing capacity, $FS \geq 3.0$

$$FS = \frac{q_{max}}{q} \tag{3}$$

The trapezoidal diagram of stress distribution at the wall foundation as shown in Fig. 3. The maximum and minimum stresses are given by:

$$\sigma_{max,min} = \frac{\sum F_v}{A}\left(1 \pm \frac{6e}{B}\right) \tag{4}$$

$\sigma_{max} \leq q_{max}/3$, where q_{max} is the bearing capacity stress calculate through the classical Terzaghi-Prandtl approach:

$$q_{max} = c'.N_c + q_s.N_q + 0.5.\gamma.B.N_\gamma \tag{5}$$

where:
ΣF_V : sum of vertical forces,
A and B : area (m^2) and breadth of wall base (m),
E : eccentricity of the resultant force N in relation to the base centre (m),
$B' = B - 2e$: wall breadth (m),
c' : soil cohesion (kN/m^2),
γ : unit weight of foundation soil (kN/m^3),
N_c, N_q, N_γ : bearing capacity factors,
q_s : effective overburden stress at the foundation level = $D_f.\ \gamma$
$\sigma_{min} \geq 0$

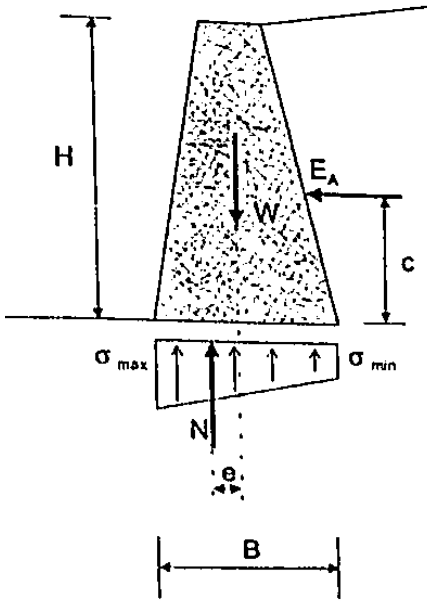

Fig. 3. The forces on gravity walls [9]

4. The factor of safety against global slope, $FS \geq 1.2 - 1.5$

The geometry of the gravity walls must produce the forces that are in the middle of a wide or $e<B/6$ distance. The thickness of the top wall is $0.3\text{-}H/12$ meters.

B. Global Slope Stability by The Limit Equilibrium Methods

The factor of safety, FS is defined with respect to the shear strength of the soil as:

$$FS = \frac{s}{\tau} \tag{6}$$

Where s is the available shear strength and τ is the equilibrium shear stress. The equilibrium shear stress is the shear stress required to maintain a just-stable slope. The equilibrium shear stress is equal to the available shear strength divided by the factor of safety. The procedures use to perform such computations are known as limit equilibrium procedures. The shear strength can be expressed by the Mohr-Coulomb Equation [11]:

$$\tau = \frac{c}{F} + \frac{\sigma \tan \varphi}{F} \tag{7}$$

All limit equilibrium methods (ordinary method of slices, Bishop simplified, Janbu simplified, Spencer's, Sarma's, and Morgenstern-Price, and others) for slope stability analysis divide a slide-mas into n smaller slices. Each slice is affected by a general system of forces [10].

III. RESEARCH METHODS

The research was conducted at the landslide in Tupa Village, North Bulango Sub-district. The topography of this area indicates a steep slope (45°-60°). The soil characteristic were obtained by hand drilling at the foot of the slope, the middle of the slope, and the top of the slope [1]. The rock shear strength parameters in this study were determined based on Geological Strength Index analysis. This analysis was carried out with the help of RocLab software. The properties of slope material in Tupa Village that used in limit equilibrium analysis is shown in Table I [12]. The unit weight of masonry gravity retaining wall was 22 kN/m^3. The method used in the global analysis of slope stability is limit equilibrium methods (General Limit Equilibrium (GLE)/Morgenstern-Price Methods) with the help of computer programs, namely Slide 2D. Based on the analysis using the GLE method, this natural slope is not stable. The factor of safety was 0.945 during dry season, and FS= 0.839 while in wet season as shown in Fig. 4 and Fig.5 [12]. There was a decrease in the value of safety factors by 11.22% One that affects the instability of the slope is a very steep slope.

TABLE I. THE PROPERTIES OF SLOPE MATERIAL IN TUPA VILLAGE FOR THE LIMIT EQUILIBRIUM ANALYSIS [12]

Material Properties	**Unit**	**Layer 1 (Silty sand)**	**Layer 2 (Quartz Diorite)**
Water content, w	%	8.18	0.81
Unit weight, γ_b	kN/m^3	17.89	25.17
Cohesion, c'	kN/m^2	3.53	130.00
Internal friction angle, φ'	°	38	31.00

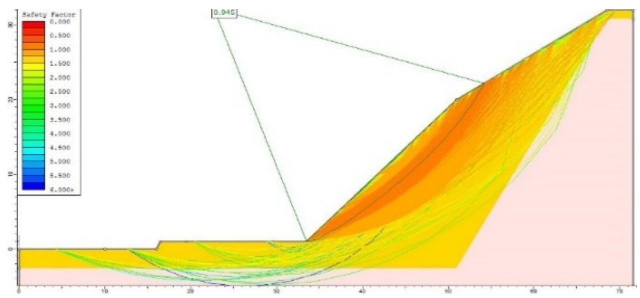

Fig.4 Natural slope analysis in Tupa Village by GLE Method during the dry season [12]

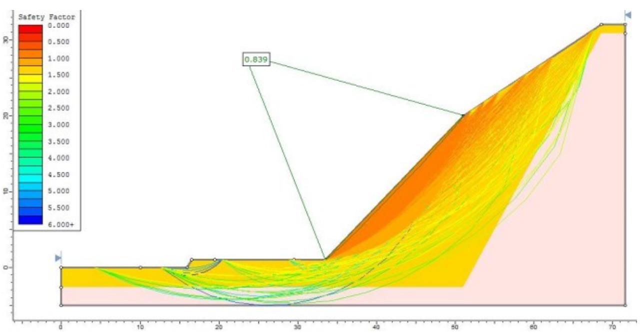

Fig.5 Natural slope analysis in Tupa Village by GLE Method during the rainy season [12]

IV. RESULT AND DISCUSSION

A. *Design of Multi-level Gravity Walls*

The design of multi-level gravity walls as shown in Fig. 6 and Fig.7. The height of the wall is H=5 m, base width of the wall is B=2.5 m, and thick of the top wall is 0.4 m. The gravity walls consisted of 7 level with a distance between the sides of the top wall to wall foot is 4 m.

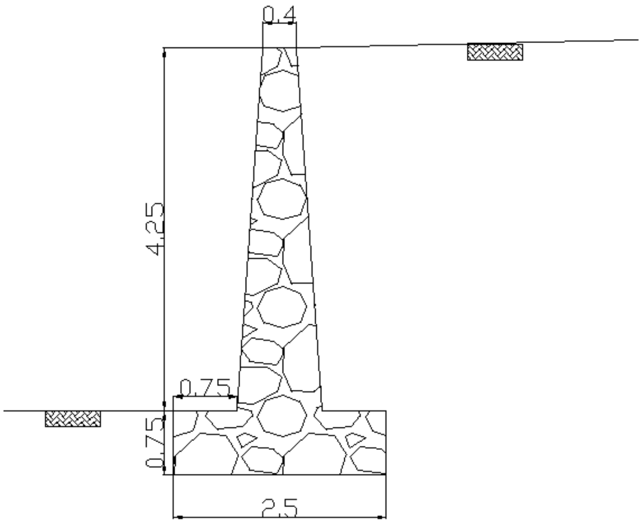

Fig.6 Dimensions of the masonry gravity walls

The results of the stability analysis of multi-level retaining walls in Tupa Village are based on Eq. (1)-(7). Table II presented of vertical force and moment analysis.

TABLE II. THE CALCULATE OF VERTICAL FORCES ON THE WALLS AND MOMENT AGAINST FRONT FOOT

Source	**Vertical Force (kN)**	**Distance to O (m)**	**Moment to O (kN.m)**
Wall (stem)	0.5 x 4.25 x 0.3 x 22	0.95	13.32
Wall (stem)	0.4 x 4.25 x 22	1.25	46.75
Wall (Stem)	0.5 x 4.25 x 0.3 x 22	1.55	21.74
Footing	2.5 x 0.75 x 22	1.25	51.56
Soil	0.5 x 4.25 x 0.3 x 17.89	1.65	18.82
Soil	0.75 x 4.25 x 17.89	2.125	121.18
	Σ = 175.13		Σ = 273.37

The active earth pressure coefficient is equal to:

$$k_A = \tan^2\left(45^o + {}^{38}/_{2}\right) = 0.24$$

The active earth pressure is:

$$E_A = 0.5 \times \gamma \times \mathrm{H}^2 = 0.5 \times 17.89 \times 5^2 = 53.67\ kN$$

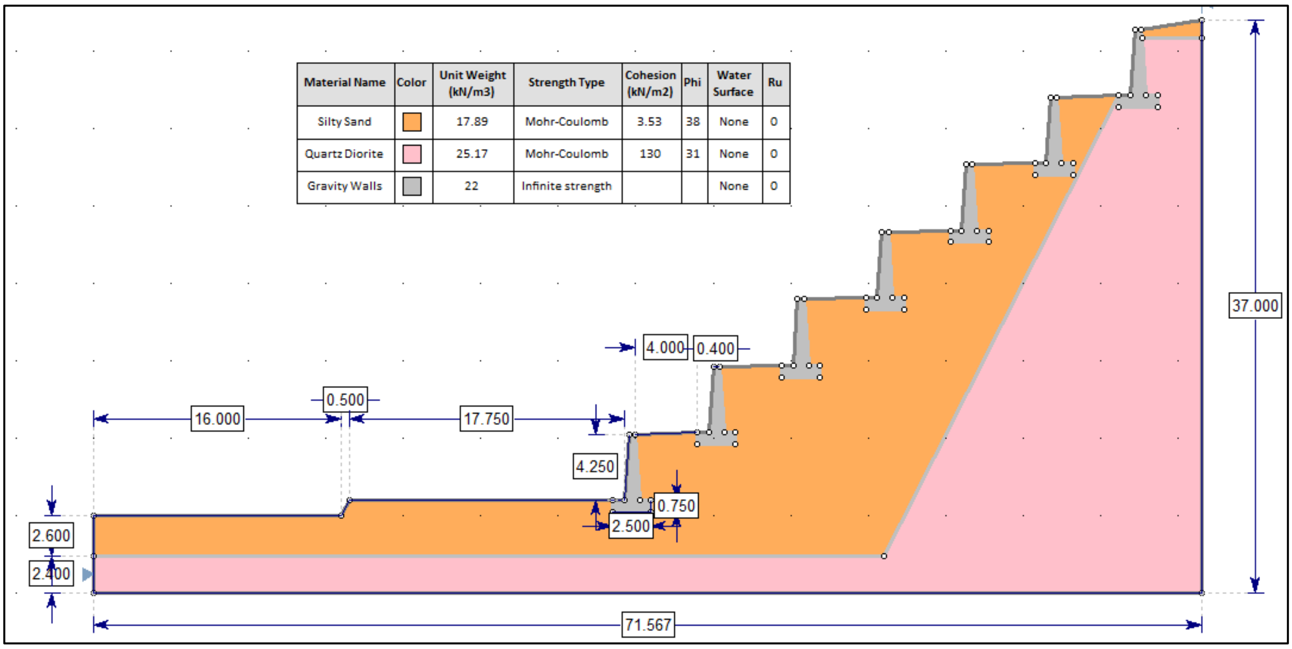

Fig.7 The geometry of multi-level gravity walls

B. Stability Check for Sliding

$$\delta_B = {}^2/_3\, \varphi_B = {}^2/_3 \times 38 = 25.3°$$

$$FS_s = \frac{F_r}{F_s} = \frac{A\,c' + \sigma' v \tan \delta_B}{E_A}$$

$$= \frac{2.5 \times {}^2/_3 \times 3.53 + 175.129 \times \tan 25.3}{53.67}$$

$$= 1.65 \geq 1.5 \ \ \text{(Ok)}$$

C. Stability Check for Overturning

$$FS_o = \frac{\sum M_r}{\sum M_s} = \frac{(W.a) + \alpha.E_p.b}{E_A.c}$$

$$= \frac{273.37}{53.67 \times {}^5/_3} = 3.06 \geq 2.0 \text{ (Ok)}$$

D. Stability Check for Bearing Capacity

The position of resultant force from O point:

$$x = \frac{273.37 - 53.67 \times {}^5/_3}{175.129} = 1.05 \text{ m from } O$$

$$e = {}^{2.5}/_2 - 1.05 = 0.2\,m < {}^B/_6 = 0.42\,m$$

$$q_{max} = c'.N_c + q_s.N_q + 0.5.\gamma.B.N_\gamma$$

$$= (3.53 \times 31.02) + (0.75 \times 17.89 \times 17.34) + (0.5 \times 17.89 \times 2.5 \times 15.32)$$

$$= 684.75 \text{ kN/m}^2$$

$$q_{un} = 684.75 - (0.75 \times 17.89) = 671.34 \text{ kN/m}^2$$

$$q_a = \frac{671.34}{3} + 0.75 \times 17.89 = 237.19 \ \text{kN/m}^2$$

The maximum and minimum stresses are given by:

$$\sigma_{max} = \frac{\sum F_v}{A}\left(1 \pm \frac{6e}{B}\right)$$

$$= \frac{175.129}{2.5}\left(1 + \frac{6 \times 0.2}{2.5}\right)$$

$$= 103.68 \text{ kN/m}^2 < q_a = 237.19 \text{ kN/m}^2 \text{ (Ok)}$$

$$\sigma_{min} = \frac{\sum F_v}{A}\left(1 \pm \frac{6e}{B}\right)$$

$$= \frac{175.129}{2.5}\left(1 - \frac{6 \times 0.2}{2.5}\right) = 36.43 \text{ kN/m}^2 > 0 \text{ (Ok)}$$

$$FS = \frac{q_{max}}{q} = \frac{671.34}{103.68} = 6.48 \ > 3 \text{ (Ok)}$$

E. Stability Check for Global Slope by GLE Methods/Morgenstern-Price

The global stability analysis of the slope is based on limit equilibrium concept using the GLE/Morgenstern-Price method (Fig. 8). The results of the analysis by 2D Slide software show the value of the safety factor, FS=1.64>1.5. This value describes that the slope is stable.

The summary of slope analysis with reinforcement by multi-level gravity walls as described in Table III. Safety against the sliding, FS_s = 1.65, safety against the overturning, FS_o = 3.06 safety against of the bearing capacity failure, FS = 6.48, and the safety factor of global slope, FS = 1.64. There was a significant increase in the safety factors of global slope, which was 73.54%. The design multi-level gravity walls expected this to be one of the alternatives in selecting the type of slope reinforcement.

TABLE III. THE SUMMARY OF STABILITY ANALYSIS ON MULTI-LEVEL GRAVITY WALLS IN TUPA VILLAGE

Description	*FS*
Stability check for sliding	1.65 > 1.5
Stability check for overturning	3.06 > 2.0
Stability check for bearing capacity	6.48 > 3.0
Stability check for global slope by General Limit Equilibrium Method (GLE)/Morgenstern-Price	1.64 > 1.5

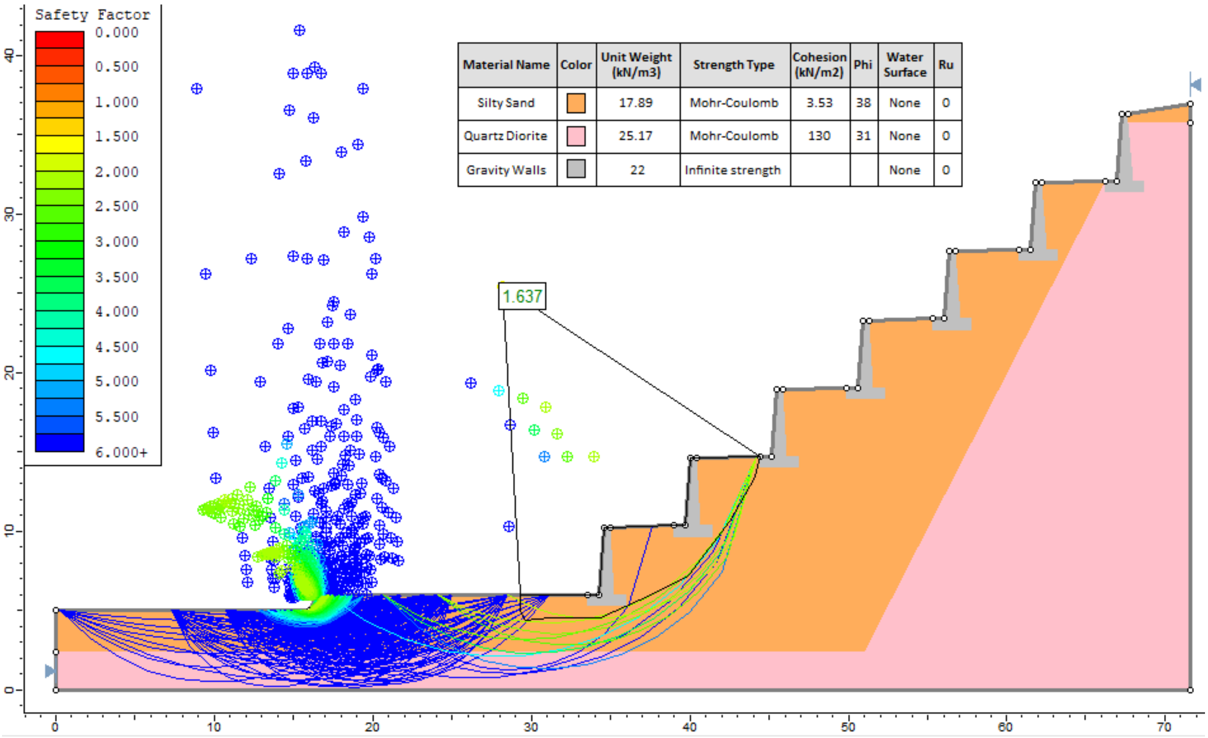

Fig.8 The analysis slope stability by Slide 2D

V. Conclusions

Based on the results of the multi-level gravity wall analysis by the equilibrium limit methods (GLE/Morgenstern-Price Method), it concluded that the slope in Tupa Village was stable. Safety against the sliding, $FS_s = 1.65$, safety against the overturning, $FS_o = 3.06$ safety against of the bearing capacity failure, FS = 6.48, and factor safety of global slope, FS = 1.64. There was a significant increase in the safety factors of global slope, which was 73.54%. The design multi-level gravity walls expected this to be one of the alternatives in selecting the type of slope reinforcement.

Acknowledgment

The author acknowledges the support and assistance during the soil and rock material testing to the head and staff of -the Soil Mechanics Laboratory and Structures Laboratory of Gadjah Mada University.

References

[1] I. M. Patuti, A. Rifa'i and K. B. Suryolelono, "Mechanism and Characteristics of The Landslides in Bone Bolango Regency, Gorontalo Province, Indonesia," *International Journal of GEOMATE,* vol. 12, no. 29, pp. 1-8, Jan 2017.

[2] S.-j. JIN, J.-m. XIONG and T.-q. YU, "Experimental Study on Deformati of Multistage Retaining Wall," in *ICCTP*, 2010.

[3] R. Jalla, "Design of Multiple Level Retaining Walls," *Journal of Architectural Engineering,* vol. 5, no. 3, pp. 82-88, 1999.

[4] S. Sharma, "Teaching Retaining Wall Design with Case Histories," in *Geo-Frontiers Congress 2011*, Dallas, Texas US, 2011.

[5] H.-H. (. Chiang, J. B. Kerrigan and D. E. Bennetts, "Case History-Southlands Orchard Road Retaining Walls," in *Biennial Geotechnical Seminar Conference*, Colorado, US, 2008.

[6] L. Wei, T. Koutnik and M. Woodward, "A Slope Stability Case Study by Limit Equilibrium and Finite Element Methods," in *GeoFlorida 2010*, Orlando, Florida US, 2010.

[7] Y. Liu, C. Wang and Q. Yang, "Stability Analysis of Soil Slope Based on Deformation Reinforcement Theory," *Finite Element in Analysis and Design,* vol. 58, pp. 10-19, October 2012.

[8] I. M. Patuti, A. Rifa'i, K. B. Suryolelono and S. Siswosukarto, "Model of Timber Crib Walls Using Counterweight in Bone Bolango Regency Gorontalo Province Indonesia," *International Review of Civil Engineering (I.RE.C.E.),* vol. 9, no. 3, pp. 98-104, May 2018.

[9] J. A. Ortigao dan A. S. Sayao, Handbok of Slope Stabilisation, 1st penyunt., J. A. Ortigao dan A. S. Sayao, Penyunt., Berlin: Springer, Verlag, 2004.

[10] L. W. Abramson, T. S. Lee, S. Sharma and G. M. Boyce, Slope Stability and Stabilization Methods, Kedua ed., New York: John Wiley dan Sons, 2002.

[11] J. M. Duncan, S. G. Wright and T. L. Brandon, Soil Strength and Slope Stability, New Jersey: John Wiley & Sons, Inc., 2014.

[12] I. M. Patuti, A. Rifa'i and K. B. Suryolelono, "The Effect of Water Content Change in Plutovolcanic Subsurface Slope Stability Based on Limit Equilibrium and Finite Element Methods," in *The 1st Warmadewa University International Conference on Architecture and Civil Engineering*, Denpasar, 2017.