OPTIMIZATION OF REINFORCED SLOPE GEOMETRY USING CEMENT-STABILIZED SOIL WASTE

[1]CHETRA HAM, [2]THAILENG OENG, [3]SOUNEAN HENG

[1,2]Faculty of Civil Engineering, Institute of Technology of Cambodia, Russian Federation Blvd, P.O. Box 86, Phnom Penh, Cambodia
[3]Research and Innovation Center, Institute of Technology of Cambodia, Russian Federation Blvd., P.O. Box 86, Phnom Penh, Cambodia
E-mail: [1]hamchetra18@gmail.com, [2]oeng.thaileng@itc.edu.kh, [3]souneanheng55@gmail.com

Abstract - This research presents the reusing of local natural soil waste as stabilized backfill of reinforced slope. Initial step of this study is to analyze of the existing structure. Contiguous bored piles are used as a retaining wall with ground anchors. The analysis is performed under computer-aided simulation. Mohr-Coulomb and Hoek-Brown models are used for soil and rockcriterion, respectively. For soil properties, cohesive strength and friction angle are estimated from the plasticity index. For rock criterion, which is adopted (Hoek-Brown criteria, rock mass modulus, UCS, and GSI) are transferred from Rock Quality Designation (RQD). After that, the redesign of various vertical slopes' steepness is performed with untreated and treated soil as backfill. The treated soils consist of 6, 8, 10, and 12 percent of cement by dry mass. The properties of this soil are derived from lab experiments. Slope stability analysis comprises both finite element modeling (computer-aided simulation) and numerical computation. An algorithm for computing Bishop's simplified method of slices by Newton Raphson Method is developed. Results reveal that increasing cement content significantly improves soil strength, making the structure safer. Steep slopes provide lower safety factors, while the failure consist at toe. Simulation and numerical analysis results show strong agreement, validating the reliability of both methods. Two optimum cases are proposed as the alternative to the conventional structure.

Keywords - Computer-Aided Simulation, Numerical Computation, Slope Stability, Soil Reinforcement

I. INTRODUCTION

While soil waste management in Cambodia is not yet perceived as a major issue, like other developing countries, as Cambodia continues to experience urbanization and economic development, it is essential to take proactive measures to address soil waste before it becomes a critical challenge [1], [2].

Soil stabilization with cementis a proven effective method in increasing its strength[3], [4], [5], [6].There are many researchers who have attempted and proven the feasibility of slope stabilization using numerical analysis and computer aids. Shafna & Anjana [7] proposed a method of analysis for soft soil embankment. PLAXIS software simplified and enhances the practicality of the analysis. The factor of safety is primarily influenced by factors such as slope height, soil cohesion, and overall soil conditions, while an increase in soil cohesion leads to a higher factor of safety. Similarly, Abbas [8] stated that for clayey soil, the slope angle results in a higher factor of safety and lower risk compared to sandy soil. Deformation mostly occurs at the toe of the slope, with vertical displacement observed at the tip. Sharma et al. [6] studied on analyzing slope stability using Bishop's method integrated with a MATLAB program that applies the Particle Swarm Optimization (PSO) technique. The approach employed the method of slices based on limit equilibrium principles and static equilibrium to identify the critical slip surface and calculate the minimum factor of safety. Salunkhe et al. [10] performed a laboratory test to evaluate the stability of steep slopes made of fly ash, with and without the addition of recycled plastic polymer, placed on soft foundation soils such as fly ash and Powai soil. The structures were analyzed using experimental investigation by Bishop's simplified method and PLAXIS 3D.

Infrastructure located near uneven terrain, especially mountain passes, must be designed to withstand erosion and the risk of landslides. The quality and stability of the soil and rock beneath the road must be evaluated and analyzed to make sure they are suitable for the design. Maintaining slope stability is critical for ensuring the long-term integrity of structures to prevent and mitigate the damage caused by slope failure.

This paper proposes the reusing of excavated soil from the same site with the same moisture content as stabilized reinforced backfill, particularly addressing the weak soil conditions and the potential for stabilization using cement. The research's objectives are: (*i*) to study the stability of the existing structure (with retaining wall), (*ii*) to investigate the correlation between the provided data and the parameters needed for the analysis, (*iii*) to redesign the structure into a man-made slope (without retaining walls) and simulate the stability without and with stabilized backfill, and (*iv*) to validate the safety factor result between numerical analysis and simulation.

This case study tackled a slope along a segment of National Road No. 4 (PK 102 + 450 HRS) located in Ti Prammuoy village, Yeay Mao Pich Nil commune,

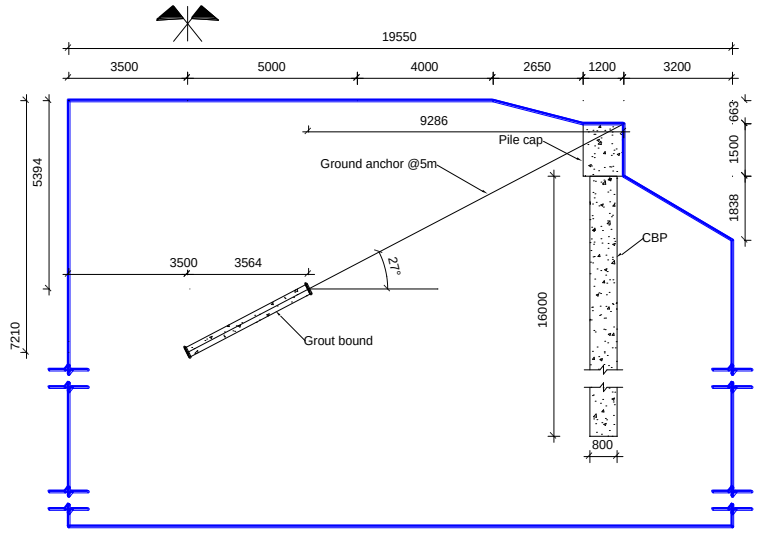
Phnom Sruoch district, Kampong Speu Province (11°12'16.5"N 104°05'46.3"E).

II. METHODOLOGY

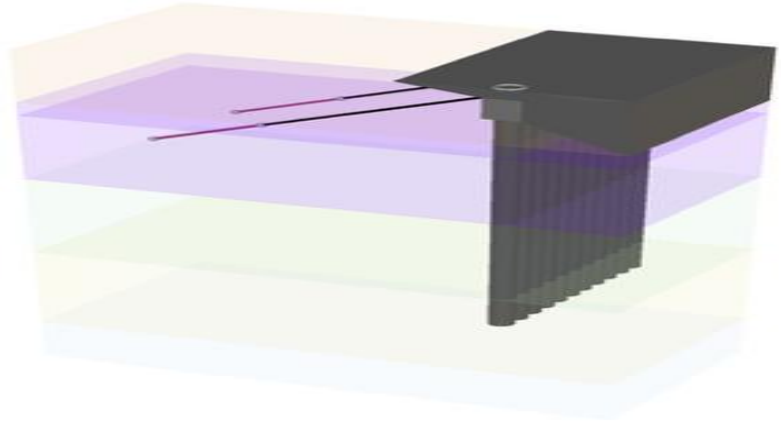
The methodologies of this research consists as follow.

A. Analysis of the existing structure

Contiguous bored piles were used as a retaining wall with ground anchors (See Fig. 1). Mohr-Coulomb and Hoek-Brown models were used for soil and rock, respectively (see Fig. 2). The analysis was performed under dry conditions.



(a)



(b)

Fig. 1.Geometry of the existing structure: (a) cross-sectional dimension, (b) 3D scheme.

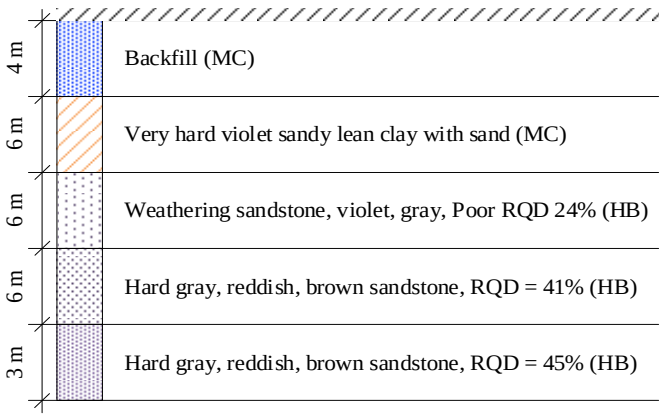


Fig. 2. Soil layer in the borehole for simulation.

B. Parameter estimation

For soil properties, cohesive strength and friction angles were estimated from the plasticity index. For rock, Hoek-Brown criterion (rock mass modulus, UCS, and GSI) were transferred from Rock Quality Designation (RQD).

1) Parameters of clay soil

According to Tchakalova & Ivanov [11], cohesive strength *c*, given in (1) appeared to be appropriate to a range of clays with a plasticity index,*PI*, of 11% to 78%,

$$c = 0.776\ PI + 0.8476 \quad (1)$$

For friction angle, φ, Winn *et al.*[12] found it decreased with increasing clay content with a value of φ, it was expressed as

$$\varphi = 0.14\ (234.5 - \%\text{Clay}) \quad (2)$$

The value of Young's modulus for clay is from Table 1 where its description based on cohesion (Table 2) and *PI* (Table 3).

USCS	Description	V. soft to soft	Medium	Stiff to V. stiff	Hard
ML	Silts with slight plasticity	2.5 - 8	10 - 15	15 - 40	40 - 80
ML, CL	Silts with low plasticity	1.5 - 6	6 - 10	10 - 30	30 - 60
CL	Clays with low-medium plasticity	0.5 - 5	5 - 8	8 - 30	30 - 70
CH	Clays with high plasticity	0.35 - 4	4 - 7	7 - 20	20 - 32
OL	Organic silts	-	0.5 - 5	-	-
OH	Organic clays	-	0.5 - 4	-	-

Obrzud &Truty [13] afterKézdi & Rétháti [14], Prat *et al.*[15]

Table 1. Typical Young's modulus (MPa) for cohesive soil.

Soil condition	Very soft	Soft	Medium	Stiff	Very stiff
Cohesion (kPa)	< 24	24-48	48-96	96-192	192-383

After Rendana *et al.*[16]extract from Terzaghi *et al.*[17]

Table 2. Classification of cohesion values for soil.

PI	Description
0	Non-plastic
1-5	Slightly plastic
5-10	Low plasticity
10-20	Medium plasticity
20-40	High plasticity
>40	Very high plasticity

Table 3. Plasticity index in a qualitative by Burmister [18].

Turk & Dearman [19] estimated Poisson's ratio, *v*, by

$$v = (1 - \sin\varphi) / (1 + \sin\varphi) \quad (3)$$

2) Parameter of sandstone

According to Hoek [20], the material constant for sandstone is 15 and when slope construction with excavation machinery, $D = 0.7$ was chosen.Hoek *et al.*[21]considered

$$GSI = \begin{cases} RMR - 5, & RMR > 23 \\ RMR, & RMR < 23 \end{cases} \quad (4)$$

where *RMR* is the Rock Mass Rating developed by Bieniawski [22]. By not having access to the other properties, the only approach to determine *RMR* was direct comparison with *RQD* (Table 4).

Notation	RQD (%)	USC (kPa) [1]	*RMR* (%) [2]
Excellent	100	28,000	100
Good	90	19,000	80
Passable	75	11,000	60
Bad	50	6,000	40
Very bad	25	2,800	20
-	0	900	0

[1] after Prakash & Sharma [23]

[2] after Bieniawski [22]

Table 4. Relation between *RQD*, *UCS*, and *RMR*

Kallu & Roghanchi [24]correlation between direct and indirect strength test methods of sandstone is

$$E = 2.17\ (\mathrm{UCS})^{0.57} \quad (5)$$

where UCS was estimated from Table 4.Narimani *et al.*[25]suggested (6) to estimate Poisson's ratio from *RMR*.

$$v = 0.5 - 0.2RMR\ /\ (0.8RMR + 20) \quad (6)$$

Table 5. shows the estimated values of each parameter.

Name	Unit	Layer*					
		1	2	3	4	5	6
γ_{unsat}	kN/m^3	23.00	23.00	20.12	23.58	24.23	24.56
γ_{sat}	kN/m^3	26.25	26.25	24.40	25.57	26.12	26.34
v	-	0.47	0.40	0.40	0.39	0.36	0.35
Criterion: Mohr-Coulomb							
E'_{ref}	kN/m^2	-	-	5.0e3	-	-	-
c'_{ref}	kN/m^2	-	-	21.23	-	-	-
φ	Deg	-	-	25.66	-	-	-
Criterion: Hoek-Brown							
E_{rm}	kN/m^2	2.5e6	3.6e6	-	3.8e6	5.3e6	5.7e6
$\lvert\sigma_{ci}\rvert$	kN/m^2	1.3e3	2.4e3	-	2.7e3	4.8e3	5.4e3
RMR	-	4	16	-	19.2	32.8	36
GSI	-	4	16	-	19.2	27.8	31

*Layer1. Gray, light gray sandstone, V.poor RQD 5%
2. Gray, violet sandstone, poor RQD 20%
3. Very hard violet sandy lean clay with sand
4. Weathering violet gray sandstone, Poor RQD 24%
5. Hard gray, reddish, brown sandstone, RQD 41%
6. Hard gray, reddish, brown sandstone, RQD 45%

Table 5. Values of the estimated soil/rock properties

C. Analysis of the proposed slope

The continuous bore piles and ground anchors were not considered. 4 m of the existing soil was replaced due to the weak nature of the layer 1 and 2. The proposed slope considered two cases of slope height of 4 m and 3 m with its boundary and ratio (*H*/*L*) shown in Fig. 3.

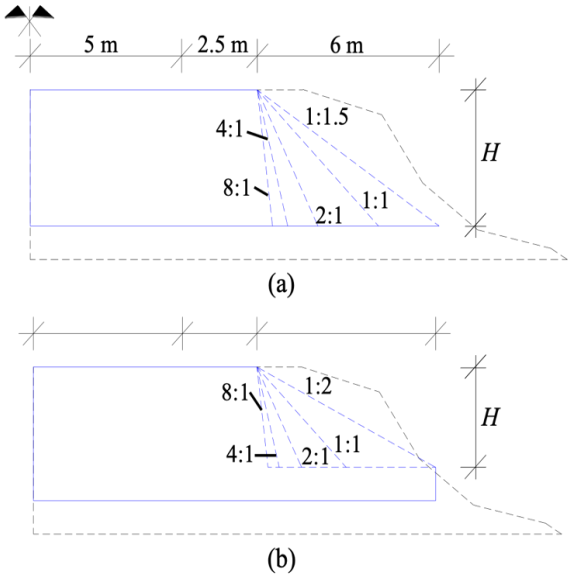


Fig. 3. The proposed slope:

(a) Case 1: *H* = 4 m, (b) Case 2: *H* = 3 m.

Although traffic loads are cyclic, the slope was analyzed using a static equivalent load for design simplicity and conservatism. The AASHTO[26] ESAL tire pressure (587 kPa) was applied as a uniformly distributed static load to represent the critical effect of heavy vehicles (see Fig. 4). Since vehicle loading duration is short and dynamic effects dissipate quickly within compacted embankment soils, the response can be reasonably approximated as

quasi-static. The adopted Load Safety Factor (LSF = 1.2, recommended by MPWT [27]) accounts for load variability and traffic repetition, making the static assumption suitable for stability evaluation.

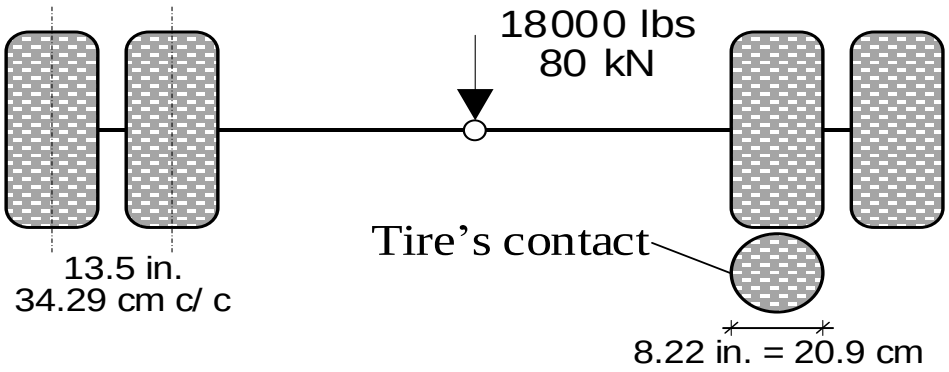


Fig. 4. Diagram presentation of the Standard AASHTO ESAL (Wheel loading: uniformly distributed pressure of 587 kPa, after Perraton *et al.*[28]).

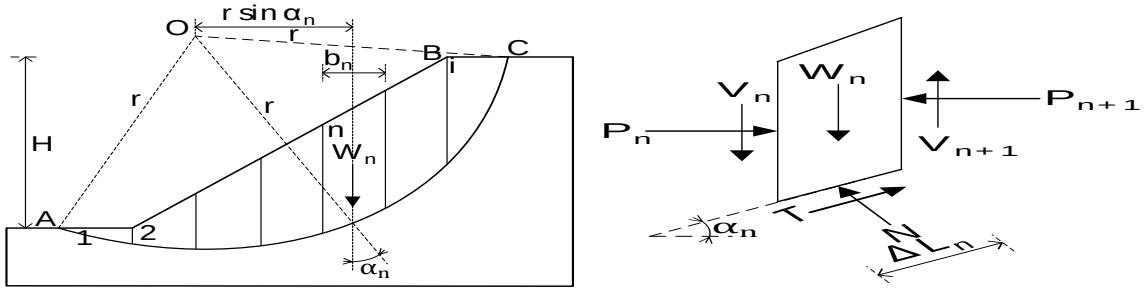
Table 6 presents the baseline properties of the untreated and treated backfill derived from mechanical testing at 14 days curing.

Name	Unit	% Cement				
		0	6	8	10	12
γ_{unsat}	kN/m^3	20.42	20.89	20.65	20.72	20.93
γ_{sat}	kN/m^3	22.14	22.61	22.22	22.26	22.55
e	-	0.3	0.3	0.3	0.29	0.27
E	kN/m^2	547.95	4.5e3	4.7e3	6.5e3	13.8e3
v	-	0.38	0.36	0.36	0.35	0.33
c'_{ref}	kN/m^2	34.03	174.84	198.28	254.58	302.60
φ	Deg	23.68	25.50	25.68	26.64	26.85

Table 6. Properties of ordinary and treated soil with cement

D. NUMERICAL COMPUTATION algorithm for slope stability

In limit equilibrium method, a failure mechanism is assumed (see Fig. 5), and equilibrium equations are used to determine the collapse load. It is also known as method of slices[29].



(a) (b)

Fig. 5. a) Failure surface, b) Forces acting on nth slice.

In 1955, Bishop proposed a refined solution to the method of slices. This method neglects only *V*, but the effect of forces on the sides of each slice (*P*) is accounted for to some degree

$$SF = \frac{\sum_{n=1}^{i}(c'b_n + W_n \tan\varphi')}{\sum_{n=1}^{i} W_n \sin\alpha_n [\cos\alpha_n + (\tan\varphi' \sin\alpha_n / SF)]} \quad (8)$$

which is a non-linear equation that in this case was solved by Newton-Raphson method [30]. The formular can be derived based on the Taylor series approximation[31] as

$$x_{n-1} = x_n - \frac{f(x_n)}{f'(x_n)} \quad (9)$$

The iterative loop algorithm of *SF* by Bishop's simplified method is presented in Fig. 6.

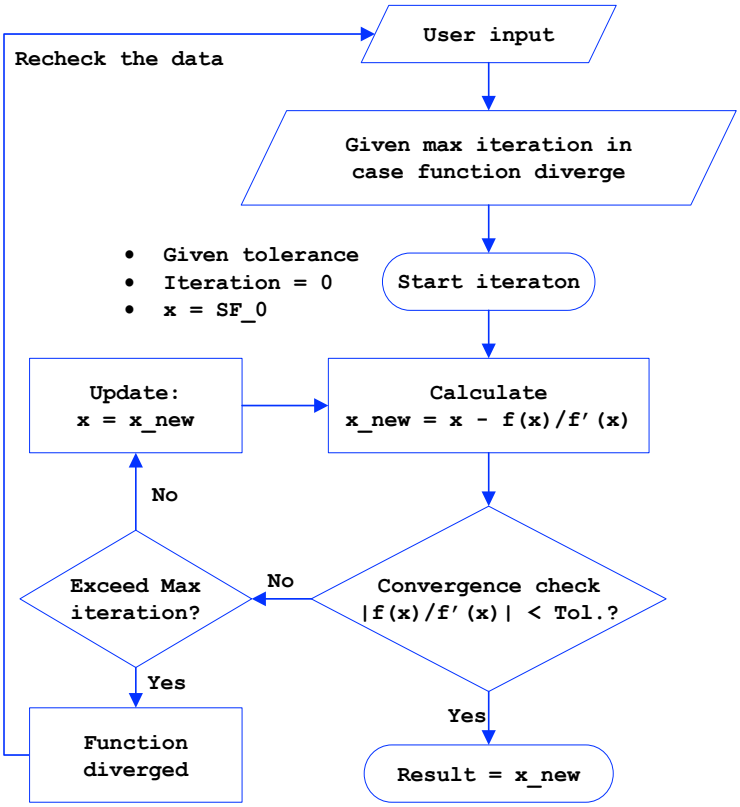


Fig. 6. Algorithm Bishop's simplified by Newton-Raphson.

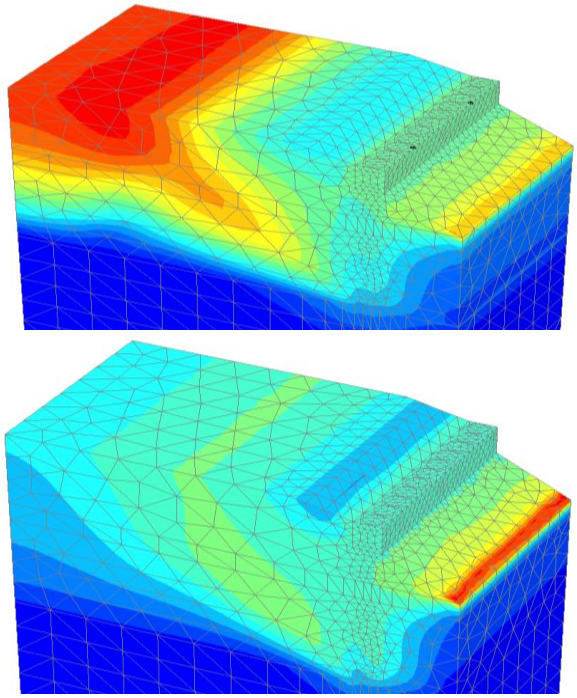
It starts with an initial guess, a tolerance, and a maximum number of iterations. At each step, a new approximation is computed using (9). If the correction term is smaller than the tolerance, the method converges and returns the root. Otherwise, the value is updated and the process repeats until convergence or until the maximum iteration limit is reached, indicating divergence.

III. RESULTS AND DISCUSSION

After analysis, the safety factor result of each case is shown and compared as following.

A. PLAXIS modeling result of existing structure

The simulation was achieved through soil characteristic in Fig. 2 with properties presented in Table 5, and structure properties (concrete, node-to-node anchor, and embedded beam). The ground water effect was not considered in this dry analysis and assumedrigid interface. The simulation results are presented in Fig. 7. In Fig. 7(a), the presence of a ground anchor results in a deeper and more extended failure surface. The anchor enhances resistance against soil movement, leading to higher safety (SF = 3.0). In Fig. 7(b), without the ground anchor, the failure surface is slighter and more localized, showing reduced stability and a lower safety (SF = 2.3).



(a) With anchor, SF = 3.0 (b) Without anchor, SF = 2.3

Fig. 7. Failure surface: (a) ordinary, (b) no ground anchor.

B. Results of the proposed slope

The results of simulation for case 1 (H = 4 m) and 2 (H = 3 m), consistently demonstrate that increasing the slope steepness leads to a reduction in SF (presenting in Fig. 8 and Fig. 9 respectively).

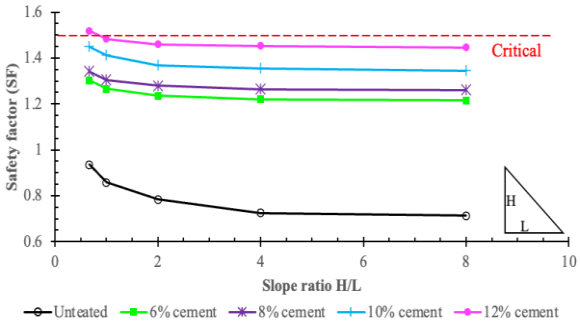


Fig. 8. Safety factor of the proposed slope case 1.

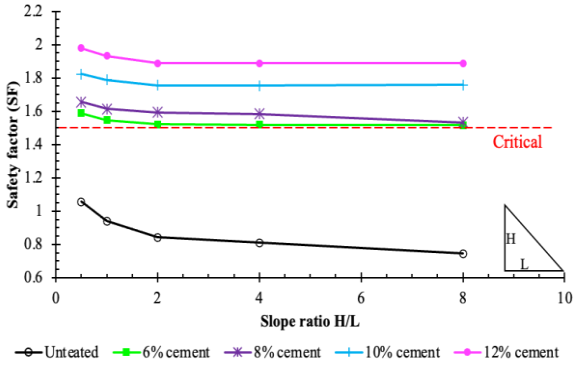
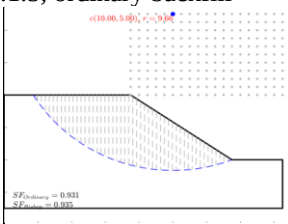
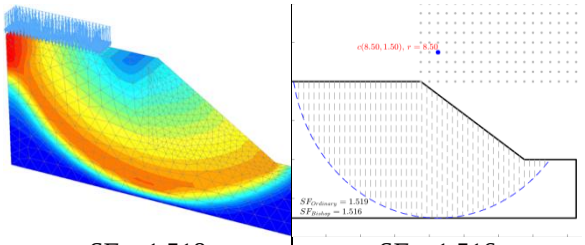


Fig. 9. Safety factor of the proposed slope case 2.

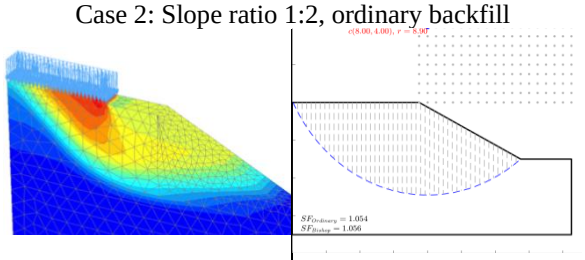
Table 7 shows the graphical representation of failure surfaces across different cases in how slope geometry and soil treatment influence the shape, depth, and location of the potential slip surface. For untreated backfill, the failure surface typically initiates near the slope crest and follows a shallow, laterally extended circular path that reaches the slope toe. This shape indicates low shear strength and reduced resistance to sliding. As the slope becomes steeper (e.g., from 8:1 to 1:2 or 1:1.5), the failure surface tends to deepen and become more confined.The introduction of cement treatment alters the geometry and depth of the failure surface substantially. Treated slopes exhibit failure circles that are either deeper or shifted downward, suggesting a redistribution of shear stress due to increased soil strength. Higher cement contents (10% and 12%) result in more stable and less critical failure mechanisms, with failure surfaces appearing further below the slope interface.

PLAXIS Modeling	NUMERICAL COMPUTATION algorithm
Case 1: Slope ratio 1:1.5, ordinary backfill	
Not Available*	
	SF = 0.935
Case 1: Slope ratio 1:1.5, 12% cement backfill	



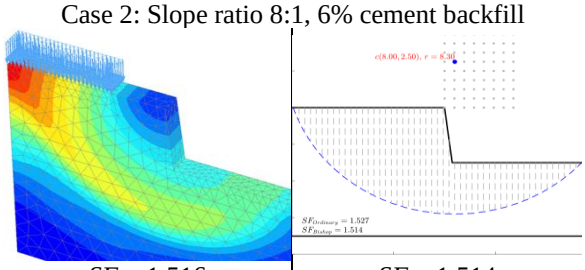
SF = 1.518	*SF* = 1.516

Case 2: Slope ratio 1:2, ordinary backfill



SF = 1.056	*SF* = 1.056

Case 2: Slope ratio 8:1, 6% cement backfill



SF = 1.516	*SF* = 1.514

* The data is not available since the soil body collapses (*SF*< 1), marking failure in the simulation process.

Table 7. Selected slope stability simulation result.

C. Analytical discussion

Numerical simulations of the existing structure confirmed that the combined use of contiguous bored piles (CBP) and ground anchors provided sufficient stability, with the safety factor (*SF*) exceeding the minimum design requirement. The removal of ground anchors led to a marked reduction in *SF*. This highlights the anchor's critical role in reinforcing the structure. The decrease of safety demonstrates that the anchor acts as an additional resisting force against soil movement, thereby increasing the shear strength along the potential failure surface, reduces the risk of slope failure, enhancing the overall safety of the design.

For the proposed slope models (Cases 1 and 2), untreated soil exhibited rapid reductions in *SF* with increasing depth, particularly under surcharge loading where imminent failure was predicted. Cement stabilization significantly improved slope behavior, with higher dosages yielding increased stiffness, cohesion, and friction angle as the literature stated [4], [6]. These enhancements not only raised the *SF* above 1.5 but also shifted critical slip surfaces deeper into the subgrade, reducing the risk of shallow translational failures.

It can be said that slope geometry had a pronounced influence on stability. This agree with Shafna &

Anjana [7] and Abbas [8]. Gentler slopes (1:2, 1:1.5) maintained acceptable *SF* even with lower cement contents, while steeper slopes required higher treatment levels. Under surcharge, cement-treated slopes retained consistent failure arcs and depths, reflecting the robustness of chemical stabilization against real-world traffic loading.

Comparative analysis showed strong agreement between computer-aided simulation and numerical computation, with differences below 3%. Both approaches revealed consistent trends in *SF* and failure mechanisms, confirming the reliability of the modeling framework and the accuracy of the implemented numerical methods.

IV. CONCLUSIONS

This study established the technical feasibility of reusing locally sourced soil waste as stabilized backfill material for reinforced slope construction. By integrating advanced numerical modeling in computer-aided simulation (utilizing both Mohr-Coulomb and Hoek-Brown constitutive models) with numerical computation, the investigation verified that cement stabilization markedly enhances soil strength, cohesion, and overall slope stability. It demonstrated that incorporating 6-12% cement by dry weight substantially improves the factor of safety, maintaining values above 1.5 across most configurations. Furthermore, the strong agreement between numerical computationand simulation outputs underscores the reliability and robustness of both analytical frameworks.

The principal findings indicated that: (*i*) excavated soil waste could be effectively repurposed for slope reinforcement, (*ii*) cement stabilizationmitigated both environmental impact and material expenditure relative to conventional retaining systems, and (*iii*) the proposed slope structure is suitable for the redesign.

Based on the safety factors presented in Fig. 8 and 9, Case 2 was identified as the optimal configuration. All slopes stabilized with cement stabilization satisfied the stability criterion ($SF > 1.5$). However, in Case 1, only the configuration with 12% cement content at 1:1.5 slope ratio achieved adequate safety ($SF = 1.518$). It emerged as sustainable and structurally efficient alternatives to traditional reinforcement methods.

Future research should validate these findings under macro-scale conditions, incorporating the effects of groundwater fluctuations, seismic loading, and long-term hydro-mechanical behavior. Further exploration of alternative stabilizers, such as lime or lime-cement blends along with the inclusion of curing time effects, could yield additional performance optimizations.

ACKNOWLEDGMENTS

This research was conducted in Soil Reinforcement Project with funding from INACON Project under the scheme LBE “Lab Based Education”. The authors also appreciated the technical advising of Prof. Hossein NOWAMOOZ (INSA Rennes).

REFERENCES

[1] F. Lord, “TRANSFORMATION TO SUSTAINABLE AND RESILIENT URBAN FUTURES IN SOUTHEAST ASIA,” ISPRS Ann. Photogramm. Remote Sens. Spat. Inf. Sci., vol. VI-3/W1-2020, pp. 43–50, Nov. 2020, doi: 10.5194/isprs-annals-VI-3-W1-2020-43-2020.

[2] R. F. M. Ameen and M. Mourshed, “Urban environmental challenges in developing countries—A stakeholder perspective,” Habitat Int., vol. 64, pp. 1–10, Jun. 2017, doi: 10.1016/j.habitatint.2017.04.002.

[3] [3] D. N. Little and S. Nair, “Recommended practice for stabilization of subgrade soils and base materials,” 2009.

[4] J. Szendefy, “Impact of the soil-stabilization with lime,” in Proceedings of the 18th international conference on soil mechanics and geotechnical engineering, Paris, 2013, pp. 2601–2604.

[5] A. Ullah, D. Boumezerane, and F. Ahmad, “Subgrade improvement with mixed lime and cement as additives,” Mater. Today Proc., p. S2214785323033734, Jun. 2023, doi: 10.1016/j.matpr.2023.05.706.

[6] V. T.-A. Phan and K. N.-A. Pham, “Improvement in engineering properties of soft-soil using cement and lime additives: A case study of southern Vietnam,” IOP Conf. Ser. Earth Environ. Sci., vol. 143, p. 012039, Apr. 2018, doi: 10.1088/1755-1315/143/1/012039.

[7] P. M. Shafna and T. R. Anjana, “Generalized Three-Dimensional Slope Stability Analysis of Soil Using Plaxis 3D,” in Proceedings of the Indian Geotechnical Conference 2019, vol. 133, S. Patel, C. H. Solanki, K. R. Reddy, and S. K. Shukla, Eds., in Lecture Notes in Civil Engineering, vol. 133. , Singapore: Springer Singapore, 2021, pp. 745–752. doi: 10.1007/978-981-33-6346-5_64.

[8] J. M. Abbas, “Slope stability analysis using numerical method,” J. Appl. Sci., vol. 14, no. 9, pp. 846–59, 2014.

[9] R. K. Sharma, A. Kaur, and A. Kumar, “Slope Stability Analysis by Bishop Analysis Using MATLAB Program Based on Particle Swarm Optimization Technique,” in Proceedings of the 1st International Conference on Sustainable Waste Management through Design, vol. 21, H. Singh, P. Garg, and I. Kaur, Eds., in Lecture Notes in Civil Engineering, vol. 21. , Cham: Springer International Publishing, 2019, pp. 285–293. doi: 10.1007/978-3-030-02707-0_34.

[10] T. V. Salunkhe, S. M. Nawghare, M. B. Nadaf, S. Dutta, and J. N. Mandal, “Stabilization of Fly Ash Slope Using Plastic Recycled Polymer and Finite Element Analysis Using Plaxis 3D”.

[11] B. Tchakalova and P. Ivanov, “Correlation between effective cohesion and plasticity index of clay,” Geol. Balc., vol. 51, no. 3, pp. 45–49, Nov. 2022, doi: 10.52321/GeolBalc.51.3.45.

[12] K. Winn, H. Rahardjo, and S. C. Peng, “Characterization of residual soils in Singapore,” Geotech. Eng., vol. 32, no. 1, pp. 1–14, 2001.

[13] R. Obrzud and A. Truty, “The Hardening Soil Model—A Practical Guidebook Z Soil PC 100701 report, revised 31.01,” Obrzud R Truty A, 2012.

[14] Á. Kézdi and L. Rétháti, Handbook of soil mechanics, vol. 1. Elsevier Amsterdam, 1974.

[15] M. Prat, BISCH, P, MILLARD, A, MESTAT, P, and PIJAUDIER-CALOT, G, La modélisation des ouvrages. in Emploi des éléments finis en génie civil, no. 1. Paris: Hermès, 1995.

[16] M. Rendana, W. M. R. Idris, S. Abdul Rahim, Z. Ali Rahman, and T. Lihan, “Rubber bark dust-zeolite composite improved mechanic strength of soft paddy soil through improved

microstructure," SAINS TANAH - J. Soil Sci. Agroclimatol., vol. 18, no. 2, p. 194, Dec. 2021, doi: 10.20961/stjssa.v18i2.56345.
[17] K. Terzaghi, R. B. Peck, and G. Mesri, Soil mechanics in engineering practice, 3rd ed. New York Chichester Brisbane [etc.]: J. Wiley & sons, 1996.
[18] D. M. Burmister, "Principles and techniques of soil identification," in Proceedings of Annual Highway Research Board Meeting. National Research Council. Washington, DC, 1949, pp. 402–434.
[19] N. Turk and W. R. Dearman, "Estimation Of Friction Properties Of Rocks From Deformation Measurements," presented at the The 27th U.S. Symposium on Rock Mechanics (USRMS), Jun. 1986, p. ARMA-86-0093.
[20] E. Hoek, "Estimating Mohr-Coulomb friction and cohesion values from the Hoek-Brown failure criterion," in International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts, Pergamon, 1990, pp. 227–229.
[21] E. Hoek, P. K. Kaiser, and W. F. Bawden, Support of underground excavations in hard rock, Nachdr. London: Tayler & Francis, 2006.
[22] Z. T. Bienawski, "Rock mass classifications in rock engineering.," 1976.
[23] S. Prakash and H. D. Sharma, Pile foundations in engineering practice, 2. [print.]. in A Wiley-Interscience publication. New York: Wiley, 1990.
[24] R. Kallu and P. Roghanchi, "Correlations between direct and indirect strength test methods," Int. J. Min. Sci. Technol., vol. 25, no. 3, pp. 355–360, May 2015, doi: 10.1016/j.ijmst.2015.03.005.
[25] S. Narimani, S. M. Davarpanah, and B. Vásárhelyi, "Estimation of the Poisson's Ratio of the Rock Mass," Period. Polytech. Civ. Eng., vol. 68, no. 1, pp. 274–288, Oct. 2023, doi: 10.3311/PPci.22689.
[26] G. AASHTO, "Guide for design of pavement structures," Am. Assoc. State Highw. Transp. Off. Wash. DC, vol. 1, 1993.
[27] MPWT, ROAD DESIGN STANDARD - PART 2: PAVEMENT, CAM PW.03.102.99, West Conshohocken, PA, USA., 2003.
[28] D. Perraton, H. Baaj, and A. Carter, "Comparison of Some Pavement Design Methods from a Fatigue Point of View: Effect of Fatigue Properties of Asphalt Materials," Road Mater. Pavement Des., vol. 11, no. 4, pp. 833–861, Jan. 2010, doi: 10.1080/14680629.2010.9690309.
[29] B. M. Das and K. Sobhan, Principles of geotechnical engineering, Ninth edition. Boston, MA: Cengage Learning, 2018.
[30] R. B. J. Brinkgreve and H. L. Bakker, "Non-linear finite element analysis of safety factors," in International conference on computer methods and advances in geomechanics. 7, 1991, pp. 1117–1122.
[31] T. J. Ypma, "Historical Development of the Newton–Raphson Method," SIAM Rev., vol. 37, no. 4, pp. 531–551, Dec. 1995, doi: 10.1137/1037125.

★★★