Failure of Riverbank Protection Structure and Remedial Approach

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ABSTRACT: This paper presents the case study of the collapsed riverbank protection structure along the Pasak river in Saraburi province, Thailand. The site investigation and finite element analysis using PLAXIS 2D results show that the failure occurred in sliding mode due to the natural forces. During the rainy season, water flow from the farmlands to the river by crossing the backfill of the retaining wall. Hence, seepage force was developed in the direction of the flow and induced the stability of the riverbank protection. Furthermore, the rivers and streams continuously scour the banks and undermined the natural slope, which caused the soil erosion in passive zone and resulted in instability. Based on these causes of failure, a new reinforced retaining wall structure using bored pile, geocomposite, and riprap at the front of retaining wall to protect the circular failure mechanism, seepage forces, as well as soil erosion and sedimentation, respectively was designed. The finite element verification on the new retaining wall structure showed that this structure had a sufficient factor of safety against the external and internal slope failure.

KEYWORDS: Riverbank protection structure, Seepage flow, Erosion, Finite element analysis

1. INTRODUCTION

The Paksak River is a river in a central Thailand. It originates in Loei Province and passes through Phetchabun Province and Saraburi Province, until it joins together with the Lopburi River in Lupburi Province before it flows into the Chao Phraya River in southeast of Ayutthaya Province, Thailand. The valley of the Paksak River through the Sao Hai District is the main part of Saraburi Province. However, as the watershed of the river is rather narrow, the amount of water in the river varies seasonally. As a result, the erosion and sedimentation are natural phenomenon and processes, which cause the destruction of tremendous properties and land nearby the river bank. Thus, the Department of Public Works and Town & Country Planning (DPT) under the Ministry of Interior has constructed the riverbank protection (retaining wall) to protect the devastation of the river bank.

The retaining wall construction consists of 400 m long and lies along the Paksak River in Sao Hai District, Saraburi Province. However, approximately 68 m long of the embankment (from the Station 10+022 to 10+090) collapsed at the curvature of the watershed area, where faced to the direction of river flow. Hence, a new reinforced riverbank protection structure was designed and constructed to protect this failure section. The retaining wall structure was constructed by using the anchor system with reinforced concrete structure. The construction completely ended on March 2012.

After 7 months of the construction, on October 2012, the failure of the anchor retaining wall structure caused by the lateral force, which approximately 5.5 m of the retained soil mass moved laterally toward the river side was observed. Therefore, the DPT has designed a new retaining wall structure by using driven piles with reinforced concreted beams. Furthermore, the additional design of riprap structure was applied on the front of the retaining wall in order to protect the attack of erosion on the river bank. After one year of the construction, again the lateral movement occurred along the retaining wall, though the major settlement of the embankment was not observed. However, during the rainy season a large lateral movement occurred and caused extreme settlement of the embankment due to the erosion and sedimentation of the bank and resulted failure of the retaining wall.

Therefore, the improvement and rehabilitation of the retaining wall structure are very much important. Regarding to these issues, the DPT cooperated with the Department of Groundwater Resource has an attempt to explore the geotechnical engineering experts to solve and design the durable and sustainable retaining wall.

2. SITE INVESTIGATION

For riverbank improvement and rehabilitation, first and foremost a feasibility study is needed, which has to be carried out. In order to carry out the different rational riverbank protection designs, the site investigation and the existed retaining wall structures were carefully investigated.

The retaining wall construction consists of 400 m long and locates along the Paksak River in Sao Hai District, Saraburi Province. The collapse of 68 m embankment occurred at the curvature of the watershed area, which probably caused by the high flow velocity of the river (See Figure 1).

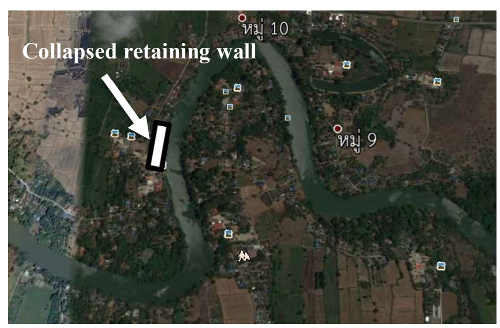
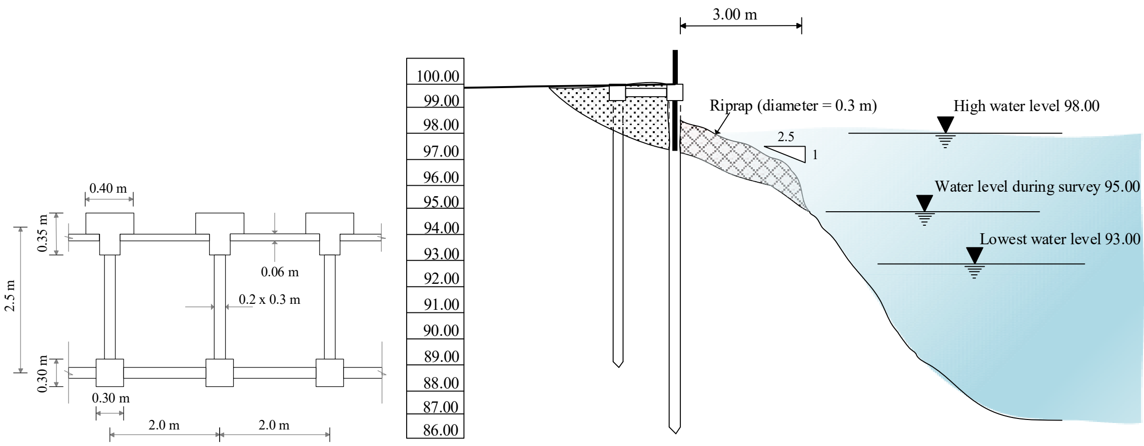


Figure 1 The location of the collapsed retaining wall

The collapsed retaining wall was repaired by the anchor retaining wall structure. However, the structural detailing of this construction method is not available in this study. Then, the new retaining wall structure using driven piles and reinforced concrete beams were constructed and replaced the existed anchor structure. Figure 2 shows the geometry and structural detailing of the retaining wall. The retaining wall structure used a double driven pile system with reinforced concrete beams. The back piles of the retaining wall structure were rectangular piles (0.3x0.3x10 m), while the front piles were T-section (0.35x0.40x14 m). The spaces between the back and the front pile were 2.5 m and the spaces between the T-section piles were 2 m. The size of rectangular reinforced concrete beams was 0.2 m in width and 0.3 m in height, while the thickness of the reinforced concrete retaining wall was 0.06 m. In addition, the utilization of riprap with 0.3 m in diameter was used in front of the retaining wall to protect the erosion problem.



(a) (b)

Figure 2 Details of the driven pile retaining wall structure: (a) plan views, and (b) side view

3. ANALYSIS CAUSES OF FAILURE

Figure 3 obtained during the side investigation evidently confirmed that the massive lateral movement of the embankment occurred in the lateral direction and tiled to the riverside. This can be contributed to the extreme lateral force of the embankment, which exceeded the resistance of the retaining wall structure. As a result, the retaining wall was damaged from the reinforced beams as shown in Figure 4.

Figure 3 Failure surface in plan (lateral moment)

Figure 4 Damages to existing retaining wall

Based on the literature review, the lateral pressure exerted on a retaining wall depends on several factors such as the mechanical and geometrical properties of the backfill, the friction between the backfill and the wall. In addition, the distribution of earth pressures on the wall depends on the wall movement (Benmeddour et al., 2012).

Obviously, in this case study, the soil was at the point of incipient failure by shearing due to the lateral force caused by lateral earth pressure, in which the retained soil mass was allowed to deform laterally and slid the retaining wall outward to the river side. For the rigid retaining wall, the active failure wedge in the backfill in bounded by the wall and the plan with an inclination angle of (45º + ϕ/2) from the horizontal and may result in interference of the development of the active state behind the wall (Fan & Fang, 2010; Rankine, 1857).

The development of the sophisticated computer hardware and advanced numerical methods, allows the geotechnical engineers and researchers studying the behaviour of the earth-retaining structure quickly and consistently. The reliable non-linear finite element program PLAXIS 2D is widely used by geotechnical engineers and researchers to solve earth-retaining structure problems Fan and Fang (2010) and Yu et al., (2015) was used as a tool to analyze the stability of the retaining wall and to diagnose the cause of failure in this study.

The soil profile data was obtained from the boring log near the collapsed retaining wall. It demonstrated that the backfill soil layers were typical loose to dense sandy materials. The parameters used for the backfill soil and retaining wall structures to execute the stability analysis of retaining wall in PLAXIS 2D program are summarized in Figure 5 and Table 1 respectively.

3.1 Modelling of backfill, walls, and interfaces

Soil elements used in this study were six-node triangular isoperimetric elements, with three Gauss points for each element. The Mohr–Coulomb constitutive model using the effective stress analysis was used to model the stress–strain behavior of soils. This model required five parameters, i.e., Yong's modulus (E'), Poisson's ratio (ν'), friction angle (ϕ'), cohesion (c'), and dilatancy angle (ψ'). The dilatancy angle (ψ') is normally used in cohesionless angle of the soil. For a soil material with friction angle greater than 30º, the soil tends to dilate at small strain conditions, where active earth pressure develops. The dilatancy angel (ψ') is approximately equal to ϕ' – 30º (Bolton, 1986) and it is used in this study. Interface element between the wall and the soil backfill was also considered in the analysis. Thin rectangular interface elements, six-node elements, were used between the soils and structure elements (Brinkgreve & BROERE, 2015).

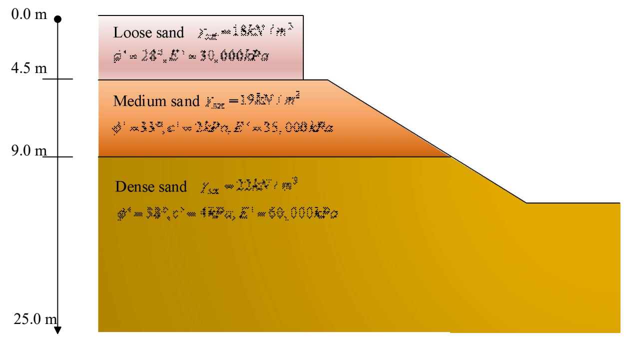


Figure 5 Soil layers and their engineering properties for FE analysis

Table 1 Material properties of driven pile retaining wall structure

Parameter	Front Pile (0.45x0.30 m)	Back Pile (0.30x0.30m)	Reinforced Beam (d = 0.2m, h = 0.3m)
Material model	Elastic	Elastic	Elastic
Young's modulus, E' (kN/m^2)	25.5x10^6	25.5x10^6	2.04x10^8
Area, A (m^2/m)	0.135	0.09	4.99x10^{-3}
Moment of inertia, I (m^4/m)	2.27x10^{-3}	0.338x10^{-3}	1.56x10^{-3}
Poisson's ratio, υ'	0.25	0.25	0.25
Density, γ (kN/m^3)	23.5	23.5	23.5

3.2 Finite element model

The lowest water level of 7 m at the front of the retaining wall measured from the surface of the embankment (at water label = 93 m in Figure 2), which was considered as the worst case was, used in the finite element (FE) analysis. The simulation of FE analysis and its result depicted in Figure 6 show that the factor of safety (FS) equal to 1.613 and greater than the required design FS = 1.5, which commonly used by the geotechnical engineers and researchers for FS against sliding (Budhu, 2008). This demonstrates that the lowest water level simulation is not a major phenomenon cause of retaining wall failure.

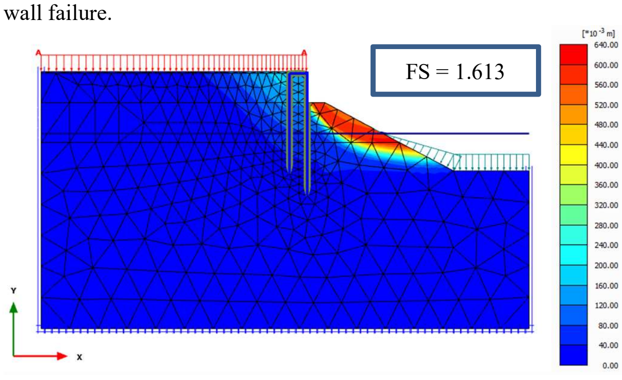


Figure 6 The simulation of FE analysis with the lowest water level at the front of the retaining wall

Therefore, the in-situ reinvestigation has been reconducted by interviewing residents living nearby the riverbank, and suffering from the collapse of retaining wall in order to collate more information. It was found that there were farm lands behind the failure retaining wall. The retaining wall collapsed during the rainy season, in which the strong water flow from the farm to the river by crossing the embankment. Based on geotechnical theory, water can flow between the interconnected voids of soil particles sizes.

In other words, the viscous drag of water flowing through a soil imposes a seepage force on the soil in the direction of flow, which increases in the interganular pressure. This phenomenon has caused instability and failure of many geotechnical structure including roads, bridges, dams, and excavation (Budhu, 2008; Mizal-Azzmi, Mohd-Noor, & Jamaludin, 2011). Furthermore, as the failure retaining wall located on the curvature of the watershed, the strong force of river flow may cause the loss of soil mass at the front of the retaining wall (passive zone). Thus, again the elevation survey has been carried out and it was revealed that the soil mass in the passive zone has been decreased. The soil mass at the back of the wall is causing failure, while the soil mass at the front of the wall is resisting failure. Hence, the sliding failure of the retaining wall caused by the insufficient base friction or lack of passive resistance in front of the wall (Abdullahi, 2009; Budhu, 2008; Rankine, 1857).

Based on these observations, two new cases of FE analyses were simulated. First case, the water level of 0.5 m below the embankment surface simulating the seepage force in the back of the retaining wall was considered, while the lowest of the water level (-7 m) at the front of the retaining wall was keep constant. In addition, the decrease of the soil mass at the front of the retaining wall was considered in the second case. Figures 7 and 8 show the simulations and their results for the first and second case respectively. The FE simulation analysis result for the first case shows that the FS = 1.102 marginally lower than the required design FS. This evidently proof that the water flow (seepage force occurred in the backfill) can reduce the stability of the embankment. Furthermore, the worst factor of safety (FS < 1.0) obtained from the FE simulation analysis for the second case demonstrates that the loss of soil mass in passive zone is significantly induce the stability of the retaining wall structure.

From the site investigation, and the interview with residents nearby the riverbank as well as the FE analysis results, it can be concluded that the water flow (seepage force) occurred in the backfill and the loss of soil mass in passive zone caused by the erosion and sedimentation from the river flow are the main major causes of retaining wall failure.

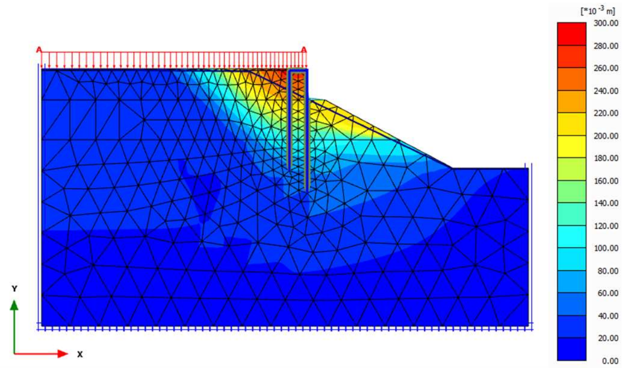


Figure 7 Case 1: simulation of seepage force in the backfill without the loss of mass at the front of retaining wall

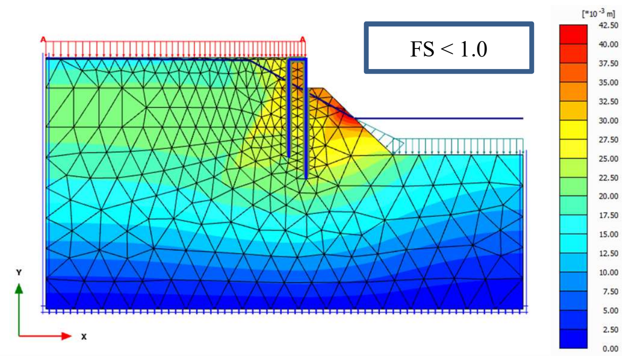


Figure 8 Case 2: simulation of seepage force in the backfill and the loss of mass at the front of retaining wall

4. REMEDIAL APPROACH

4.1 Design concept

In order to repair and rehabilitation of the collapsed retaining wall structure, the causes of its failure, which is the fundamental factor lead to instability must be comprehensively studied. Based on the site investigation and the FE analysis method on the collapsed retaining wall, the failure of retaining wall caused by natural forces. As the retaining wall collapsed during the rainy season, rainfall is one of the causes of failure. Long periods of rainfall saturate, soften, and erode soils. The water flow enters into existing cracks and may weaken underlying backfill soil layers. Furthermore, it produces more seepage forces leading to failure the embankment. Another reason is that rivers and streams continuously scour the banks and undermining their natural slopes. Erosion changes the geometry of the slope in passive zone, ultimately resulting in slope failure.

Therefore, a new stability of earth-retaining structure has been designed. It must guarantee that a geotechnical system will not fail or collapsed under any conceivable loading condition. The new structure is designed based on three main approaches as follows:

For the first approach, a retaining wall must have an adequate factor of safety to prevent excessive translation, rotation, bearing capacity failure, deep-seated failure, and seepage-induced instability. Hence, the new pile system has been proposed. The pile length must be designed to produce enough capacity to prevent the circular failure mechanism. Based on the circular failure or slip mechanism in Figures 7 and 8, the new designed length of piles is approximately 80 m. In addition, the spaces of bored piles have been extended in order to increase the pile work efficiency in overlapping zones and active wedges of the backfill, hence the circular failure area has been reduced (Khari, Kassim, & Adnan, 2013; Lee, Hull, & Poulos, 1995). Due to the very dense sandy soil foundation, the bored pile method is used instead of driven pile. The bored pile with diameter of 60 cm is used for the front and back of the pile retaining wall. The spaces of the longitudinal and cross section are 5.5 m and 1.2 m, respectively. The reinforced concrete walls are constructed in the front and the back of bored pile heads, while the steel H-beam structure is used between the piles as bracing beams.

In recent year, several researchers have extensively studied on the geocomposite drainage under seepage condition in earth-retaining structure. They reported that the seepage through the earth-retaining structure due to the rainfall causes the increase in the lateral stress and reduction in the effective stress, stiffness and strength of the backfill, hence the reduction in the factor of safety against external and internal failure. On the other hand, the geocomposite drainage reduces the water pressure in the reinforced zone, thus the improvement of the stability of the retaining wall (Chinkulkijniwat et al., 2017; Udomchai et al., 2012). Therefore, a design of drainage system using geocomposite in the embankment was considered as the second approach for reducing the impact of water flow.

For the last approach, the utilization of riprap has been designed and installed on the crest and the toe of the slope at the front of the retaining wall in order to protect the erosion and sedimentation problem. The design procedure was carried out according to the previous technical paper (Galay, Yaremko, & Quazi, 1987; Maynord, Ruff, & Abt, 1989), which is based on the local average channel velocity and local depth of the river. The riprap design procedure according to the DPT's regulation (DPT, 2006) can be expressed as follows:

Required design diameter of riprap

$$d = \frac{CV^2}{g(s-1)\Omega} \tag{1}$$

where:

V = Velocity of the river flow
C = Coefficient of the river flow
C = 0.3 for low turbulent flow, and
C = 0.7 for high turbulent flow
g = gravity acceleration, (g = 9.81)
s = specific gravity of riprap, and
Ω = side slope correction factor

Velocity of the river flow (V) can be calculated by:

$$V = \frac{\text{Discharge}}{\text{Area}} = \frac{1500 m^3/s}{350 m^2} = 4.3 m/s \tag{2}$$

Side slope correction factor (Ω) can be calculated by:

$$\Omega = \left[1 - \frac{\sin^2\alpha}{\sin^2\phi}\right]^{1/2} = 0.628 \tag{3}$$

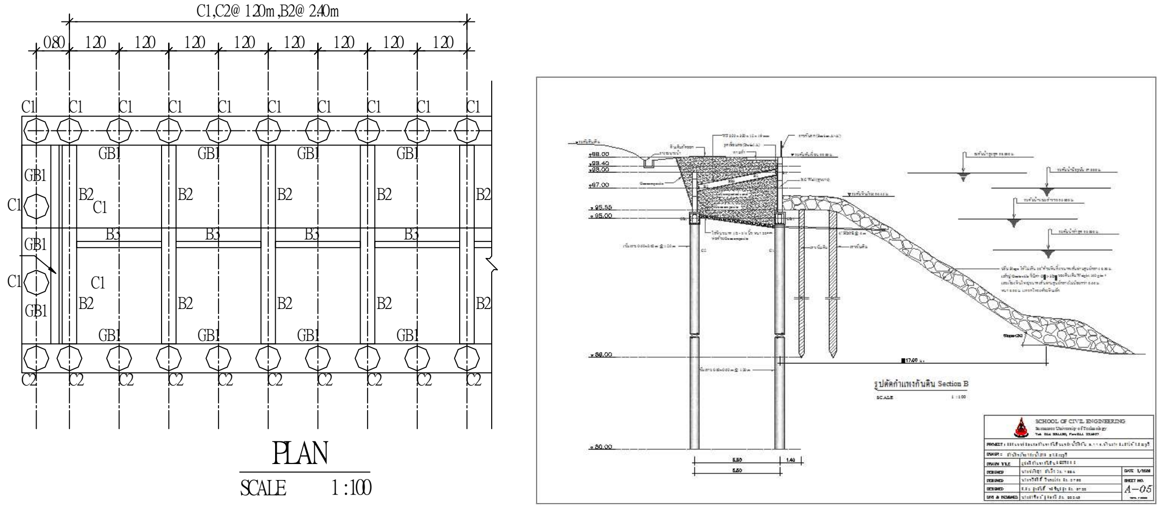
Where:

Friction angle of slope $\phi \geq 40^{\circ}$
Angle of slope at the front of the retaining wall $\alpha \leq 30^{\circ}$

Hence, the required design diameter of riprap is

$$d = \frac{0.3 \times 4.3^2}{9.81 \times (2.65-1) \times 0.628} = 0.55 m$$

Finally, the required 60 cm diameter of riprap was installed with the thickness of 90 cm and 180 cm at the crest and the toe of the slop at the front of the retaining wall, respectively. Furthermore, the textile layer prepared as a filter was installed beneath the riprap layer. Figures 9 and 10 show the structural detailing and the schematic drawing of the retaining wall system, respectively.



(a) (b)

Figure 9 Structural detailing of retaining wall: (a) plan view and (b) side view

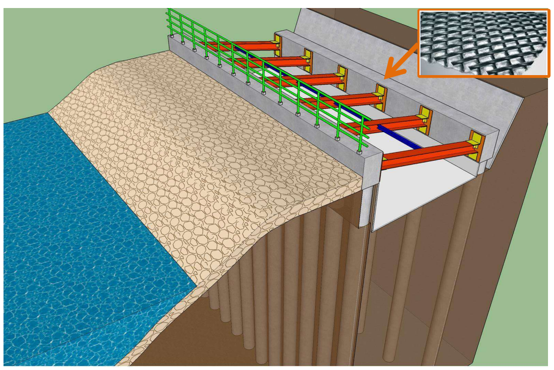


Figure 10 Schematic drawing of the retaining wall system

4.2 Finite element verification

The stability of the new reinforced retaining wall system has been verified by FE analysis method using PLAXIS 2D program. The parameters used for the backfill soil are depicted in Figure 5 and the parameters for the bored pile retaining wall structure are summarized in Table 2.

The effect of water flow in the back and in the front of the retaining wall is significant impact and was considered in the simulation of FE analysis. In addition, due to the variation of water level in the river seasonally, the reservoir nearby the retaining wall can be subjected to rapid drawdown phenomenon (Budhu, 2008). In this case, the lateral water force is removed and the excess porewater pressure does not have enough time to dissipate (Figure 11). The net effect is that the slope can fail under undrained condition. If the water level in the reservoir remains at low levels and failure did not occur under undrained condition, seepage of ground water would occur and the additional seepage forces could provoke failure. Therefore, a case study with the lowest water level at – 7 m obtained from the groundwater station (see Figure 2, water label 93 m) and a case study without water in the reservoir (water level at the bed of the river, water label 87 m) were investigated in FE analysis.

Figure 12 shows the simulation results of FE analyses for both cases study. The FE analysis results show that the FS = 1.98 and 1.79 for case studies with water and without water level in the reservoir, respectively (see Figures 12a and b). The factor of safety for both cases study is greater than the required design factor of safety (FS > 1.50), which demonstrates that the retaining wall design is stable.

Table 2 Material properties of bored pile retaining wall structure

Parameter	**Bored Pile (ϕ = 0.6m)**	**Strut**
Material model	Elastic	Elastic
Young's modulus, E' (kN/m^2)	25.5x10^6	2.04x10^8
Area, A (m^2/m)	0.235	4.99x10^{-3}
Moment of inertia, I (m^4/m)	5.30x10^{-3}	85x10^{-6}
Poisson's ratio, υ'	0.25	0.3
Density, γ (kN/m^3)	23.5	78.5

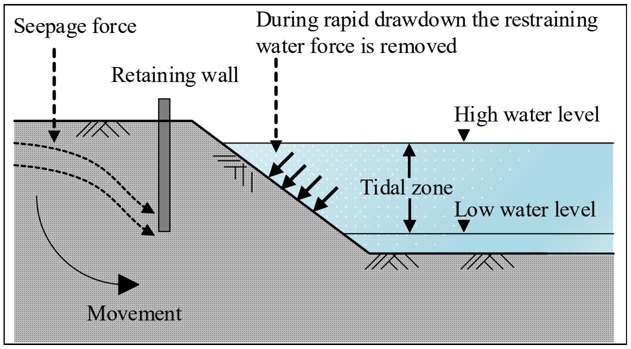
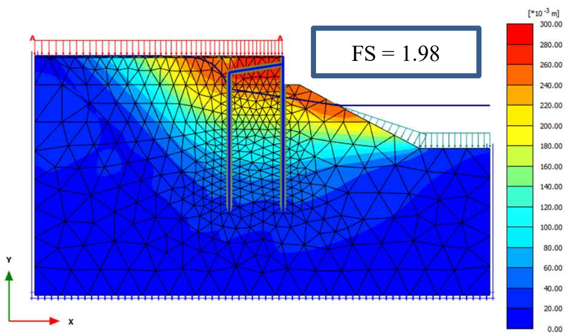
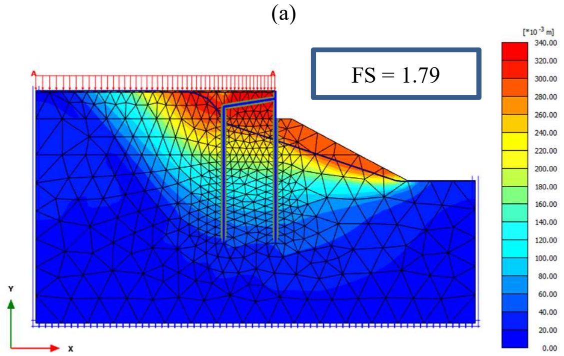


Figure 11 Rapid drawdown phenomenon



(a)



(b)

Figure 12 FE analysis of bored pile retaining wall: (a) a case study with water level, and (b) a case study without water level

5. CONCLUSION

This research paper presents a case study of the collapsed earth-retaining structure and the remedial approach. The retaining wall has been constructed to protect the riverbank along the Paksak river in Suraburi province, Thailand. However, a part of the retaining wall was collapsed during the rainy season. The retaining wall remedy has been undertaken twice, one using the anchor retaining wall structure and other using pile driven retaining wall structure, but these two construction methods were defective to protect the failure of the retaining wall.

A new feasibility study on the improvement and rehabilitation of the collapsed retaining wall, hence was deliberated. First, the site investigation and numerical method using non-linear finite element analysis program PLAXIS 2D were investigated the causes of failure. The remedial approaches were then proposed to improve the stability of the retaining wall.

Based on the site investigation and the FE analysis method on the collapsed retaining wall, the failure of retaining wall caused by natural forces. Long periods of rainfall saturate, soften, and erode soils. The water flow enters into the permeable backfill soil layers and directs to the river. This can develop seepage force leading to failure the embankment. Another reason is that rivers and streams continuously scour the banks and undermine the natural slope at the front of the retaining wall. Erosion changes the geometry of the slope in passive zone, which reduces the resistance of passive earth pressure and ultimately resulting in slope failure.

Therefore, three fundamental approaches have been proposed for the new reinforced retaining wall structure. Changing a pile construction method is designed for the first approach.

The bored pile system with steel bracing is constructed as a new retaining structure. The length and the spacing of piles are extended in order to increase the work effectiveness of the pile led to reduce the circular failure mechanism of the backfill. The geocomposite is also installed under the backfill materials as a drainage to defend the seepage force problem. The utilization of the riprap on the crest and the toe of the slope at the front of the retaining wall is applied to protect the erosion and the sedimentation problem, which is the main factor reduction of the soil mass in the passive zone.

The finite element verification has been conducted to check the stability of the new retaining wall. The rapid drawdown phenomenon, a well-known factor that can induce the stability of the earth-retaining structures, is used in the simulation of finite element analysis. The finite element analysis results confirm that the new retaining wall structure is very stable, i.e., the factor of safety is greater than the allowable value.

6. REFFERENCES

Abdullahi, M. a. M. (2009). Evaluation of Causes of Retaining Wall Failure. *Leonardo Electronic Journal of Practices and Technologies*(14), 11-18.

Benmeddour, D., Mellas, M., Frank, R., & Mabrouki, A. (2012). Numerical study of passive and active earth pressures of sands. *Computers and Geotechnics, 40*, 34-44.

Bolton, M. (1986). The strength and dilatancy of sands. *Geotechnique, 36*(1), 65-78.

Brinkgreve, R., & BROERE, W. (2015). PLAXIS 2D Reference Manual 2015. *Delft, Netherlands2010*.

Budhu, M. (2008). *SOIL MECHANICS AND FOUNDATIONS, (With CD)*: John Wiley & Sons.

Chinkulkijniwat, A., Horpibulsuk, S., Van, D. B., Udomchai, A., Goodary, R., & Arulrajah, A. (2017). Influential factors affecting drainage design considerations for mechanical stabilised earth walls using geocomposites. *Geosynthetics International, 24*(3), 224-241

DPT. (2006). Dam protection design, Department of Public Works and Town & Country Planning, Thailand.

Fan, C.-C., & Fang, Y.-S. (2010). Numerical solution of active earth pressures on rigid retaining walls built near rock faces. *Computers and Geotechnics, 37*(7), 1023-1029.

Galay, V., Yaremko, E., & Quazi, M. (1987). River bed scour and construction of stone riprap protection. *Sediment Transfer in Gravel-Bed Rivers. John Wiley & Sons New York. 1987. p 353-379.*

Khari, M., Kassim, K. A., & Adnan, A. (2013). An Experimental Study on Pile Spacing Effects under Lateral Loading in Sand. *The Scientific World Journal.*

Lee, C., Hull, T., & Poulos, H. (1995). Simplified pile-slope stability analysis. *Computers and Geotechnics, 17*(1), 1-16.

Maynord, S. T., Ruff, J. F., & Abt, S. R. (1989). Riprap Design. *Journal of Hydraulic Engineering, 115*(7), 937-949.

Mizal-Azzmi, N., Mohd-Noor, N., & Jamaludin, N. (2011). Geotechnical Approaches for Slope Stabilization in Residential Area. *Procedia Engineering, 20*, 474-482.

Rankine, W. M. (1857). On the stability of loose earth. *Philosophical transactions of the Royal Society of London, 147*, 9-27.

Udomchai, A., Chinkulkijniwat, A., & Horpibulsuk, S. (2012). Physical model tests on Mechanically stabilized earth walls with geocomposite drainage under seepage condition. *Geosynthetics Asia (2012) Proceeeding of the 5th Asian Regio-nal Conference on Geosynthetics, Bangkok, Thailand*, pp.613-616.

Yu, Y., Damians, I. P., & Bathurst, R. J. (2015). Influence of choice of FLAC and PLAXIS interface models on reinforced soil–structure interactions. *Computers and Geotechnics, 65*, 164-174.