Geomechanics and Engineering, *Vol. 21, No. 5 (2020) 401-411*
DOI: https://doi.org/10.12989/gae.2020.21.5.401

Numerical study on the optimal position of a pile for stabilization purpose of a slope

Khalifa Boulfoul*[1], Farid Hammoud[1] and Khelifa Abbeche[2]

[1]*Department of Civil Engineering, Faculty of Science, Mustapha Ben Boulaid, Batna2 University, Batna, Algeria*
[2]*Research Laboratory of Applied Hydraulics RLAHYA, Department of Civil Engineering, Faculty of Science, Mustapha Ben Boulaid, Batna2 University, Batna, Algeria*

(Received July 10, 2019, Revised March 7, 2020, Accepted April 3, 2020)

Abstract. The paper describes the influence of pile reinforcement on the stability of the slope behaviour, and the exploitation of the results of in situ measurements will be conducted. In the second part, a 2D numerical modelling will be conducted by using the finite element code PLAXIS2D; in order to validate the proposed modelling approach by comparing the numerical results with the measurements results carried out on the slides studied; to study the effect of positioning of piles as a function of the shear parameters of the supported soil on the behaviour of the soil. For various shear strength of the soil a row of pile position is found, at which the piles offer the maximum contribution to slope stability. The position of piles is found to influence the safety factor in granular soil whereas it shows a slight influence on the safety factor in coherent soil. The results also indicate that the ideal position for such stabilizing piles is in the middle height of the slope. Comparison of results of present study with literature from publication: indicated that to reach the maximum stability of slope, the pile must be installed with L_x/L ratio (0.37 to 0.62) and the inclination must be between 30° to 60°. Even, after a certain length of the pile, the increasing will be useless. The application of the present approach to such a problem is located at the section of PK 210+480 to 210+800 of the Algerian East-West Highway.

Keywords: landslide; reinforcement; numerical model; shear strength reduction finite element method; pile; safety factor

1. Introduction

The rapid development of road network becomes the basis for economic development, in Algeria. This development based on the construction of infrastructure such as the grand highway East-West axis from Tunisian border (El Kala) to the Moroccan border (Maghnia) where passing near major cities in the north such: Annaba, Skikda, Constantine... over a distance of 1216 km. During the realization of this highway, they face difficulties; especially slipping that requires some form of study for a viable project.

Stability can be an important problem during the construction of geotechnical features for surface transportation facilities. Several methods used in slope stability such as piles, geotextiles, and nails. Stabilizing slopes with piles is becoming more common. The analysis of slopes stabilized with piles can be performed using uncoupled and coupled approach. The uncoupled analysis is the most prevalent method that have been used in stabilizing slopes, in which slope stability and the pile response are considered separately, was effected by evaluating the load transfer of passive pile groups subjected to lateral soil movements in slope (Jeong *et al.* 2003, Nian *et al.* 2008, Munawir *et al.* 2013); however, coupled analysis approach considers both slope stability and pile response in simultaneous way. For better understanding the coupled approach for pile reinforced slopes numerous numerical research studies have been recently accomplished (Won *et al.* 2005, Li *et al.* 2011, Güllü 2013, Wu *et al.* 2014, Xu *et al.* 2018, Jiang *et al.* 2018, Tran *et al.* 2019, Fenu *et al.* 2018).

The coupled analysis is performed using a 2D finite element method to analyze the efficiency of stabilizing the existing slope with piles and to study the effect of different parameters location, inclination and length on the factor of safety of slope. Our results are compared with the literature results.

The studied landslide occurred on a stretch of the East-West highway, from PK210 + 480 to PK211 + 260, near the town of Didouche Mourad, Province of Constantine (North-East of Algeria) (Fig. 1), where a specific study was carried out at the level of risk areas according to the information gathered from the maps of the potentially unstable area. For a specific study, we take into consideration the peripheral configuration. In a state of limiting equilibrium of the block, the stability of the slope must be performed considering this block, thus the residual resistance of the potential slip is taken into account for the calculation of stability. A study is carried out to illustrate the effect of piles on slope stability. First we present a summary description about the geological, geomorphological context and the hydraulic regime of the study site, also the

*Corresponding author, Ph.D. Student
E-mail: kh.boulfoul@univ-batna2.dz

http://www.techno-press.org/?journal=gae&subpage=7 ISSN: 2005-307X (Print), 2092-6219 (Online)

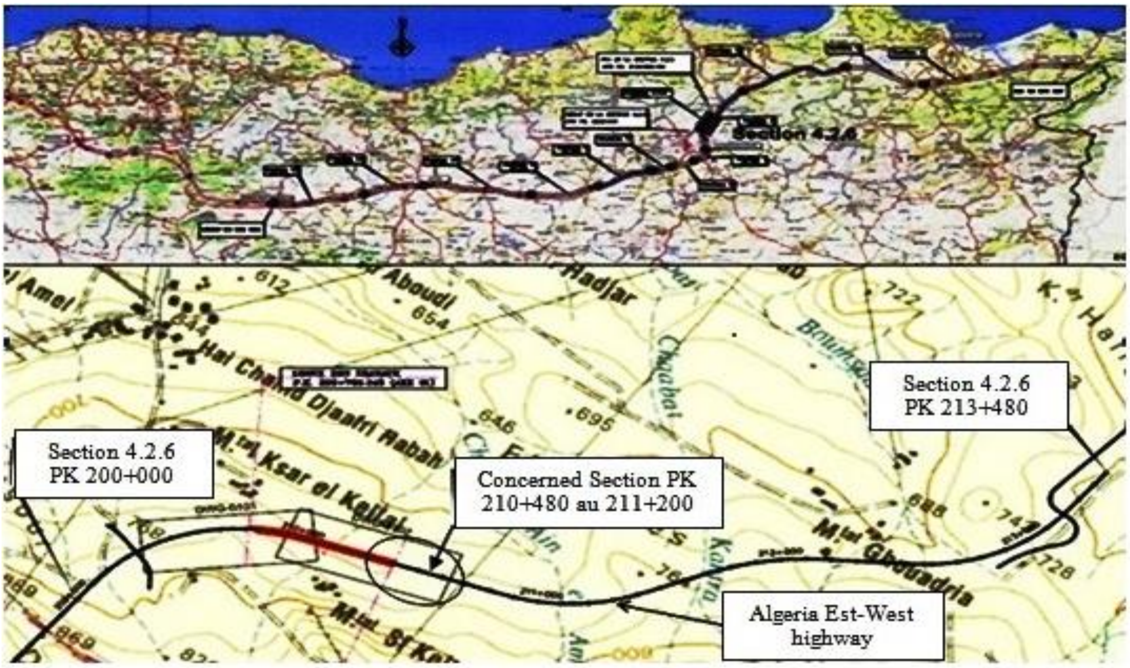


Fig. 1 Location map of the landslide

(a)

(b)

Fig. 2 Crack on the of the bank and expansion joint of OH05

geotechnical characteristics of this block.

Then, the conditions used for the stability calculations using the 2D Plaxis software and their implementation is then explained. Also, a parametric study was carried out on the position and the length of the pile as well as the inclination and then the results are analyzed obtained.

2. Description of events

The first signs of an unstable landslide were observed on May 17, 2009, when the embankment height was 2.5 m. the crack appeared at the top of the embankment along PK 210 + 450 at PK 210+500 over a distance of 50 m.The cracks were discovered for the first time with a thickness of 3 mm (Fig. 2(a)), then the measurement was made. In addition, a landslide existed in the deposition zone (discharge side).

An inclinometer installation was performed to observe cracks in the embankment.On May 28, an opening of the expansion joint of the hydraulic structure (culvert type) (Fig. 2(b)) located at PK 211+035 is discovered. The expansion joint was below the central reserve, so it was able to start measuring the opening width of the cracks. On June 24, cracks were discovered on the body of the hydraulic structure and on the surface of the upper face of the embankment. These last cracks appeared to extend from the embankment exit (west side) to the embankment inlet (east side). The sliding line passes in the oblique direction of the road.Consequently,and based on these results, inclinometers and piezometers were installed at the entrance and exit of the road and it was started to follow with these devices.

3. A brief description of the site

3.1 Geological and geomorphological presentation

This part of the highway (PK 209 to PK 213) is from the

Table 1 Mechanical properties of the soil deposits

Layers	Effective parameter		Non-rained parameter		Weight density γ
	C' (kpa)	Ø' (Deg)	Cu (Kpa)	Øu (Deg)	(kN/m^3)
Clay	14.5	18.5	36.4	10.6	20.9
Argillite	23.6	17.7	57.7	13	22.4
Marl	17.6	21	95	14.1	21.8
Altered marel	9.7	19.2	69	14.5	21
Clayely marl	15.5	18.1	53.2	12.2	20.9

Fig. 3 The section in question

continental basin of Constantine. Generally, it composed of mio Pliocene of continental origin. Also, we can note the presence of Tellian units (marly limestones and marl) and numidian unit (sandstone and clay). Moreover, in section (PK 209 to PK 213) we found some area may designate warning signs of an unstable slope; these movements occupied a large area. Such that: the area PK210 + 400 to PK211+260. The geological formations of this section consist of Cretaceous rock bases, alternating marl - clay - Eocene and Miocene sandstones, and Plio - Quaternary deposits.

There are different types of soil:

Recent alluvium, clayey-silty and gravelly, Colluvium, clay and marl residual surface (Quaternary), Coarse conglomerates with red silty matrix and weak soils, Sandstone and friable sands (Miocene), Marly clays, altered soils (Miocene), marls, marl-limestones, clays and

sandstones, decompressed, black, alternating marls and limestones (Cretaceous to Eocene), Compact gray marl (Cretaceous), Solid limestone of Jebel Kellal (Cretaceous), Nevertheless, layers of this part of road are mostly friable.

3.2 Geotechnical recognition

The geotechnical campaign based on core holes and exploratory wells for identifying purpose of the characteristics of soil on PK209 to PK213+480, whereat it was found: A surface layer, clay layer, marl layer, altered marl layer and Clayey marl layer. Mechanical properties of the soil deposits are tabulated in Table 1.

3.3 The determination of the slip block

3.3.1 Determination of horizontal extent

The horizontal extent of the sliding block was determined on the basis of our observation of ground fissure and recognition surveys, as shown in the following plan.

3.3.2 Determination of horizontal extent

The sliding area was determined using inclinometer measurements and core boring. Likewise, aerial photos have been used, and depicted in the slope deformation prospecting of Section PK210+480 to PK211+200 as preliminary studies. Consequently, it was found that the existence of medium-sized slip blocks is hollow in the middle of PK210+500 to PK211+000. In addition to the surface investigations, two soundings are performed for observing the geological characteristics of the area in question. The observational results on the cores found that this area contains various layers of different natures, such as marls, clay marl, and clays.

4. Cause of landslide

Preliminary Geotechnical and geological studies consider that this section was stable; however, during earthworks and backfilling, some cracking are appeared. Various additional investigations are conducted following the occurrence of cracks to identify their causes. The results show that these cracks are caused by the landslide whose causes could be those listed below:

4.1 Influence of the water table

The water table is relatively high according to the preliminary investigation. Also there is movement of water down the steepest slope (Fig. 3).

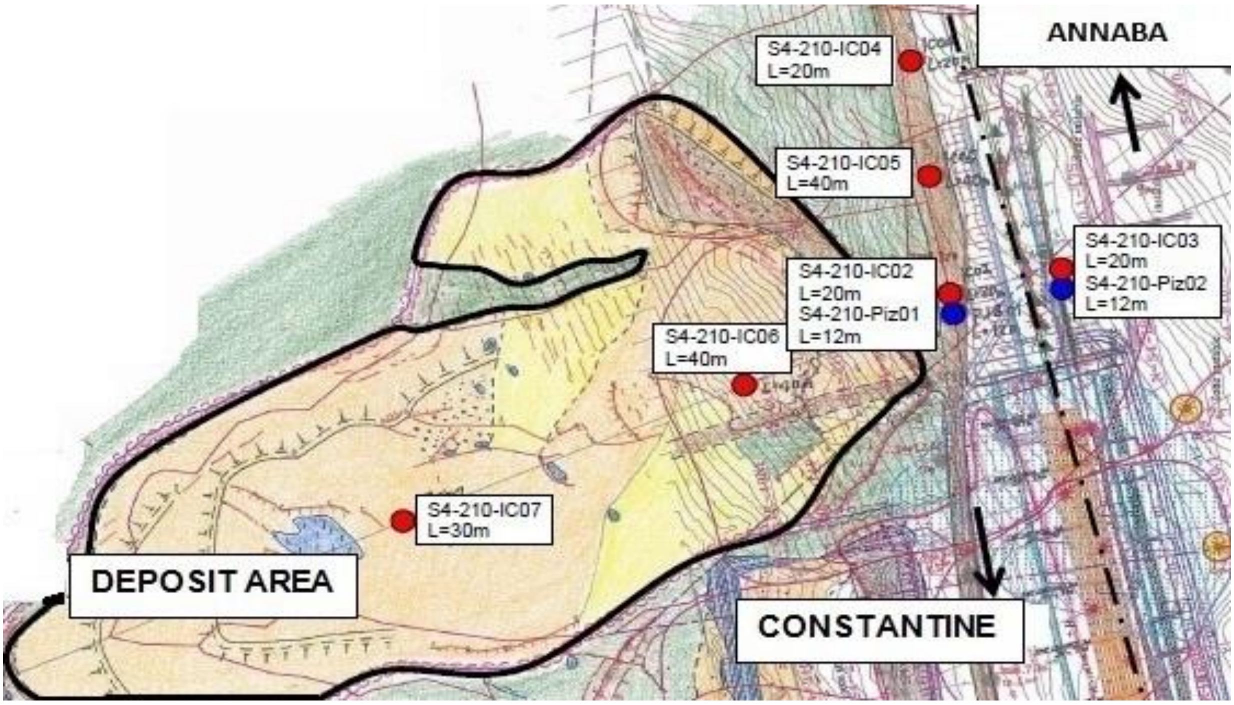


Fig. 4 Deposit area

4.2 Factor of the landslide

The zone is of unstable origin where the landslide block was not important but active. Therefore, the water level was fed at the end of the latter. A deformation was observed between June and August 2008 after the usage of the disposal area (Fig. 4) which is located at outlet of the road. The cliff of the upper part is 80 m from the axis. Consequently, the geographical location of this area is unstable.

4.3 Reason of the activation

The upper part of the hypothetical sliding mass is not stable so the landslide is occurred due to the mechanical destabilization caused by backfilling operation along the Highway.

5. Additional investigation

5.1 Geotechnical investigation

The results of the additional investigations and laboratory tests are carried out in section PK210+480 to 211+ 260 for confirmation purpose of the physical and mechanical parameters, the different layers of this zone are summarily tabulated hereunder in Table 2.

5.2 Water table level and the depth of the failure surface

The studied part is located in a depression which forms a natural drainage zone (for example location of piezometer Number 01 in PK210+470: 210.1-PIZ01). Therefore, the highest level of the water table is considered at the

Table 2 Mechanical and geotechnical parameters of soil layers

Parameter	Backfilling	Clay		Clay marly	
Weight density γ (kN/m^3)	21	20.9		20.9	
Young E density (kN/m^2)	$4E4$	$2E3$		$1.9E4$	
Poisson's ratio υ	0.25	0.25		0.25	
Friction angle φ', φ_u (deg)	30	18.5	10.6	18.1	12.2
Cohesion C', Cu (kPa)	5	14.5	36.4	15.5	53.2

Table 3 the water table levels

Drilling Operations	Water Level (m)
210.1-PIZ01	0.3
210.1-PIZ03	1
210.1-PIZ05	1
210.1-PIZ06	0.5

Table 4 The depths of the slip surface

N°	Number Serial	Location	Depth of the landslide surface
1	S4-210-IC11	PK211+060_Left	Below the 19 m ground surface level
2	S4-210-IC12	PK211+140_Right	Below the 19 m ground surface level
3	S4-210-IC13	PK210+880_L	Below the 6 m ground surface level
4	S4-210-IC16	PK211+020_L	Below the 23.5 m ground surface level
5	S4-210-IC04	PK210+660_L	Below the 12 m ground surface level
6	S4-210-IC02	PK210+480_L	Below the 13.5 m ground surface level

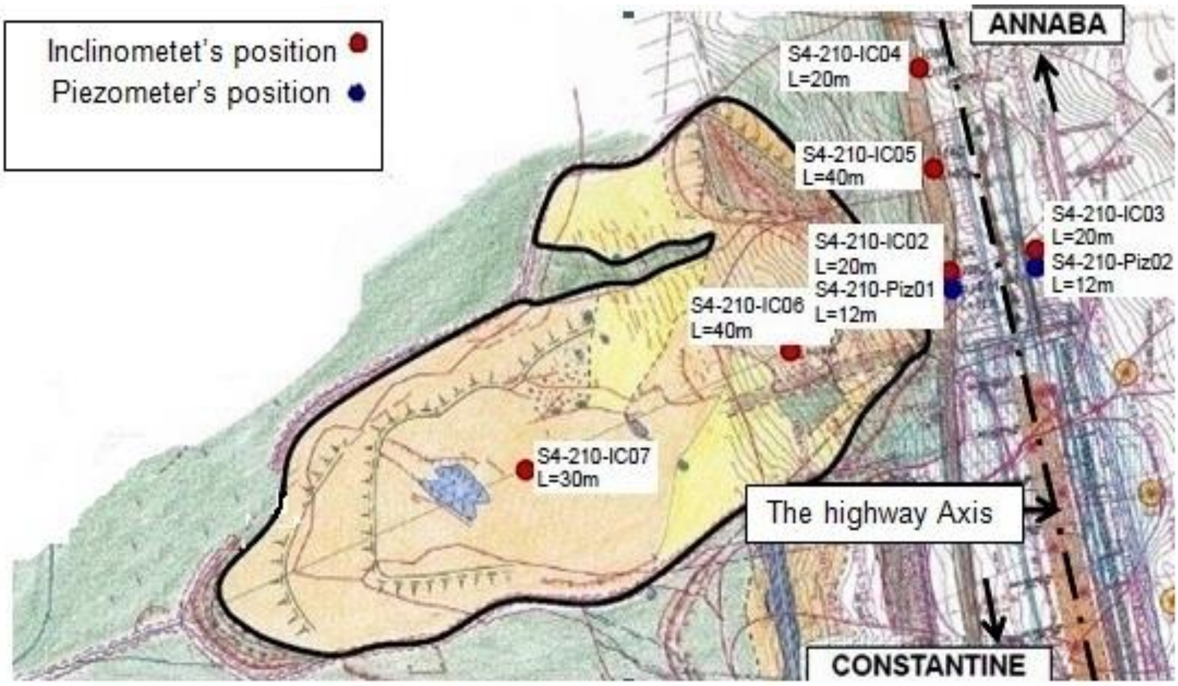


Fig. 5 Plan of the investigation point and the piezometer's positions

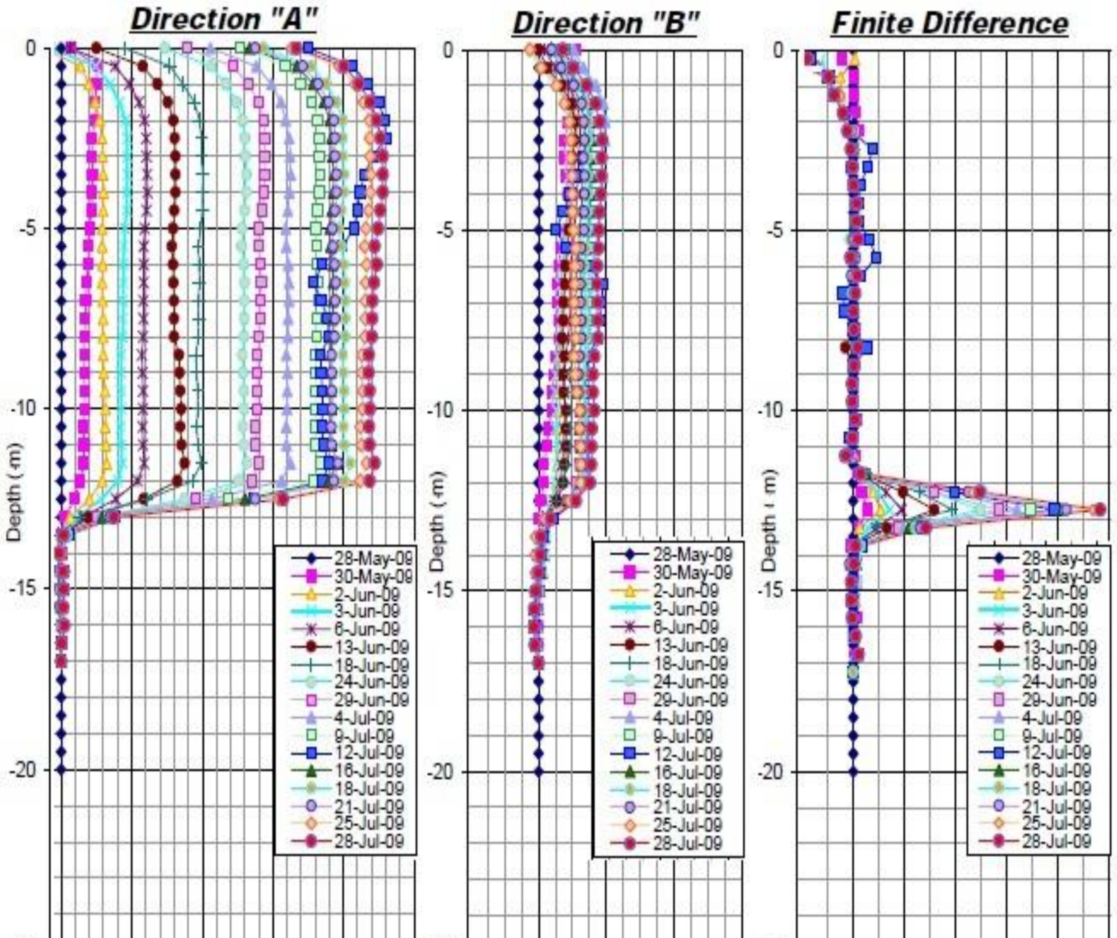


Fig. 6 Results of the measures corresponding to the Inclinometer (S4-210+480-IC02)

ground surface. The groundwater was encountered in the following drilling operations (vide the Table 3 above).

Clinometric results: Fig. 5 shows the implementation plan of position of clinometric and peizometric tubes set up in the drilling operations.

The results of the observations of different inclinometers have shown that near the OH 3 Structure at PK210+500 (210S4-210-IC02: in Section 4 to PK210, is the number on Inclinometer 02), the landslide is occurring on a depth of 13.5 m below the level of the natural ground. The table below (Table 4) summarizes the depths of the slip surface of the various inclinometers located on a linear of 800 m in inlet and in outlet in according to the highway axis.

According to the periodic control of the inclinometer during two month (May 28 to July 28th), we have noticed that the landslide was stabilized in July 28th with a displacement. up to 89,9 mm and depth equal to 13.5 m below ground surface as shown in Fig. 6.

It should be it highlighted that the results of the inclinometer surveys, that correspond to S4-210+480-IC02 compared with the numerical results, have been obtained by the PLAXIS finite element code. The horizontal displacements according to the depth of these inclinometers are given in Fig. 6.

6. Choice of reinforcement

The choice of pile to stabilize a slope of Didouche Mourad constrained by cost and the project schedule. In this study, Pile wall is the solution that was adopted, consisted of piles with diameter D=1.2 m and spacing of (2-3)*D. Also, the coupled treatment of groundwater seepage with vertical drain trenches of depth equal to 5 m to reduce hydrostatic pressures. This drainage is applied on a section that affected by the slip.

7. Stability calculation

7.1 Numerical modeling of the landslide

We used a cross-section before the stability analysis as shown in (Fig. 7), the most unfavorable profile: PK210+478), where the height of embankment reached the max, from PK210+400 to PK210+800.

The model with height equal to 80 m and width equal to 410 m; it consists of two layers (Table 2).

Layer 01: Backfilling, Layer 02: Clayey marl.

We adopt the backfill materials with mechanical properties: C' = 5 Kpa and φ' =30°, and for the backfilling foundation are φ '=8°, and the value of the most appropriate cohesion C'=1 kPa.

The soil behaviour is supposed to behave as elastic perfectly-plastic constitutive relation based on the Mohr

Coulomb criteria. The analysis was then performed under undrained conditions. For the security factors, we based on "Eurocode 7 Technical Guideline, Stabilization of landslides, February 1998":

• 1.25 for the static analysis in effective stresses.
• 1.4 for the static analysis in total stresses.

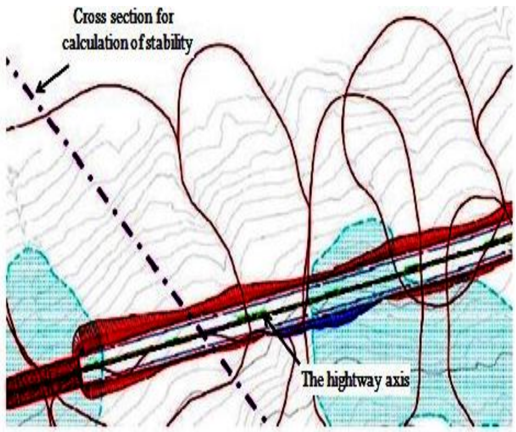


Fig. 7 Extent of the sliding mass PK 210+480

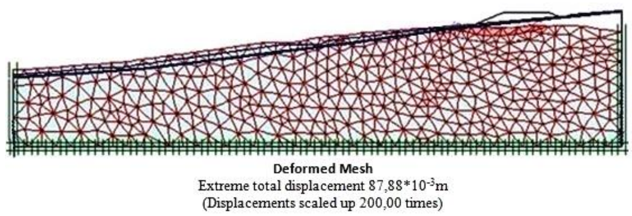


Fig. 8 Mesh of the slope

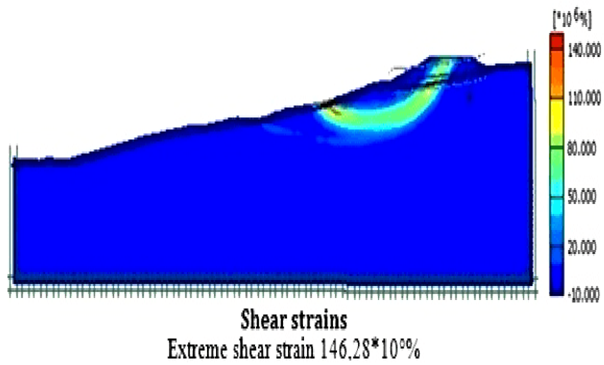


Fig. 9 Rupture surface (Shear stresses; phase ph/c reduction)

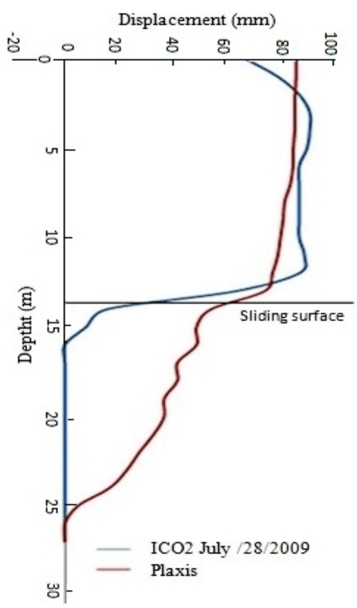


Fig. 10 Horizontal displacements (before the reinforcement)

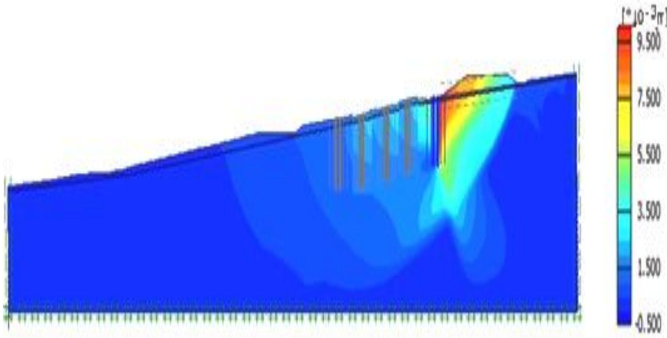


Fig. 11 Horizontal displacements with pile (9.55 mm) (c=1 kPa, φ = 8º : position $L_x/L \approx 0.25$)

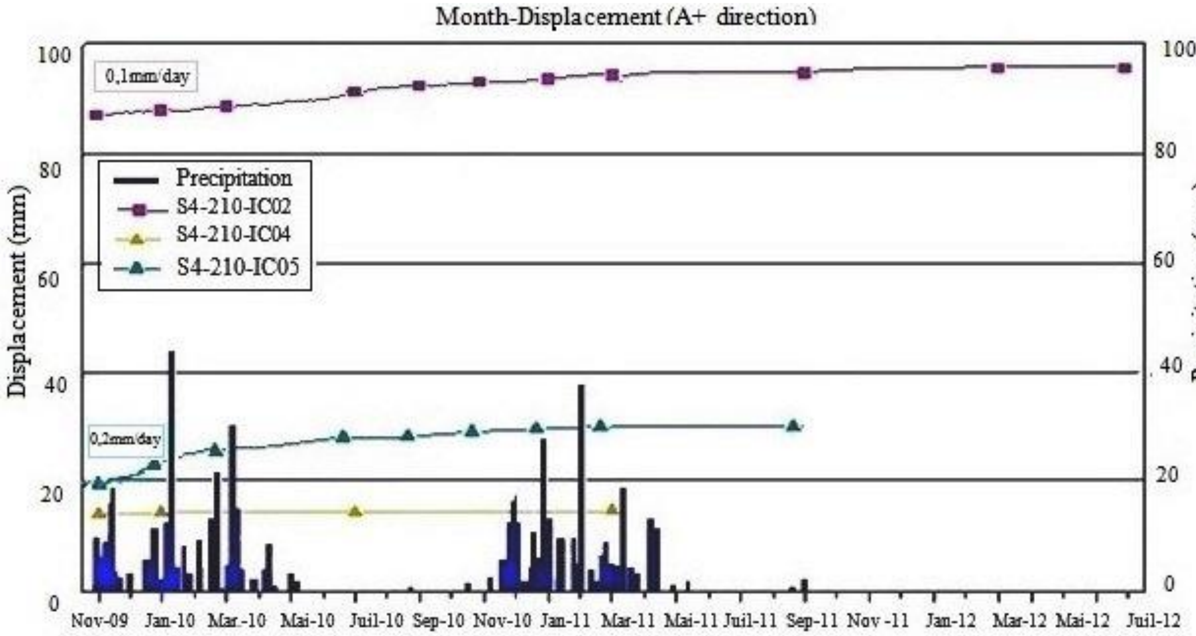


Fig. 12 Clinometridal measures of displacements

7.2 Numerical modeling of the Landslide

7.2.1 Mesh generation

The soil is meshed by triangular elements with 15 nodes. It consists of 312 elements, 2739 nodes and 3744 stres points with an average element size equal to 4.3 m for the ground and 2 nodes plate elements are used to simulate the behaviour of piles. (Fig. 8).

Fixed boundary conditions are applied at the bottom of the model, which means no deformation occurred in boundary. Moreover only vertical deformation can be occurred on both sides of the model.

7.2.2 Main results

The calculation shows the total displacements of the soil 87.99 mm (shown in Fig. 8), with horizontal displacements equal to 87.85 mm, obtained by PLAXIS 2D shows the displacements that took place in the soil before the pile design starts.

Fig. 9 shows the failure surface and the principal shear stresses in the phase phi/C reduction calculated by finite elements code PLAXIS 2D.

Basing on the stability analysis by FE and the inclinometer drilling operations (S4-210-IC02), the drawings of the active landslides are identified. The "virtual thickness" of the slipe plan, which is an imaginary dimension, is calculated according to the average of FE dimensions multiplied by the virtual thickness factor, which is taken equal to 0.1 by default. The virtual thickness or the width of the landslide or fractures is about 4.3 m * 0.1 = 0.43 m. This thickness is smaller when the FE mesh is denser. The landslide stability evaluation by use of the FE modelling gives a safety coefficient equal to 1.05 which is lower than 1.25. The slope is considered unstable. The technical measures are consisting of cast-in-place piles for purpose of the landslide stabilization.

7.2.3 Exploitation of results

For the purpose that the model be more reasonable and closer to reality, the horizontaldisplacements obtained by numerical modelling 2D are compared with experimentally results obtained by the inclinometer measurements S4 210+480-IC02 (Fig. 10).

According to the numerical results, the slope has a total

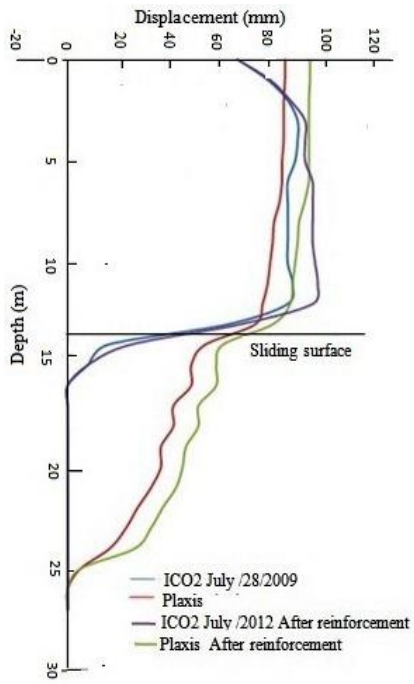


Fig. 13 Horizontal displacements(before and afer reinforcement)

displacement equal to 87.99 mm on the depth (0-14 m), compared to the inclinometer which indicates the displacement equal to 87.85 mm on the depth 13.5 m.

7.3 With reinforcement

In this section, the effect of the existence of piles on the factor of safety of slopes is presented. A reference case of a soil with cohesion C = 1 Kpa, friction angle φ =8° and a pile length l=27 m, diameter D = 1.2 m is first used. Numerical analysis and the inclinometer measurements show decrease in displacement from 87.99 mm (without pile) to 9.55 mm after the soil nailing (about 89%), where the factor of safety is 1,34. (Figs. 11 and 12). (Ausilio *et al.*2001, Griffiths *et al*. 1999, Ugai *et al.*1995, Kahyaoglu *et al.* 2012).

After reinforcement, we compared the numerical results of displacements with the measurements of the inclinometer (S4-210+480-IC02) for the purpose of validation.

At the beginning of November 2009, the company proceeded to the implementation of the piling wall. The procedure requires the resumption of displacement measures every two months as a precaution, the works have been continued until it is finished; the total deformations measured are reached to 9.84 mm (velocity in order of 0.1 to 0.2 mm / day—see Fig.12). After realisation of the piles, the deformations stabilize in September 2010.

Complementary measures carried out until July 2012 after embankment confirm the stabilization of the movements in September 2011: with a total displacement of 98.69 mm, these results are approximate with the results obtained in numerical analysis.

The figure above (Fig. 13) presents the comparison between numerical analysis (Plaxis) and the measurements of the inclinometer S4-246-INC02, according to the direction A, before and after reinforcement of the sliding; we notice that there is a concordance between them.

8. The effect of pile parameters on the factor of safety

8.1 Effect of pile position

The effect of changing pile position (L_x /L) which varied between the effect of changing pile position (L_x/L) (0.12, 0.25, 0.375,0.50, 0.625, 0.75, 0.87, 1) for different soil cohesions C (1, 2, 5, 8, 10, 12.5, 15 kN/m^2) ; thus , the safety factor of slope is obtained.

Fig. 14 shows piles position (L_x/L) in the slope where L is slope length.

Fig. 15 the safety factor values are given according to the (L_x/L) ratio. Results are in good agreement with that obtained by Jeong *et al.* (2003). To achieve the stability of slope, the pile must installed at the range (L_x/L = 0.37 to 0.62) compared with Jeong *et al.* (2003) (0.50 to 0.70).

From the figure, the values of factor of safety are higher

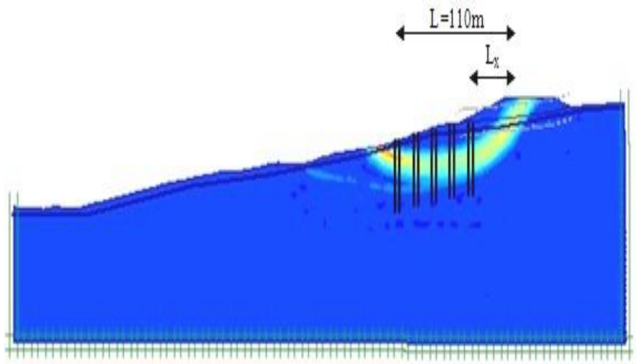


Fig. 14 Phase ph/c reduction (piles position)

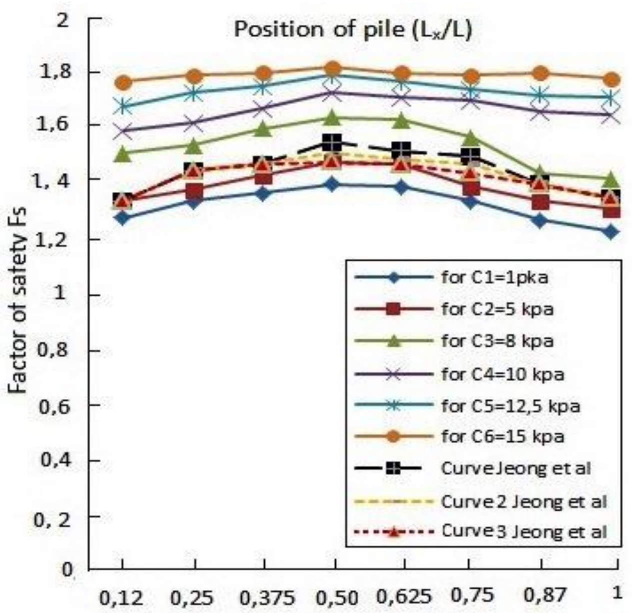


Fig. 15 Effect of pile position on factor of safety

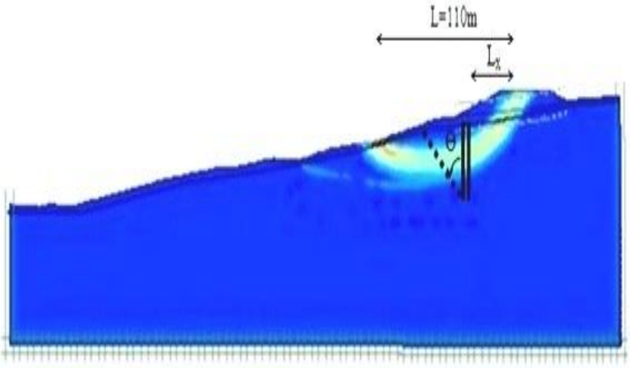


Fig. 16 Different pile inclination

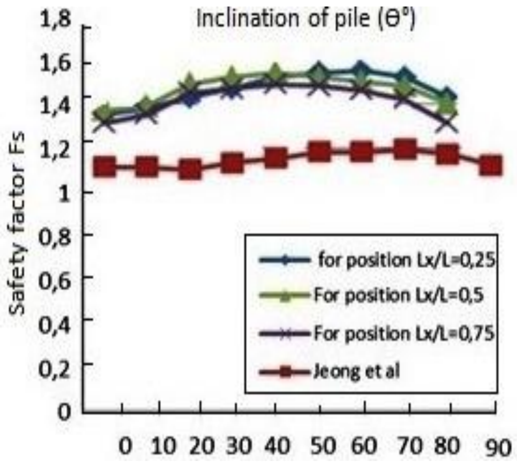


Fig. 17 Effect of pile position on factor of safety

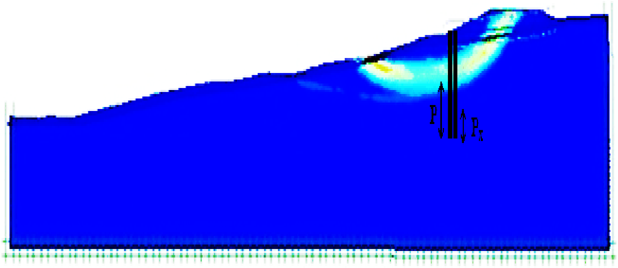
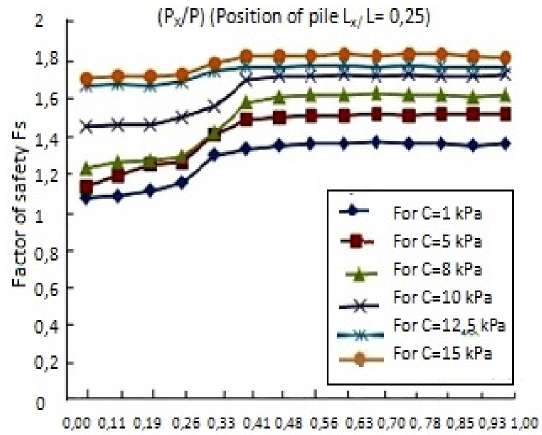
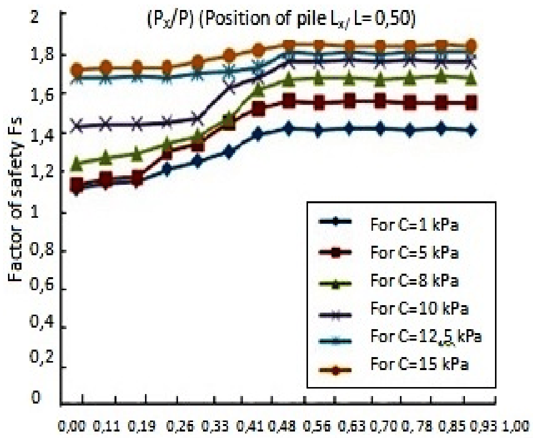


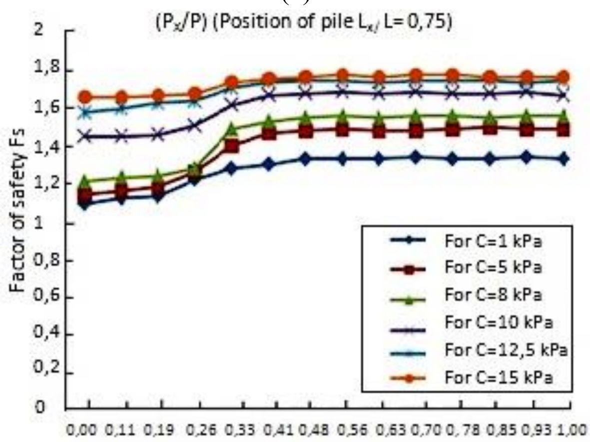
Fig. 18 Different pile length



(a)



(b)



(c)

Fig. 19 Effect of pile length on factor of safety

when the pile are located at the top of the slope than the values of the factor of safety when the pile are located at the slope toe and at the middle. By increasing the ratio (L_x/L), better amelioration in the stability of the slope is achieved. The best location of pile is when (L_x/L) is equal to 0.5 so the factor of safety reaches its maximum value (Abdelaziz *et al.* 2017, Wei *et al.* 2009).

8.2 Effect of pile inclination

Based on the results from the previous section, piles are placed with length equal to 27 m (Fig. 16). The Inclination of the pile from the vertical is varied (Θ = 0°, 10°, 20°, 30°, 40°, 50°, 60°, 70°, 80°, 90°), and its position L_x / L (0.25,0.50, 0.75).

Fig. 17 shows that the factor of safety increases with increasing of the inclination up to angle 60° because pile is located at (L_x/L = 0.25 or L_x/L= 0.5). Then, the factor of safety decrease with increasing of the inclination over angle 50° because pile is located at (L_x/L = 0.75).

It can be deduced that the factor of safety increases with increasing of the inclination up to angle 70° however, it decreased gradually after that angle (Jeong *et al.* 2003).

8.3 Effect of pile length

Based on the previous sections the pile parameters are taken as: *P* is the length of pile below the rupture surface

Fig. 20 The study section after the opening of the highway

and it is equal to double value of slip plan depth (27 m), with varying step sizes (2 m). Thus, the cohesions varied between (1, 5, 8, 10, 12.5, 15 MPa) and pile position (L_x/L = 0.25, 0.50, 0.75) as shown in Fig. 18.

Fig. 19(a): The factor of safety increases with increasing (p_x/p) ratio up to (p_x/p=0.38) because pile is located at (L_x/L= 0.25), because weak soils cohesion (cohesion less than 10 kN / m²). Also, the factor of safety increases with increasing (p_x/p) ratio up to (p_x/p=0.50) because pile is located at (L_x /L =0.50) as shown in Fig. 19(b).

On the other hand, the factor of safety increases with increasing (p_x/p) ratio up to (p_x/p=0.32) because pile is located at (L_x/L=0.75). However, after that (p_x/p=0.32) the factor of safety remained constant as the pile length ratio increases Fig. 19(c).

Pile is located at (L_x/L= 0.5), the factor of safety reaches the max value (Fs = 1.39) where it is equal to Fs ≈1.28 in pile which is located at (L_x/L = 0.25-75) for weak soils cohesion.

Pile is located at (L_x/L =0.25-0.5), the factor of safety reaches the max value compared to the pile which is located at (L_x/L = 75) for cohesive Soil.

The influence of the pile length depends on its location. If pile is located at (L_x/L <0.25), length has a little effect on the factor of safety. On the other hand, if pile is located at (L_x /L= 0.50), length has a considerable influence on the factor of safety (Yang *et al.* 2001).

9. Conclusions

The paper presents a 2D finite element study of slopes stabilized with piles of Didouche Mourad (Constantine, Algeria). This study demonstrates that the contributing factors of Didouche Mourad landslide are considerably different as compared to other countries.

• In this region, the presence of marl soil after saturation instigates landslides. Also, backfilling works along the highway mainly lead to the progressive slope failure.

• The comparison between the numerical analysis and the inclinometer displacement curves shows that our reference model is constitutes effectively a "reasonable approximation" of the actual slope behaviour.

• The numerical analysis using the finite element program (PLAXIS) gives a good agreement with the behavior of the existing slope at PK 210 +480.

• The existing slope reinforced with pile whose length is equal to 27 m (p_x/p = 0.50) and pile position L_x/L≈0.45 (L_x/L= 0.37 ~ 0.62), where the Section (PK 210+480 to PK210+800) was supported with piling wall positioned on the upper part and the middle of the slope, no cracks are detected even the highway has been put into service (Fig. 20).

• The 2D model was used to study the effect of different parameters on the factor of safety of slopes. These parameters contained the pile position from the top of the slope (L_x/L), the pile inclination from the vertical (Ө) and the pile length (l).

The results were compared with results from the literature. The main conclusions are summarized as follows:

• To calculate a landslide where soil is reinforced by piles determine their perfect position and their ideal length for all types of soil', we propose that the pile must be placed in the middle of sliding circle and it length is equal to twice the depth of sliding circle. Then, we need to change the position (L_x/L=0.37 to 0.62) and to vary the length (p_x/p = 0.32 to 0.60).

• The difficulties of reinforced soils modelling do not take into consideration the influence of the construction of these structures; some specifications are taken for each case of reinforcement processes.

• For better understanding the behaviour of soils after reinforcement, we must introduce monitoring instruments during the highway works.

Acknowledgments

We would like to show our gratitude to laboratory the company COJAAL, and we thank two anonymous reviewers for their so-called insights for their comments on an earlier version of the manuscript.

References

Abdelaziz, A., Hafez, D. and Hussein, A. (2017), "The effect of pile parameters on the factor of safety of piled- slopes using 3D numerical analysis", *HRBC J.*, **13**(3), 277-285. https://doi.org/10.1016/j.hbrcj.2015.06.002.

Ausilio, E., Conte, E. and Dente, G. (2001), "Stability analysis of slopes with piles", *Comput. Geotech.*, **28**(8), 591-611. https://doi.org/10.1016/S0266-352X(01)00013-1.

Fenu, L., Briseghella, B. and Marano, G.C. (2018), "Optimum shape and length of laterally loaded piles", *Struct. Eng. Mech.*, **68**(1), 121-130. http://doi.org/10.12989/sem.2018.68.1.121.

Griffiths, D.V. and Lane, P.A. (1999), "Slope stability analysis by finite elements", *Geotechnique*, **49**(3), 387-403. https://doi.org/10.1680/geot.1999.49.3.387.

Jeong, S., Kim B., Won, J. and Lee J.Y. (2003), "Uncoupled analysis of stabilizing piles in weathered slopes", *Comput. Geotech.*, **30**(8),671-682. https://doi.org/10.1016/j.compgeo.2003.07.002.

Jiang, S., Du, C. and Sun, L. (2018), "Numerical analysis of sheet

pile wall structure considering soil-structure interaction", *Geomech*. *Eng*., **16**(3), 309-320. http://doi.org/10.12989/gae.2018.16.3.309.
Kahyaoglu, M.R., Imançli, G., Önal, O. and Kayalar, A.S. (2012), "Numerical analyses of piles subjected to lateral soil movement", *KSCE Civ*. *Eng*. *J*., **16**(4), 562-570. https://doi.org/10.1007/s12205-012-1354-6.
Li, X., He, S., Luo, Y. and Wu, Y. (2011), "Numerical studies of the position of piles in slope stabilization", *Geomech*. *Geoeng*., **6**(3), 209-215. https://doi.org/10.1080/17486025.2011.578668.
Munawir, A., Dewi, S.M., Zaika, Y. and Soehardjono, A. (2013), "Bearing capacity on slope modelling with composite bamboo pile reinforcement", *Int*. *J*. *Eng*. *Adv*. *Technol*., **2**(5), 114-118.
Nian, T.K., Chen, G.Q., Luan, M.T., Yang, Q. and Zheng, D.F. (2008), "Limit analysis of the stability of slopes reinforced with piles against landslide in nonhomogeneous and anisotropic soils", *Can*. *Geotech*. *J*., **45**(4), 1092-1103. https://doi.org/10.1139/T08-042.
Tran, A.T., Kim, A.R. and Cho, G.C. (2019), "Numerical modeling on the stability of slope with foundation during rainfall", *Geomech*. *Eng*., **17**(1), 109-118. http://doi.org/10.12989/gae.2019.17.1.109.
Ugai, K. and Leshchinsky, D. (1995), "Three-dimensional limit equilibrium and finite element analysis: A comparison of results", *Soils Found*., **35**(4), 1-7. https://doi.org/10.15866/irece.v8i1.11147.
Wei, W.B. and Cheng, Y.M. (2009), "Strength reduction analysis for slope reinforced with one row of piles", *Comput*. *Geotech*., **36**(7), 1176-1185. https://doi.org/10.1016/j.compgeo.2009.05.004.
Won, J., You, K., Jeong, S. and Kim, S. (2005), "Coupled effects in stability analysis of pile-slope systems", *Comput*. *Geotech*., **32**(4), 304-315. https://doi.org/ 10.1016/j.compgeo.2005.02.006.
Wu, J.J., Cheng, Q.G., Liang, X. and Cao, J.L. (2014), "Stability analysis of a high loess slope reinforced by the combination system of soil nails and stabilization piles", *Front*. *Struct*. *Civ*. *Eng*., **8**(3), 252-259. https://doi.org/10.1007/s11709-014-0260-z.
Xu, J., Li, Y. and Yang, X. (2018), "Stability charts and reinforcement with PILEs in 3D nonhomogeneous and anisotropic soil slope", *Geomech*. *Eng*., **14**(1),71-81. http://doi.org/10.12989/gae.2018.14.1.071.
Yang, S., Ren, X. and Zhang, J. (2011), "Study on embedded length of piles for slope reinforced with one row of piles", *J*. *Rock Mech*. *Geotech*. *Eng*., **3**(2), 167-178. https://doi.org/10.3724/sp.j.1235.2011.00167.

CC