Conventional and modified design methods for landslide stabilizing piles: a comparison of results

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ABSTRACT

A limit equilibrium-based procedure for design of landslide stabilizing piles is often used in practice in Japan. It usually involves three main steps: i) determining the strength parameters from back analysis of a known failure surface; ii) evaluating the lateral resistance which should be provided by piles to increase the factor of safety to the desired value; and iii) selecting the type/number of piles and the most suitable location of these piles within the slope. Each one of the above elements has been studied individually in the literature. However, the interrelations between these elements have not been sufficiently emphasized. This paper is presented to examine the interrelation between the first two steps (i.e. steps i) and ii)), describing defects of the conventional design procedure used in Japan, restating a modified approach to overcome some of these defects, and more importantly, performing a numerical comparison of the results obtained using these two different procedures. Comparisons are made for a typical landslide slope. The findings are summarized to support a recommendation of the use of the modified method for a reliable design of pile-reinforced slopes.

1 INTRODUCTION

In Japan, cast-in-place piles as large as 1.0 to 3.5m diameter are used to stabilize active landslides or to improve the stability of unstable slopes. Limit equilibrium-based design of such piles in landslide slopes is routinely carried out. The design procedure usually consists of three main steps [Yamagami et al. 1992]: i) determining strength parameters (cohesion c and internal friction angle ϕ) from back analysis of a known failure surface in a landslide slope; ii) evaluating the stabilizing force needed to increase the factor of safety to the desired value; and iii) selecting the type/number of piles as well as the most suitable location of these piles within the slope.

A number of techniques are now available for step i). Empirical methods [Duncan & Stark 1992] were proposed to estimate c and ϕ by assuming one of these parameters and back-calculating the other for a safety factor of unity. The Japan Road Association [1999] suggested an analysis method in which c (kN/m^2) is assumed equal to maximum depth (meter) of a known failure surface first and then ϕ for F (the factor of safety) =1.0 is calculated. On the other hand, it has been shown that the magnitude of both c and ϕ can be determined from single slips in a homogeneous slope by considering that 1) the actual failure surface is consistent with the theoretical critical slip surface and 2) the factor of safety should be equal to unity [e.g. Saito 1980, Yamagami & Ueta 1989, 1992, Wesley & Leelaratnam 2001].

Step ii) generally uses slope stability analysis by means of limit equilibrium methods. By specifying a desired value of the factor of safety and using the strength parameters obtained from step i), the required total resisting force can easily be evaluated. This resisting force should be exerted on the upside soil by piles. Different approaches were recommended to include this additional resistance in the factor of safety equations used for analysis of pile-stabilized slopes. The inclusion of a lateral concentrated force at the intersection of the pile and the slip surface is suggested by the Japanese Committee on Stabilizing Technology of Landslides [2003].

It is obvious that the final step of pile design for landslide stabilization is greatly influenced by both procedures used in steps i) and ii). Each one of the above elements has been studied widely in the literature. However, the interrelations among these elements have not been sufficiently emphasized. This paper is presented to investigate the interrelations between the first two steps; stating defects of the conventional design procedure used in practice in Japan, redescribing a modified approach to overcome some of these defects, and more importantly, conducting a numerical comparison of results obtained using these two different procedures for a typical landslide slope.

2 BACKGROUND

2.1 Conventional and modified methods for back-calculating strength parameters

An empirical method was recommended by the Japan Road Association [1999] to determine strength parameter values from a known failure surface. In this method, c (kN/m^2) is assumed equal to vertical maximum depth, D in meter, of the slip surface first and then ϕ for F =1.0 is calculated. The soil shear strengths so obtained are then used for the design of stabilization works for landslide slopes. The use of c (kN/m^2) = D (meter) (the c=D method hereafter) is a rather crude manner of estimating the soil cohesion for a failed slope. Therefore, strength parameters back-calculated using this conventional method may not be reliable (e.g. Jiang et al. 2005).

As mentioned previously, the magnitude of both c and ϕ can be determined by considering the position of the actual slip surface together with the fact of *F*=1.0 [e.g. Saito 1980, Yamagami & Ueta 1989, 1992, Wesley & Leelaratnam 2001]. The approach by Yamagami & Ueta [1989, 1992] can easily be incorporated into any existing method of slices for back calculating c and ϕ from a known failure surface. This approach is based on two essential considerations: 1) the required strength parameter values, c_0 and ϕ_0, must satisfy F_0 =1.0 where F_0 is the safety factor of the failure surface; and 2) F_0 should be the smallest factor of safety of the slope under consideration.

Combining the first consideration above with an existing method of slices yields the relationship between *c* and tanϕ, as shown in Figure 1 (a). Values of c_{max} and $\tan\phi_{max}$ in Figure 1 (a) show a possible range of change in c and tanϕ satisfying F_0 =1.0 and can readily be computed from the failure surface by giving ϕ=0.0 and c=0.0 respectively. It is obvious that the required strength parameters, c_0 and $\tan\phi_0$, should lie at some point on the c-tanϕ curve shown in Figure 1 (a). To determine the position of this point, Yamagami and Ueta [1989, 1992] presented a unique procedure by selecting a number of trial slip surfaces slightly inside and slightly outside the failure plane [Figure 1 (b)] and examining the variation of their safety factors [Figures 1 (c), (d)]. A three-dimensional space with three coordinates F, c and tanϕ is described in Figure 1 (c) in which the curve PIQ represents the c-tanϕ relationship in Figure 1 (a). The first consideration mentioned above implies that c_0 and $\tan\phi_0$ should correspond to a point on the curve PIQ. To locate the point exactly, the second condition was considered by Yamagami and Ueta [1989, 1992]. That is, the change in the safety factor, *F*, of each of the trial slip surfaces in Figure 1 (c) was examined using a number of c and tanϕ combinations taken from the curve PIQ (i.e. these combinations of c and tanϕ satisfy the first condition). As an example, the curve RIS in Figure 1 (c) schematically illustrates the distribution of the safety factor calculated for a trial slip surface near the failure plane. If the curve RIS is redrawn in the F-c plane and F-tanϕ plane, respectively, then the relationships shown in Figure 1 (d) are obtained.

The second consideration by Yamagami and Ueta means that the safety factor of any trial slip surface should be greater than F_0. In other words, the condition of "F for all trial slip surfaces $\geq F_0$" must be satisfied. It is seen from Figure 1 (d) that only c and tanϕ values corresponding to the part IQ of the curve PIQ satisfy the second condition. Therefore, c_0 and $\tan\phi_0$ should locate at some point on the curve IQ. With reference to Figure 1 (d), the following two inequalities can be written:

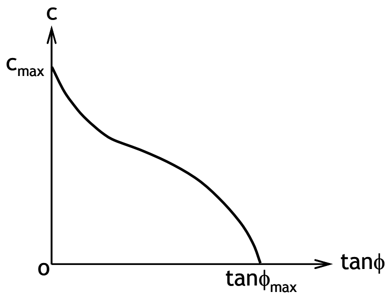
$$c_I \leq c_0 \leq c_{max} \quad \text{and} \quad 0 \leq \tan\phi_0 \leq \tan\phi_I \tag{1}$$

It was also shown by Yamagami and Ueta [1989, 1992] that if a trial slip surface near the failure plane was selected in the opposite side of the above-mentioned slip surface, its safety factor would vary in a reverse way of curve RIS, as shown by curve R′JS′ [Figure 1 (d)]. Similarly, considering $F \geq F_0$ yields two inequalities as follows, indicating a possible range of change in the required (c, tanϕ) values.

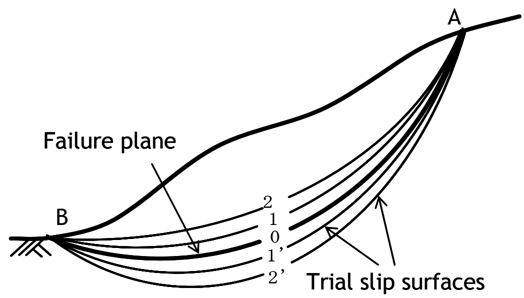
$$0 \leq c_0 \leq c_J \quad \text{and} \quad \tan\phi_J \leq \tan\phi_0 \leq \tan\phi_{max} \tag{2}$$

From Equations (1) and (2), we have: $c_I \leq c_0 \leq c_J$ and $\tan\phi_J \leq \tan\phi_0 \leq \tan\phi_I$.

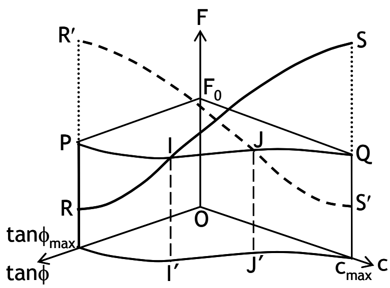
It is seen that the ranges within which c_0 and $\tan\phi_0$ exist have significantly been narrowed by examining two trial surfaces which are chosen separately above and below the actual failure surface. Yamagami and Ueta [1989, 1992] have shown that when the above operation is repeated for several trial slip surfaces slightly inside and slightly outside the failure surface, strength parameter values of satisfying the both considerations would be restricted within an extremely narrow range; thereby enabling us to identify a pair of c and tanϕ values uniquely. One of the advantages of the Yamagami



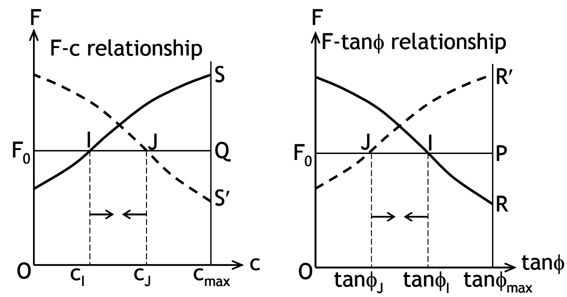
(a) c-tanϕ relationship for F_0=1.0



(b) Failure plane and trial slip surfaces



(c) Variation of the safety factor of trial slip surfaces



(d) Restriction of possible ranges of change of the required c and tanϕ values

Figure 1: Basic principle of back analysis method proposed by Yamagami and Ueta (1989, 1992)

And Ueta's method is that it can easily be incorporated into any existing factor of safety equations to back calculate c and ϕ from a know failure surface. The details of the analysis method were described in the previous publications [e.g. Yamagami & Ueta 1989, 1992].

2.2 Conventional method to evaluate resisting forces needed to increase slope stability

A conventional approach is routinely used to design landslide slopes stabilized by (rigid) piles in Japan. In this method, resistance exerted on the sliding body by piles is considered as a concentrated force at the intersection of the pile and the slip surface. Its direction is assumed to be tangential to the slip surface at the pile position, and its magnitude is estimated based on the Ordinary Method of Slices, as shown in Equation (3) [e.g. Nakamura, 1984].

$$F_0 + \Delta F = [\Sigma cl + \Sigma (W con\alpha - ul)\tan\phi + P] / \Sigma T \qquad (3)$$

where F_0 is the current safety factor value of the slip surface, which is usually taken as 1.0 for failed slopes, ΔF is a required increase in the factor of safety, W is total weight of a slice, α, l is inclination and length of the base of the slice, respectively, c and ϕ are Mohr-Coulomb strength parameters of soil, P represents the stabilizing force, per unit thickness of soil, that must be provided by the pile(s) to increase the safety factor from F_0 to F (= F_0+ΔF) (where F is normally called target factor of safety). By specifying a target value for F, P can readily be computed from Equation (3).

Problems and limitations of the above-mentioned method were discussed by Yamagami et al. [1992]). One of the main problems is that P calculated by Equation (3) would be constant irrespective of the location of the pile row installed in a slope. This is not true because the stabilizing force by pile should change with the location of the pile row. Another problem of the conventional method is that Equation (3) does not offer any information on the necessary number of rows of piles. Equation (3) is also used in practice for noncircular failure planes although it is derived only for circular slip surfaces. As discussed by Yamagami et al. [1992], the design results (step iii)) of stabilizing piles based on the conventional procedure may be misleading due to its defects involved in steps i) and ii).

2.3 Modified method to evaluate resisting forces needed to increase slope stability

To overcome the defects of the conventional method, Yamagami et al. [1992] proposed a simplified procedure to design stabilizing piles for landslide slopes. It consists of the following 4 steps: i) determining strength parameters from back analysis of a known failure surface; ii) evaluating the lateral resisting force, P, needed to increase the safety factor to the desired value; iii) determining the ultimate lateral resistance of each pile, P_R; and iv) selecting the type and number of piles and the most suitable location of these piles within the slope. A similar design of slope stabilizing piles was also presented by Poulos [1995].

A cross section of a typical landslide with piles in a row is illustrated in Figure 2. The method by Yamagami et al. assumes that the factors of safety, F_a and F_b, of the upslope and downslope sliding masses bounded by the pile row may be different. It is quite natural to make such a consideration as the piles can provide resistance only to the upslope soil mass. In order to justify this, let us consider an example in which strong (rigid) piles have been installed deeply enough in an active landslide slope. The upslope soil mass would be stopped due to the sufficient resistance of the piles, and therefore become stable. However, the downslope soil mass might still be unstable (thus the movement would continue). Consequently, the factor of safety values of the upslope and downslope sliding masses differ from each other. Generally, the magnitude of these two factors of safety is unknown.

The second consideration made by Yamagami et al.[1992] is that interactive forces between the pile row and soil masses are estimated by prescribing a target value of the factor of safety. This implies that a desired, target value for F_a and F_b is first specified by the designer, and then piles are designed to provide resisting force that is needed to achieve the desired value. By neglecting friction between the piles and soil masses and assuming the position of acting point of the interactive forces, P_a and P_b (see Figure 2) corresponding to the prescribed values of F_a and F_b can readily be computed using an existing method of slices. The piles must provide an resistance not smaller than P_a-P_b so as to ensure the desired factor of safety for the pile-slope system (Yamagami et al. 1992).

The design of landslide stabilizing piles is dependent upon both the method of back-calculating the soil strengths and the approach of evaluating the resistance needed to increase the slope stability. While each one of the two aspects has been studied widely, the interrelations between these elements have not been sufficiently emphasized in the previous work. The following section presents a numerical comparison of the results using the conventional and modified procedures described in 2.1-2.3 for a typical landslide slope. The comparison is meaningful because it could provide a more comprehensive view on pile-reinforced slopes designed using these two different procedures.

3 NUMERICAL COMPARISON AND DISCUSSION

The Selset landslide in boulder clay, reported by Skempton [1964], is considered to be a good case for a comparison of the first two steps of the conventional and modified design procedures for stabilizing piles, because 1) the c and ϕ values of the uniform boulder clay involved in the slide were determined by extensive laboratory tests and previous back analysis; 2) the location of the failure surface was investigated in detail based on the tension cracks observed near the top of the slope and the stability analysis results [Skemptom & Brown 1961]; and 3) the free water surface was defined by the shallow piezometers and open boreholes and the flow net was well drawn to describe the distribution of pore-water pressures in the slide [Skemptom & Brown 1961].

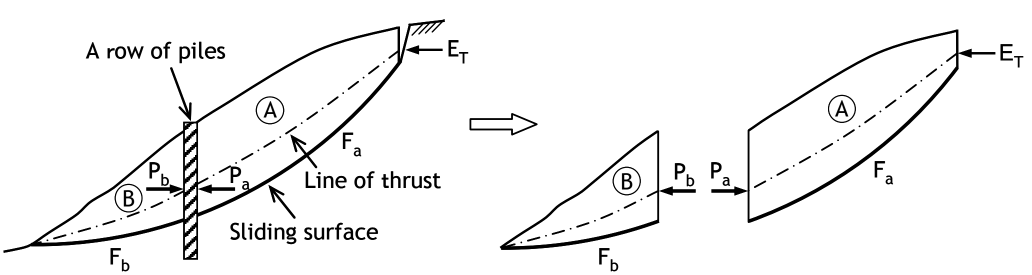
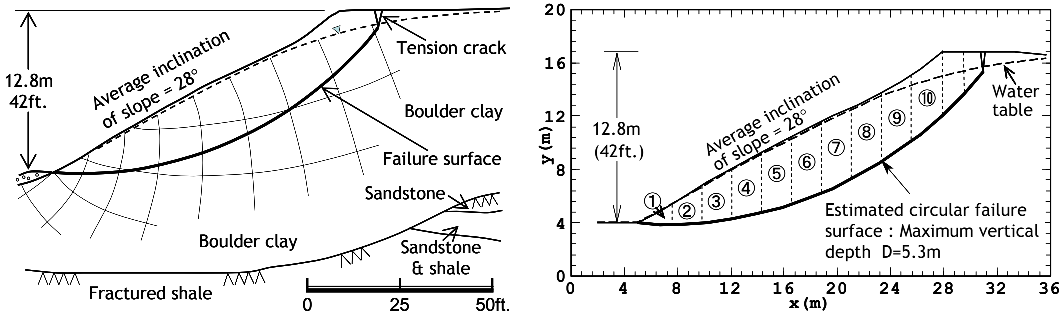


Figure 2: Method to estimate the required resisting force of piles [Yamagami et al. 1992]



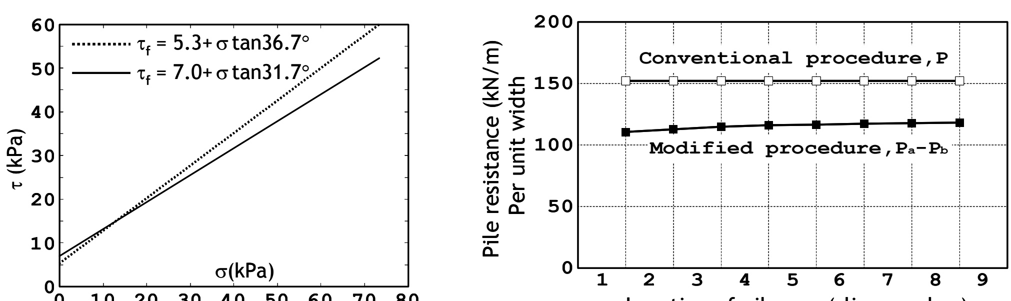
(a) A cross section [after Skempton 1964] (b) divided vertical slices

Figure 3: Selset landslide

After investigating the site in details, Skempton et al. [1964] concluded that the slope failure in the Selset slide was caused by river water erosion to the valley side. Triaxial and shear box tests on eight samples of the boulder clay gave a consistent set of results, showing that the peak strength could be expressed by c'=8.61kN/m^2 (180.0 lb/sq.ft) and ϕ'=32.0°. If the slip had been analysed using these strength parameters, the calculated factor of safety would have been very close to (but a little larger than) its correct value of 1.0. Therefore, Skempton [1964] concluded that the peak strength had been almost fully mobilized simultaneously along the entire length of the slip surface.

A back analysis of the Selset landslide is first carried out using the *c=D* method. The sliding mass over the circular slip surface in Figure 3 (a) was divided into 12 vertical slices, as shown in Figure 3(b), and then maximum depth of the failure surface was found to be *D*=5.3m. Thus, c=5.3 kN/m^2 was obtained from the *c=D* method and ϕ=36.7° was calculated for *F*=1.0 using the Ordinary Method of Slices. As the failure surface in Figure 3 is a circle, the standard Bishop's method was adopted to back calculate the strength parameters using the Yamagami and Ueta's method. This yields c'=7.0 kN/m^2 and ϕ'=31.7° for F=1.0. These results are quite close to (but a little smaller than) the Skempton's laboratory values. Figure 4(a) shows the strength envelopes defined by the back calculated parameters. Noting that the effective normal stress on the failure surface in Figure 3 ranges from 30 to 60kP$_a$, the soil strengths by the c=D method was higher than its real values by 10% to 14% with an average of 12%.

Now let us assume that the Selset landslide would be reinforced by piles in a row to increase its factor of safety from the current 1.0 to a desired value of 1.2. The pile resistance, P (per unit width), was first calculated using c=5.3 kN/m^2, ϕ=36.7° and Equation (3) for different location of the pile row. Then, the Yamagami et al.'s method [1992] with c'=7.0kN/m^2 and ϕ'=31.7° was used to evaluate the magnitude of (P_a-P_b) for F=1.2. The results obtained are illustrated in Figure 4(b). Although the larger soil strengths are used, Equation (3) overestimates the pile resistance by approximately 30.0%. It is clear that the use of the correct (c, ϕ) values in the conventional procedure will lead to a larger overestimation of the resisting forces required for an increase in the safety factor [Yamakawa 1995]. It is noted that Equation (3) is also employed for noncircular failure surfaces in engineering practice. In such cases, the uncertainties of the conventional procedure will become more pronounced due to the defects in steps i) and ii) of the method [Yamagami et al. 1992, Yamakawa 1995].



(a) back calculated strengths (b) Pile resistance required for F=1.2

Figure 4: Comparison of results of the conventional and modified procedures for Selset landslide

4 CONCLUSIONS

A limit equilibrium-based method is routinely used in Japan to analyse landslide slopes stabilized by piles. It involves three main steps: i) determining the strength parameters from back analysis of a known failure surface; ii) evaluating the resistance which should be provided by piles to increase the factor of safety to the desired value; and iii) selecting the type/number of piles and the most suitable location of these piles within the slope. This paper was presented to discuss significant defects in the conventional design of stabilizing piles and to describe a modified procedure to overcome some of theses defects. The interrelations between the first two steps were examined through a comparison of the results using the two different methods.

The results presented in this paper together with those reported in the previous work [Yamagami et al. 1992, Yamagami & Ueta 1989, 1992, Yamakawa 1995] lead to the following conclusions: 1) The soil strengths back calculated by the c=D method are usually larger than their correct values along an actual failure surface; and 2) Although the larger soil strengths are used, Equation (3) significantly over- estimates the pile resistance required for an increase in the factor of safety. It was also shown by Yamakawa (1995) that the uncertainties of the conventional procedure become more pronounced for noncircular failure surface cases. Consequently, the use of the modified procedure as described in 2.1 for step i) and 2.3 for step ii) is strongly recommended for a reliable design of landslide slopes reinforced by piles.

REFERENCES

Duncan, J. M. and Stark, T. D. (1992). Soil strengths from back analysis of slope failures. Stability and Performance of Slopes and Embankments-II. ASCE Geotechnical Special Publication No.31, 1, 890-904.

Japan Road Association. (1999). Guidelines for slope stabilization works and slope stability. Japan Road Association, (in Japanese).

Japanese Committee on Stabilizing Technology of Landslides (2003). Design guideline for steel pipe piles to stabilize landslides (in Japanese).

Jiang, J.-C., Yamagami, T. and Kanda, Y. (2005). Estimation of strength parameters from maximum depth of slip surafce in a failed slope. J. of the Japan Landslide Society, 42(1), 8-16, (in Japanese).

Nakamura, H. (1984). Design of rigid dowel piles for landslide control. Proceedings of the 4th International Symposium on Landslides. Toronto, 2, 149-154.

Poulos, H. G. [1995]. Design of reinforcing piles to increase slope stability. Can. Geotech. J., 32, 808-818.

Saito, M. (1980). Reverse calculation method to obtain c and ϕ on a slip surface. Proc. 1st Int. Symp. on Landslides. New Delhi, 1, 281-284.

Skemptom, A. W. (1964). Long-term stability of clay slopes. Geotechnique, 14(2), 77-102.

Skemptom, A. W. and Brown, J. D. (1961). A landslide in boulder clay at Selset, Yorkshire. Geotechnique, 11(4), 280-293.

Wesley, L. D. and Leelaratnam, V. (2001). Shear strength parameters from back-analysis of single slips. Géotechnique, 51(4), 373-374.

Yamagami T., Suzuki, H. and Yamakawa O. (1992). A simplified estimation of the stabilizing effect of piles in a landslide slope applying the Janbu Method. Proc. 6th Int. Symp. on Landslides, 613-618.

Yamagami, T. and Ueta, Y. (1989). Back analysis of average strength parameters for critical slip surfaces. Proc. of the Int. Symp. on Computer and Physical Modelling in Geotech. Engrg., 53-67.

Yamagami, T. and Ueta, Y. (1992). Back analysis of strength parameters for landslide control works. Proc. 6th Int. Symp. on Landslides, 619-624.

Yamakawa O. (1995). Simplified design method of slope reinforced with piles and anchors. Ph.D. Thesis, The University of Tokushima, (in Japanese).