

Nature of Shear Strength Mobilization and Initiation of Progressive Failure Along Shear Plane of Soil Slope

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Received: September 28, 2025, Accepted: November 7, 2025

Abstract: Several cases of failed soil slopes have been analyzed using a method of progressive failure to identify the nature of shear strength mobilization at failure. The failure analysis used method of slices within the limit equilibrium technique with a closed form solution. The study found maximum shear strength was mobilized towards the places where local failure was supposed to occur. The numerical calculations also showed that shear strength in several local places along shear planes were readjusted to retain local factors of safety more than unity. Failures eventually occurred (F closes to unity) where strength mobilization failed to retain local factor of safety as unity for most of the places along the shear plane. Shear strength and its readjusted distribution along the entire shear plane was found to be non-uniform.

Keywords: Progressive failure, Failure plane, Shear strength mobilization, Slope.

Introduction

In many cases, especially with soil slopes and earth embankments, shear failure does not occur instantaneously. Instead, small, localized sections of the potential failure plane may reach their peak strength first. Methods considering local failure mechanism which extends progressively along entire potential failure plane can be useful in assessing actual state of stress in any slope. There are several slope stability analysis methods considering progressive nature of failure with strain-softening behavior (Chowdhury et al., 1987; Wieberg et al., 1990; Hayashi and Tang, 1994). Incorporation of strain-softening properties of soil needs deformation analysis with a repetitive nature of calculation and also requires large amount of input data for the closed form solution. One of the most popular and widely used methods is limit equilibrium method (LEM) in slope stability analysis. A progressive failure analysis approach within LEM considering slices have been developed (Khan et al., 2002) that can handle local failure mechanism with gradual strength mobilization along potential failure plane.

Shear strength mobilization is the gradual progress of resisting shear stress within a soil mass as it is subjected to active stress. The soil will only be mobilized as much shear resistance as needed to reach equilibrium with the active shear stress until it reaches its peak. Failure occurs when the active shear stress exceeds the maximum available shear strength along

the failure plane of slope. This mobilization process is not linear for entire slip surface. The shear strength does not activate simultaneously at all points along a potential slip surface, which is a key concept for soil slope stability. There are available methods (Liu, 2009) for studying the characteristics and patterns of such strength mobilization are solely based on strain-deformation based approaches.

The present article focuses on characterizing the local shear strength mobilization during progressive failure mechanism within the LEM using the method proposed by Khan et al. (2002).

Methodology

The present methodology was principally based on a method of progressive failure analysis using non-vertical slices developed by Khan et al. (2002). The local mobilized shear strength (Figure 1) for the i^{th} slice ($T_{i,mob}$) of any soil slope can be calculated using (1) simultaneously during the determination of local factors of safety, F_i . Here c, φ are the strength parameters of soil, u is the pore water pressure and W is the weight of a slice. All the geometrical parameters are self-explanatory.

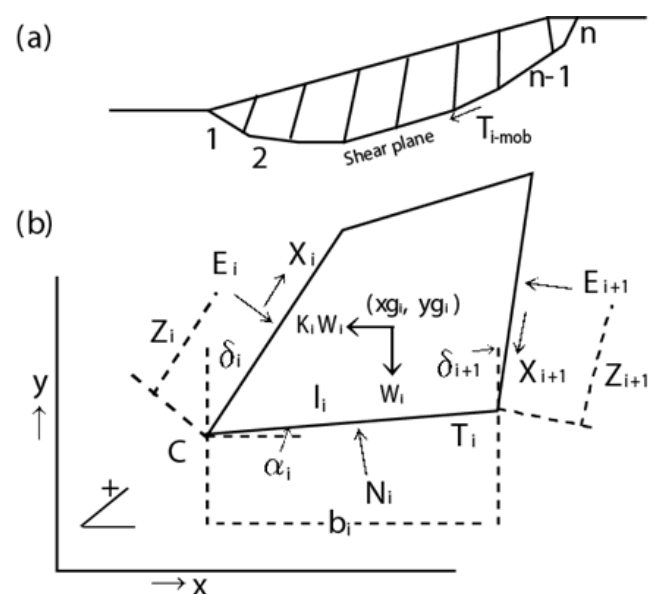


Figure 1, A general slope geometry with slices (a) and stresses acting on i^{th} -slice (b) (modified after Khan et al., 2002).

A value of F_i was assumed to reasonably be close to unity for any local failure in progress.

$$T_{i_mob} = \frac{c_i l_i + (N_i - u_i l_i) \tan \varphi_i}{F_i} \quad (1)$$

The local normal strength at slice base (N_i) can be found using the equation (2) as follows:

$$N_i = \frac{1}{m_1 F_i} (W_i - c_i l_i \sin \alpha_i + u_i l_i \sin \alpha_i + \frac{1}{m_1} (X_i \cos \delta_i + X_{i+1} \cos \delta_{i+1} - E_i \sin \delta_{i+1} + E_{i+1} \sin \delta_i)) \quad (2)$$

where, $m_1 = \frac{\sin \alpha_i + \tan \varphi_i}{F_i} + \cos \alpha_i$;

In the present analysis, the location of the stresses acting at the inter-slice boundaries for the entire slices and the location of the normal stresses at the base of the slices are not fixed. All the above-mentioned locations were allowed to vary so that boundary conditions (3 and 4) were satisfied at the end of the calculation after several iterative calculations. It may be noted that the entire calculation was completed with an optimization procedure within each iterative calculation cycle.

Within the limit equilibrium method (LEM) several boundary conditions (3 and 4) have been devised to solve the above equations.

$$E_1 = 0; X_1 = 0 \quad (3)$$

$$E_{n+1} = 0; X_{n+1} = 0 \quad (4)$$

However, to judge the overall stability of any slope one must consider the factor of safety. Therefore, an overall factor of safety (F) was defined (5) and determined at the end of the calculations when all the boundary conditions were satisfied for the entire slip surface.

$$F = \frac{\sum av. available shear strength(T, T_{mob})}{\sum average shear stress} \quad (5)$$

A thrust line connecting all the optimized points of stress action at the inter-slices boundaries was also identified at the end of the entire calculation process.

Results and discussion

A few soil slopes have been studied to understand the nature and mechanism of local strength mobilization during the initial stages of failure plane development along shear surface. The results from several slope analyses using the above-mentioned calculation procedures have been given in Table 1.

Table 1, Shear strength and mobilization of shear strength of studied slopes.

Slope failure	Before mobilization; Ave. T (kPa); Local T_{local}	After mobilization; Min. local T_{mob} (kPa)	Comments/ Reference
Carsington dam in Darbyshire, UK	T=79.88 T_{local} =18.8	4.9 (near the toe)	Failed F=1.09
Selset road cut section, Yorkshire (UK)	T=64.2 T_{local} =15.3	15.03 (near the toe)	Failed F=1.04
Rajshahi City Protection Embankment (RCPE), Bangladesh	T=82.37 T_{local} = 34.7	15.78 (near the toe)	Stable F=1.41

Two most debating slope failure namely Carsington dam failure that occurred near the end of construction in June of 1984 in Darbyshire, UK (Figure 2) and the Selset road cut section failure of Yorkshire, UK (Figure 3) were analyzed in this study.

Skempton and Brown (1961) first analyzed the Selset slide with Bishop method of slices using constant strength along the entire failure plane but failed to have a reasonable factor of safety. The present analysis using locally variable strength distribution along the slip surface found suitable explanations about the failure mechanism of this slide. Several investigations

(Vaughan,1991; Skempton, 1985; Skempton and Vaughan, 1993; Khan, et al., 2002) were also done for the Carsington Dam slide and the Selset landslide in UK.

Rajshahi City Protection Embankment (RCPE) of Bangladesh, a stable slope was also analyzed to show the successive mobilization of shear strength locally to protect the local failure initiation and thus ensuring the overall factor of safety (F) much higher than unity (F=1.41). Analyses of potential shear planes clearly depict the shear strength mobilizations in the present study.

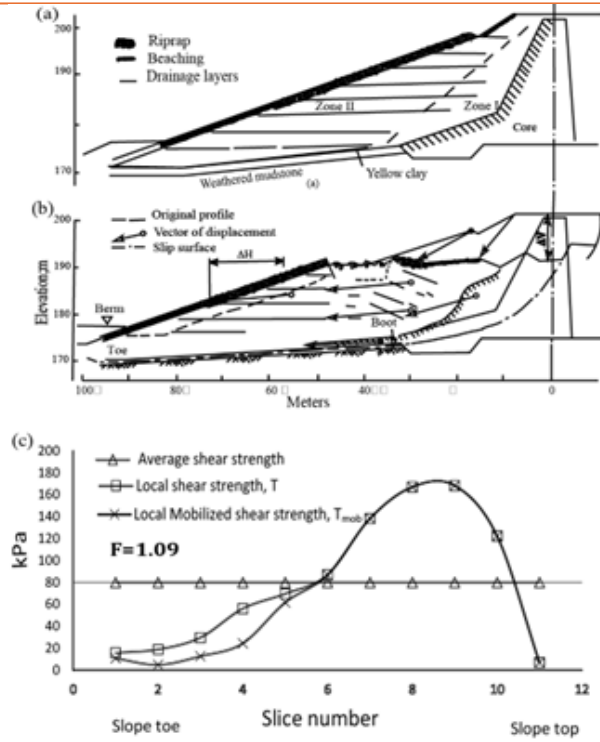


Figure 2, Carsington Dam slide: Slope profiles and materials before(a) and after(b) failure (modified after Skempton and Coats, 1985) and shear strength mobilization along the failure plane (c).

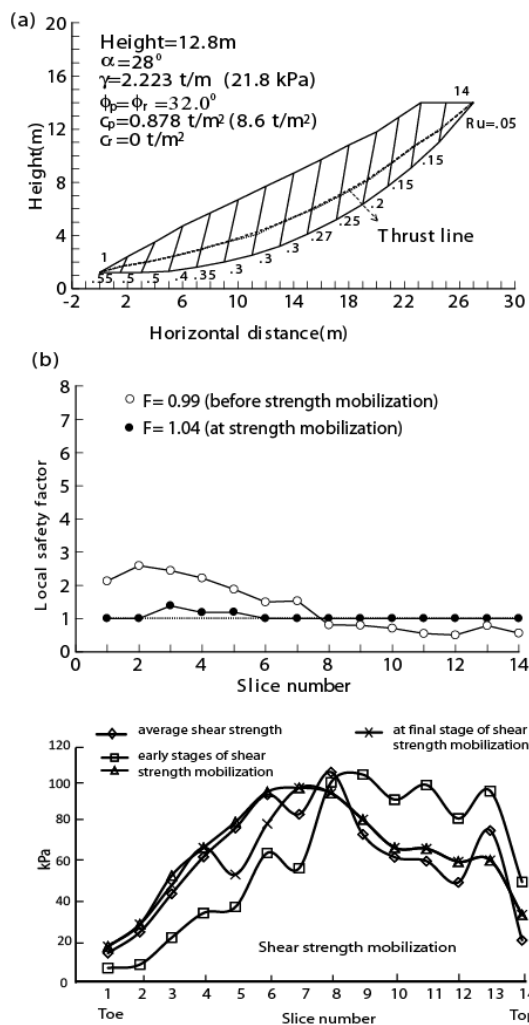


Figure 3, Selsset landslide: Slope geometry and strength properties(a), local safety factors with non-vertical slice(b) and local shear strength, T and local mobilized shear strength, T_{mob} (c).

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