Research on failure mechanism of pebble slope based on nonsynchronous double strength reduction method

HUANG Yi1,2*, ZHANG Wenhan1,2, Li Liang1,2, and CHENG Lijuan1,2, and XieZhengfu1,2
1Chengdu Engineering Corporation Limited, Power China, Chengdu, PR China
2Sichuan Urban Underground Space Survey Design and Construction Technology Engineering Laboratory, Chengdu 611130.
*Corresponding author: p2020233@chidi.com.cn; ORCID identifier: 0000-0002-1640-4953

Abstract: This project is located on the gentle slope of the third-level terrace in the upper reaches of the Dadu River. Because of the deep ice water accumulation layer and the inclusion of a large number of pebbles and silt lenses, studying the failure mechanism of the slope of the deep heterogeneous pebble accumulation body is critical. The cohesive force and internal friction angle are nonsynchronously reduced through the software program by comparing the slope equivalent to the plastic strain area calculated by nonsynchronous reduction step conditions with the stress and strain data monitored on site. Simultaneously, the displacement and load data of the sample during the staged loading process, were calculated in detail. The results proved that during the predetermined load application process, the pebbles are gradually shifted, and the silt lens interspersed between the pebbles is quickly destroyed. The friction angle between the pebbles provides the shear strength, further verifying the rationality of the nonsynchronous dual-strength reduction method. Compared with the general synchronous reduction method, analyzing the slope stability is more reasonable. Simultaneously, the double strength reduction method provides a solution to calculate the slope stability, and can be used for similar slope projects in the future.

Key words: Double strength reduction method; slope stability; pebble accumulation; effective plastic strain.
1 Introduction

The stratum of the proposed project site is mainly gravelly soil with many pebbles and silt, and the distribution is discontinuous. Because of the complex process and mechanism of slope failure, studying the failure mechanism of the deep pebble accumulation slope is necessary. Presently, some scholars have proposed to study the failure mechanism of slope by reducing cohesion and internal friction angle with different reduction factors.

According to the traditional strength reduction method, the attenuation ratio of cohesion to the friction angle are equal, and both are reduced by equal proportion of synchronous reduction coefficient through the sliding process \([1-2]\). This viewpoint has been questioned by many scholars fearing that this method cannot truly explain the failure mechanism of slope. In fact, the instability of slope is a developing process, and the strength parameters of rock and soil gradually change with this process \([3]\). Using double reduction coefficient can more realistically reflect the failure mechanism of slope \([4-8]\).

Based on the slice method, Tang Fen and Zheng Yingren proposed the double coefficient reduction method and deduced the mathematical relationship between cohesion and internal friction angle \([9]\). Yuan Wei et al. obtained the expression of safety factor of double reduction method by using data fitting method \([10]\). Zhu Yanpeng et al. used the concept of strength reserve area to deduce the relationship between safety factor and variations of shear strength parameters \([11]\). Xue Haibin et al. established the proportional relationship of two parameter reduction methods for the corresponding peak shear and residual strengths of soil \([12]\).

2 Theoretical study on slope failure mechanism

2.1 Calculation principle of traditional strength reduction method

In the traditional double strength reduction method, the cohesion and internal friction angle, which are the two indexes of soil shear strength, are reduced by the same coefficient. The reduced cohesion and internal friction angle replace the original shear strength index \([13]\):

\[ c_\text{m} = \frac{c}{m} \]  
\[ \varphi_\text{m} = \arctan\left(\frac{\tan \varphi}{m}\right) \]

\( c \) is the cohesion of soil; \( \varphi \) is the internal friction angle of soil, and \( m \) is the strength reduction factor. In the finite element calculation, the stability coefficient, when the slope reaches the critical failure status, is the strength reduction coefficient \( m \).

2.2 Double reduction mathematical model of cohesion and internal friction angle

The relationships between shear stress and normal stress, and shear stress and shear displacement are obtained by fitting the experimental data \( \tau - u, \sigma - u \) and \( \tau - \sigma \). The experimental process is described in detail later.

\[ \tau = N_\sigma \sigma + 2c\sqrt{N_\varphi} \] (3)

\( c \) is the cohesion of soil; \( \varphi \) is the internal friction angle of soil, and \( m \) is the strength reduction factor.
The constitutive relation of soil accords with Mohr Coulomb criterion. By combining equations (3) and (4), the curves of cohesion and shear displacement (c–u), and internal friction angle and shear displacement (φ–u) can be obtained.

3 Field shear test analysis

3.1 Test scheme

To simplify the analysis, the horizontal push method with the thrust direction parallel to the shear plane is adopted. The specific test procedures are as follows: (1) according to the preliminary analysis of the depth of the sliding plane and the maximum normal load perpendicular to the potential sliding plane, the horizontal push method is adopted. The estimated normal load range is 20–120 kPa. It is loaded in 6-level arithmetic order and tested at the same point. (2) When the shear deformation increases sharply or reaches 1/10 of the test block size, the test is terminated. To ensure the accuracy of the test, the shear displacement method is used to control the shear rate before the peak strength point, and the time control method is used to control the shear rate after the peak strength point. (3) By analyzing the test results, the peak strength, residual strength, and safety factors of slope, and the shear-stress–displacement curves of the same type of soil under different normal loads, are obtained. (4) The relationship between the experimental shear stress and the normal stress of each kind of soil in different shear deformation stages, is regressed by the least square method, to obtain the shear strength parameters (cohesion and internal friction angle) in different stages for slope stability analysis.

3.2 Experimental results and analysis

Three types of samples were randomly selected from the potential shear outlet of the slope as shown in Figure 1 for in-situ shear test, and the corresponding test groups are named as sample 1(breccia with broken pebbles), sample 2(gravel), and sample 3(silt mixed with crushed stone), respectively. The test results are shown in Figs. 2–4.

**Figure 1.** Sampling points of slope field test

**Figure 2.** Shear test data for sample 1 group
We found that the peak value of shear strength is observed between 6–8 mm, then, decreases to residual strength after 15–18 mm. Owing to the difference of soil properties in this area, the sample result curves show evident differences. The stress level of each type of soil layer under different shear displacements is calculated using the least square principle, and some measured values with large error deviation are discarded before calculation. To study the influences of strength parameters on slope state from stability to sliding, the stress indexes from peak strength to residual strength are taken for regression statistics by using formula (5) and formula (6). The regression results are shown in Figs. 6–8.

\[
\tan \phi(u) = \frac{n \sum \sigma_i u_i + \eta \sum \tau_i u_i - \sum \sigma_i \sum \tau_i}{n \sum \sigma_i^2 - (\sum \sigma_i)^2}
\]

\[
c(u) = \frac{n \sum \sigma_i u_i + \eta \sum \tau_i u_i - \sum \sigma_i \sum \tau_i}{n \sum \sigma_i^2 - (\sum \sigma_i)^2}
\]

Through the regression analysis, we found that cohesion and internal friction angle follow obvious reduction rules, and the reduction order and reduction ratio of the two parameters also have certain differences. The cohesion of pebble layer itself is low, and the statistical value of cohesion does not show an obvious decreasing trend, although it has a small range of fluctuations. However, the internal friction angle follows obvious reduction rule. The statistical data show that the different reduction order and ratio of the two parameters leads to considerable differences in the stability analysis.
Figure 6. Regression statistics of strength parameters in sample 2

Figure 7. Regression statistics of strength parameters in sample 3

4 Study on the mechanism of slope failure caused by double strength reduction

On performing data statistics, we found that the reduction of two parameters can be approximately expressed as a function, which can be imported into the software to analyze the slope stability. The fitting function and parameters are shown in Tables 1 and 2. The safety factors of the slope stability under different reduction curves were calculated. Fig. 9 is the window for setting strength parameter reduction of software. Figs. 10–16 are the fitting functions of shear strength parameters and the equivalent plastic strain results.

Table 1. Cohesion reduction fitting function and parameter selection

| Test soil | breccia with broken pebbles | gravel | silt mixed with crushed stone |
|-----------|-----------------------------|--------|-----------------------------|
| $C(u)$ (kPa) | $a \times \exp(b \times u)$ | $k/(1+\exp(a+b\times u))+c$ | $k/(1+\exp(a+b\times u))+c$ |
| a | 59.78 (50.44–69.11) | 4.608 ($-0.911$–$10.13$) | 10.98 (8.29–12.67) |
| b | $-0.068 (-0.092$–$-0.045)$ | $-0.884 (-1.82$–$0.057)$ | $-0.061 (-0.089$–$-0.042)$ |
| c | $-0.30$ | $11.41 (9.067$–$13.75)$ | $-$ |
| k | $-0.553 (-1.82$–$2.925)$ | $-6.96 (-9.691$–$-4.224)$ | $-$ |

Note: the values in brackets are the range of 95% confidence rate of fitting parameters.

Table 2. Internal friction reduction fitting function and parameter selection

| Test soil | breccia with broken pebbles | gravel | silt mixed with crushed stone |
|-----------|-----------------------------|--------|-----------------------------|
| $\tan \varphi(u)$ | $k/(1+\exp(a+b\times u))+c$ | $k/(1+\exp(a+b\times u))+c$ | $k/(1+\exp(a+b\times u))+c$ |
| a | $-0.919 (-11.59$–$9.756)$ | $-4.513 (-7.007$–$2.018)$ | $0.1357 (-3.40$–$3.68)$ |
| b | $0.301 (-0.840$–$1.44)$ | $0.619 (0.296$–$0.942)$ | $0.3415 (0.073$–$0.61)$ |
| c | $0.388 (0.095$–$0.682)$ | $0.2234 (0.181$–$0.266)$ | $0.429 (0.368$–$0.489)$ |
| k | $0.553 (-1.82$–$2.925)$ | $0.389 (0.307$–$0.471)$ | $1.556 (-1.796$–$4.913)$ |

Note: the values in brackets are the range of 95% confidence rate of fitting parameters.
Under the synchronous reduction, the plastic strain zone basically runs through the whole potential slip zone, and the safety factor is only 1.13 (Fig. 13). While observing the calculation results of asynchronous reduction parameter obtained from sample 2, the safety factor is increased to 1.22 (Fig. 15), and the plastic strain zone is only at the lower edge of the sliding zone of the slope. Compared with sample 2, the strength reduction rate of sample 1 is relatively closer to the synchronous reduction parameter, and the calculated safety factor is lower. The range of the plastic zone is also increased for sample 1, but it is still greater than the corresponding calculation results of synchronous reduction method. For sample 3, the differences between two parameter asynchronous reduction are more evident than that of sample 1, the reduction of the cohesion is smaller than sample1, the shear capacity is mainly borne by the mechanism of internal friction angle, and the plastic strain area is further smaller than that of sample 2, which is conducive to the stability of the slope.

Figure 8. Double strength reduction curve function imported into the midas GTS NX

Figure 9. Double strength reduction curve of sample 1

Figure 10. Double strength reduction curve of sample 2

Figure 11. Double strength reduction curve of sample 3

Figure 12. Cloud diagram of equivalent plastic strain of the traditional reduction method (f_os = 1.13)

Figure 13. Cloud diagram of equivalent plastic strain of sample 1 curve (f_os = 1.18)
In the actual slope retaining design, multistage retaining structures are often set up to prevent continuous sliding when the sliding belt is long. The current Chinese slope design code often fails to consider the reduction orders of strength parameter, and it seems too conservative in specific geological circumstances. For the pebble accumulation stratum, the calculation and analysis by using the test result curves of double strength reduction, and the range of equivalent plastic strain, can be intuitively and accurately defined.

5 Conclusion

1) The results from the field test of soil mass in pebble accumulation slope, show that the cohesion and internal friction angle of soil layer in sliding zone present nonsynchronous reduction discipline with the increase of shear displacement. The nonsynchronization varies with the properties of the soil layer. Combined with the mathematical methods, differences in the reduction rules of pebble stratum were analyzed in details, and the empirical formula of parameter reduction suitable for this kind of stratum, was obtained.

2) Using three groups of tests results, the stability of the slope was calculated in the finite element method program, and the stability safety factors were 1.18, 1.22, and 1.325, respectively, greater than the safety factor of 1.13 obtained by the traditional synchronous reduction method. Moreover, based on the analysis and calculation of the test parameters, it was observed that the calculation results using nonsynchronous reduction, can reflect the slope stability more fundamentally for pebble stratum; the strength reserve of the internal friction angle is much greater than that of the cohesive force, whereas the double reduction method after parameter correction, can better reflect the failure mechanism of slope.
3) The shear stress curve of the traditional strength reduction method changed linearly by contrast, the shear stress curves of the nonsynchronous reduction method changed nonlinearly and converged gently at the end of the calculation step (Fig. 16). Meanwhile, shear strain curve showed an increasing trend with the increase of the safety factor(Fig. 17). For different reduction ratios, different slope analysis results were produced, then, characterized by various changing trends of shear stress and shear strain.

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