

Research Progress in Methods for the Analysis of the Internal Stability of Landslide Dam Soils

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Abstract: In this paper, the research progress made in the methods used for assessing the internal stability of landslide dam soils was reviewed. Influence factors such as the gradation of soil and the stress state in the soil in different analysis methods were discussed, as these can provide a reference for the development of more accurate methods to analyze the internal stability of landslide dam soils. It focuses on the evaluation of internal stability based on the characteristic particle size and fine particle content, hydraulic conditions such as the critical hydraulic gradient and critical seepage velocity, and the stress state such as lateral confinement, isotropic compression, and triaxial compression. The characteristic particle size and fine particle content are parameters commonly used to distinguish the types of seepage failure. The critical hydraulic gradient or seepage failure velocity are necessary for a further assessment of the occurrence of seepage failure. The stress state in the soil is a significant influence factor for the internal stability of natural deposited soils. Although various analysis methods are available, the applicability of each method is limited and an analysis method for complex stress states is lacking. Therefore, the further validation and development of existing methods are necessary for landslide dam soils.

Keywords: landslide dam soils; internal stability; critical hydraulic gradient; seepage failure; stress states



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1. Introduction

Landslide dams are a natural deposition formed by unstable geological bodies such as landslides and debris flow that block rivers and streams [1–5]. These dams are characterized by high permeability, strong heterogeneity, and a wide range of gradations [6]. Figure 1, created by the authors using data from reference [7], shows the global distribution of landslide dams, with China having 758 landslide dams, accounting for 59% of the global total. Sichuan Province is a high-incidence area for landslide dams in China, with two notable events occurring in Die Xi Town, Mao County, Sichuan, in 1933 and 2017 [8,9]. The Wenchuan earthquake in 2008 triggered extensive landslides, resulting in the formation of 256 landslide dams [10]. From October to November 2018, the Jinsha River at the Sichuan–Tibet border experienced two consecutive landslide damming events within a month [11].

The breach of a landslide dam can cause incalculable damage downstream [12,13]. For example, the breach of a landslide dam in Die Xi Town, Mao County, Sichuan, on 9 October 1933, resulted in the death of over 2500 people downstream [14]. This underscores the necessity for research on the safety of landslide dams.

The main failure modes of landslide dams include overtopping, internal erosion, and slope instability [15]. Statistical data indicate that internal erosion is the second most common failure mode after overtopping [7,16]. Due to the heterogeneity of landslide dams and their naturally loose accumulation state, the likelihood of internal erosion is

high. Some studies have shown that internal erosion caused by seepage is one of the main reasons for landslide dam failures [17–20]. Therefore, neglecting the issue of internal erosion in landslide dams makes it difficult to understand the true causes of dam breaches or instability, especially when internal erosion is accompanied by overtopping.

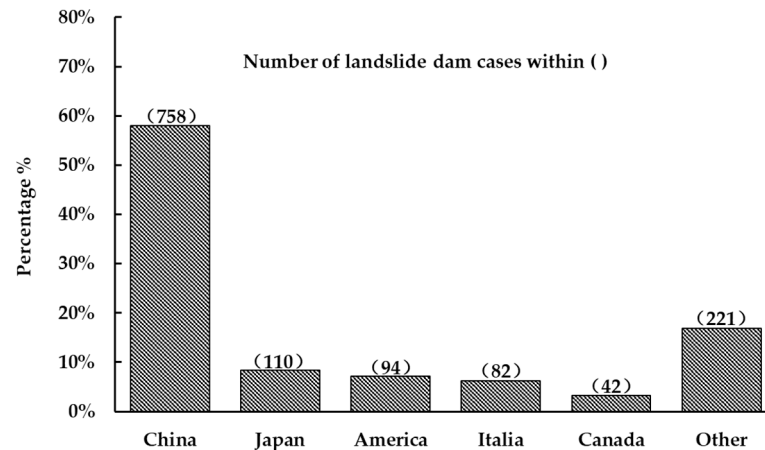


Figure 1. Global distribution of landslide dams.

Although existing research on the internal erosion of soils provides a theoretical foundation for understanding the process, most studies do not consider the impact of stress on the internal stability of the soils. In practice, the stress in landslide dams is primarily caused by their own weight, and the stress states in different parts of the dam are different. Ignoring the impact of stress conditions on internal stability may lead to a misunderstanding of the mechanisms of landslide dam instability [21].

In this paper, influence factors such as the gradation of soil and the stress state in the soil using different analysis methods were discussed, as these can provide a reference for the development of more accurate methods to analyze the internal stability of landslide dam soils. The influence factors, that is, the characteristic particle size, fine particle content, hydraulic conditions such as the critical hydraulic gradient and critical seepage velocity, and the stress state such as lateral confinement, isotropic compression, and triaxial compression, in the assessment of internal stability are summarized. This can provide a reference for the further validation and development of existing methods to assess the internal stability of landslide dam soils.

2. Qualitative Identification of Internal Stability Based on Physical Properties of Soils

The physical properties that affect the internal stability of soil under seepage include the gradation, particle shape, and pore distribution. Some scholars have proposed identification methods to qualitatively evaluate the internal stability of soils through experiments [22–25]. The gradation of soils is a key factor in assessing their internal stability [26,27], and it is usually characterized by the characteristic particle size. The fine particle content, as an important indicator for evaluating gradation, directly determines the degree of pore filling within the soil skeleton and is another crucial factor affecting the internal stability [28,29]. Therefore, the establishment of internal stability determination methods for soils often starts from these two aspects.

2.1. Identification Based on Characteristic Particle Size

Among these identification methods, representative ones include those proposed by Istomina, Kezdi, Kenney and Lau, and Burenkova, as shown in Table 1. The Istomina Identification method considers the coefficient of uniformity C_u as a control variable to determine the internal stability of soils [30]. For soils with $d_{10} > 0.5$ mm or $d_{60} > 10$ mm, most soils with $C_u < 10$ are stable, while those with $C_u > 20$ are unstable. However, for soil with $d_{10} < 0.5$ mm and $d_{60} < 10$ mm, some unstable soils are mistakenly identified as

stable [31]. Therefore, the Istomina Identification method exhibits large error in predicting the internal stability of soils.

Table 1. Identification methods considering characteristic particle size.

Researcher	Applicable Soils	Identification Method	Stability
Istomina [30]	Sand and gravel	$C_u \leq 10$ $10 \leq C_u \leq 20$ $C_u \geq 20$	Stable Moderate stable Unstable
Kezdi [32,33]	All types of soil	$\left(d_{15c}/d_{85f}\right)_{\max} \leq 4$	Stable
Kenney and Lau [34]	Non-cohesive soils	$(H/F)_{\min} \geq 1.0$	Stable
Burenkova [23]	Non-cohesive soils	$0.76\lg(h'') + 1 < h' < 1.86\lg(h'') + 1$	Stable

When analyzing the internal stability of the soils in a landslide dam, the Istomina method can only be used as a preliminary assessment method.

Analogous to the Kezdi criterion, the Kenney and Lau method is also specifically designed for non-cohesive soils. Kenney and Lau found that the internal stability of soils depends on the geometric shape of the gradation curve [22]. Particles with a diameter of d will be obstructed by particles with diameters between d and $4d$; if there are not enough particles in this range, particles smaller than d will be lost [34]. Accordingly, Kenney and Lau proposed using the $(H/F)_{\min}$ curve to evaluate the internal stability of soils, where H/F is the ratio of the mass percentage difference H of particles with diameters between d and $4d$ to the mass percentage F of particles smaller than d [35]. For broadly graded soils, when the fine particle content is less than 15%, the Kenney and Lau method has high accuracy [33]; when the fine particle content is greater than 15%, the Kenney and Lau method tends to conservatively classify stable soils as unstable [36]. Some scholars [37] have shown through experiments that the Kenney and Lau method is relatively conservative in assessing the internal stability of broadly graded coarse soils. Therefore, whether this method is suitable for assessing the stability of dam soils requires further experimental verification.

Based on internal stability tests of gravelly soils, Burenkova [23] proposed a method to analyze the internal stability of broadly graded gravelly soils using the indices $h' = d_{90}/d_{60}$ and $h'' = d_{90}/d_{15}$ (where d_{90} , d_{60} , and d_{15} are the particle sizes corresponding to 90%, 60%, and 15% of the total mass, respectively, on the gradation curve). The values of h' and h'' reflect the slopes of the gradation curve in two different intervals. The higher these values, the broader the particle size distribution of the soil [25]. If h' and h'' meet the corresponding geometric conditions in Table 1, the soil is internally stable [26]. Although the Burenkova method can be used to determine whether broadly graded soils have internal stability, it cannot accurately distinguish the boundary between stability and instability [34]. Thus, Wan and Fell [38] suggested replacing the original d_{90}/d_{15} with d_{20}/d_5 , believing that the modified Burenkova method improves applicability and effectively predicts the internal stability of broadly graded soils.

2.2. Identification Methods Based on Fine Content and Porosity

The fine particle content determines the soil's resistance to internal erosion [39–42]. Regardless of whether the soil is well-graded [25], an increase in the fine particle content can enhance the soil's resistance to water flow erosion. Specifically, when the fine content exceeds a certain threshold, the internal stability of the soil increases significantly [43]. Therefore, the fine particle content serves as a basis for assessing the internal stability of soils. Common identification methods of this type include the methods proposed by Changxi Mao, Jie Liu, and Dongsheng Chang, as shown in Table 2.

Table 2. Identification methods considering the fine particle content.

Researcher	Applicable Soils	Identification Method	Stability
Jie Liu [29]	Non-cohesive soils	$P_f \leq 0.9P_{op}$	Stable
		$0.9P_{op} < P_f < 1.1P_{op}$	Moderate stable
		$P_f > 1.1P_{op}$	Unstable
Changxi Mao [24]	Non-cohesive soils	$4P_f(1 - n) \geq 1$	Stable
		well-graded soil	
Dongsheng Chang [25]	Non-cohesive soils	$(d_{90}/d_{60})/(d_{20}/d_{10}) > 1.05$	Stable
		$P_f < 10\%$	
		$0.52 < \frac{d_{90}/d_{60}}{d_{20}/d_{10}} < 1.05$	Moderate stable
		$\frac{d_{90}/d_{60}}{d_{20}/d_{10}} < 0.52$	Unstable
		$10\% < P_f < 20\%$	
		$(H/F)_{\min} > -0.04P_f + 0.8$	Stable
		$P_f > 20\%$	Stable
		gap-graded soil	
		$P_f < 10\%$ and $G_r < 3.0$	Stable
		$10\% < P_f < 35\%$ and $G_r < 3P_f$	Stable
		$P_f > 35\%$	Stable

Note: P_{op} refers to the optimal fine content within the soil pores, as specified in the equation; d_{90}/d_{60} indicates the degree of non-uniformity among the coarse particles in the soil; d_{20}/d_{10} measures the non-uniformity among the fine particles; P_f is the mass percentage of fines that fill the voids in the coarse particles; n denotes the porosity of the soil; H is the mass percentage corresponding to any given particle size d in the soil; F represents the mass percentage for particles between any given particle size d and $4d$; G_r , the discontinuity ratio, is given by $G_r = d_{\max}/d_{\min}$, where d_{\max} and d_{\min} are the maximum and minimum particle sizes (in mm), respectively, within the discontinuity interval.

2.2.1. Theoretical Formula

Changxi Mao et al. [44] followed the Loebotsjkov theory, assessing the maximum particle size of the filling soil based on the pore size formed by the skeletal particles. A method for evaluating the internal stability of soils has been proposed. When the condition $4P_f(1 - n) \geq 1$ is met, the soil is considered internally stable. Here, P_f is the mass percentage of fine particles filling the coarse particle pores, and n is the porosity of the soil. The theoretical calculation results of Changxi Mao's method have been experimentally validated [42].

2.2.2. Semi-Theoretical and Semi-Empirical Formula

Jie Liu et al. [29] proposed that the fine particle content is the main factor assessing the internal stability of soils. When the internal stability of soil improves once the pores are filled with fine particles, this is known as the optimal fine content. Using empirical methods, a formula to determine whether internal erosion occurs based on the fine particle content was proposed, where P_{op} is the optimal fine content (%) under ideal conditions [29], as shown in Formula (1),

$$P_{op} = \frac{0.28 + 3n^2 - n}{1 - n} \times 100\% \quad (1)$$

2.2.3. Empirical Formula

Dongsheng Chang, building on the research of Kenney and Lau and Burenkova, classified soils into two categories: well graded and gap graded. Based on different fine particle contents, he further subdivided each soil type and developed geometric identification methods for internal stability that are suitable for different soil types [31]. This identification method targets broadly graded non-cohesive soils, which aligns well with the characteristics of dam soils. Therefore, this method is recommended for assessing the internal stability of dam soils.

In summary, both the characteristic particle size method and the fine content method are gradation-based methods for assessing the internal stability of soils. The characteristic particle size method focuses on assessing certain key particle sizes within the soil, while the fine content method focuses on the proportion of fine particles within the soil. These two types of identification methods provide some references for further research on geometric identification methods for the stability of dam soils. However, not all the above methods are suitable for identifying the internal stability of dam soils, and further validation is required.

3. Hydraulic Criteria

Hydraulic criteria are critical factors in assessing whether internal erosion will occur in soil under seepage conditions. These assessment criteria are usually derived from the force balance analysis of soil microelements. The critical hydraulic gradient and critical seepage velocity are two hydraulic parameters commonly used to establish these criteria [24,44–46]. Therefore, hydraulic conditions must always be considered in all seepage failure issues to ensure an accurate evaluation of internal soil erosion.

The hydraulic gradient criteria are typically derived through force balance analysis, focusing on changes in the effective stress of soil particles under seepage. When water flows through soil, seepage force will affect the effective stress of the soil particles. The seepage velocity criterion is derived through the principle of energy balance, primarily considering the impact of seepage kinetic energy on soil particles.

3.1. Criteria Based on Critical Hydraulic Gradient

The formula used to calculate the critical gradient proposed by Jinxuan Sha [47], Liangji Wu [48], Changxi Mao [24], and Jie Liu [29,49] are all based on force equilibrium, as shown in Table 3.

Table 3. Methods for hydraulic identification.

Researcher	Applicable Soils	Identification Method
Terzaghi [50]	Non-cohesive soils	$i_c = (G_s - 1) / (1 + e)$
Liangji Wu [48]		$i_c = (G_s - 1) \frac{d}{d_{\Theta K} + e d_{\Theta K}}$
Jinxuan Sha [45]		$i_c = (G_s - 1) \frac{d}{d_{\Theta K} + e d_{\Theta K}} \tan \varphi$
Jie Liu [49]		$i_c = (G_s - 1)(1 - n)\alpha d / d_{\Theta K}$
Changxi Mao [24]		$i_c = (G_s - 1)(1 - n)\alpha d \tan \varphi / d_{\Theta K}$
		$i_c = 2.2(1 - n)^2 (G_s - 1)$
		$i_c = C(G_s - 1) \frac{d}{d_f} [4P_f(1 - n)]^2$
Skempton and Brogan [42]	Sand and gravel soil	$i_c = \beta(G_s - 1)(1 + e)$

Note: G_s is the specific gravity of the soil; e is the void ratio; d is the particle diameter of the eroded particles; $d_{\Theta K}$ is the equivalent particle diameter; φ is the internal friction angle of the soil; α is the particle shape coefficient; β is the reduction coefficient; C is the coefficient; and d_f is the maximum particle diameter of the filler.

In the formulas proposed by Jinxuan Sha [47] and Liangji Wu [48], the only difference between them is the shape coefficient α . Jinxuan Sha [47] primarily referenced hydraulic principles during the derivation process, particularly the effects of dynamic water pressure on particles and the correction of the shape coefficient. Liangji Wu [48], on the other hand, focused more on a comprehensive analysis of fluid mechanics and particle mechanics, performing detailed analyses of dynamic water pressure and friction. The introduction of the shape coefficient α is intended to mitigate the impact of the particle shape on seepage resistance. The porosity n influences the size of the channels through which seepage flows through the soil. The equivalent particle size $d_{\Theta K}$ represents the diameter of hypothetical uniform particles with the same permeability characteristics, determined experimentally to accurately calculate the effects of seepage. The particle diameter d of eroded particles reflects the soil's resistance to erosion under seepage, and can be used to predict which

particles are most likely to be carried away during the seepage process. By integrating these parameters, the formulas can more accurately describe and predict the behavior of soil under seepage conditions.

Skempton and Brogan [42], addressing the issue of the significant loss of fine particles in unstable gravel long before reaching the critical hydraulic gradient, introduced a reduction coefficient β to modify Equation (1) based on experimental data and theoretical formulas. The reduction coefficient $\beta = i_c / (r' / r_w)$, where i_c is the critical hydraulic gradient at which piping occurs in gravel, and r' / r_w is the ratio of the buoyant unit weight of gravel to the unit weight of water.

Changxi Mao et al. [24] considered the parameters of the diameter of eroded particles d_f , the porosity n , and the fine content filled in pores to comprehensively describe the movement behavior of movable particles under the action of water flow. This formula, based on classical mechanics principles, reasonably explains the particle behavior and enhances applicability by converting the critical flow velocity to a seepage gradient. The experimental calibration coefficient in the formula improves the prediction accuracy, making it suitable for various soil conditions. However, the empirical coefficient C in the formula depends on experimental calibration, adding complexity to its application. The formula assumes ideal particle shapes and ignores interactions between particles. Additionally, the difficulty of accurately measuring porosity and the fine particle content in actual engineering affects its application and prediction capability, and the formula may fail under extreme conditions, requiring further validation for its applicable range.

Jie Liu et al. [49,51] generalized soil as an ideal body with an equivalent particle size of d_{20} , considering factors such as the porosity n , specific gravity G_s of soil particles, and particle size distribution. The formula is simple and easy to use, is suitable for most uniform soils and stable seepage conditions, and has been validated by numerous experiments and engineering practices. However, in practical applications, it is necessary to be aware of the differences between these idealized conditions and actual situations, and to make necessary adjustments and corrections.

3.2. Criteria Based on Critical Velocity

In addition to using the critical hydraulic gradient to assess the internal stability of soil, the critical seepage velocity was also used in some criteria especially when dealing with coarse-grained soils [52,53]. Some commonly used criteria are

$$v_{cr} = \frac{\rho' g d K}{18 \mu \alpha \rho_w K + \rho_w g n d^2} \quad (2)$$

$$v_{cr} = \frac{1}{2} n \frac{\frac{\sqrt{3} d^2}{9} (\rho_s - \rho_w) g}{\frac{4}{n} \mu_w \left(\frac{d}{d_0} \right)^2 + \frac{3 \mu_w}{2}} \left| \sin \left(\theta + \frac{\pi}{6} \right) \right| \quad (3)$$

In Equations (2) and (3), v_{cr} is the critical velocity, ρ_w is the density of water, g is the gravitational acceleration, d is the movable particle diameter, and n is the porosity of the soil.

In Equation (2), α is the correction factor of the particle shape, K is the hydraulic conductivity, ρ' is the effective density of fine particles, and μ is the dynamic viscosity of water. In Equation (3), ρ_s is the density of soil particles, μ_w is the dynamic viscosity, d_0 is the minimum diameter of the pore channel, and θ is the inclination angle of the pore channel.

The critical velocity formulas for internal erosion in non-cohesive soils were derived by Huang [52] and Zhao Zhengxin [53], with Formula (2) derived by Huang and Formula (3) derived by Zhao. Both derivations neglect the non-uniformity of pore channels, the water flow turbulence effects, and transient phenomena, but the handling details differ. Zhao's approach uses a variable cross-sectional pipe model and spherical particle assumptions to simplify the physical model, mainly describing the forces and movement of particles under laminar flow conditions through force balance equations and the Poiseuille equation,

making it suitable for practical engineering applications. In contrast, Huang's approach uses a probabilistic distribution model of pore sizes and detailed probabilistic analysis methods, combining force balance equations and momentum equations to detail the movement trajectories and conditions of particles in complex pore structures, making it suitable for complex seepage conditions. Although more complex, Huang's formula can more accurately reflect the actual conditions.

The hydraulic gradient criterion is established through the force balance analysis of soil microelements, focusing on changes in the effective stress of soil particles under seepage action. When water flows through the soil, the effective stress of the particles is influenced by the seepage force, leading to the derivation of the critical hydraulic gradient. Conversely, the permeability flow velocity criterion is based on mechanical equilibrium, utilizing the torque balance or dynamic equation. At a certain critical flow velocity, the combined drag force and buoyancy acting on the particles exceed the particle gravity and friction, causing movement and thus deriving the critical seepage velocity. In both criteria, the seepage force plays a key role in assessing the critical movement conditions of particles. The hydraulic gradient criterion emphasizes changes in the internal stress distribution within the soil, while the permeability flow velocity criterion focuses on the direct transmission of kinetic energy and force to the particles.

4. Criteria Considering Stress Conditions

Establishing internal stability discrimination methods that consider stress conditions not only accurately reflects the internal stability of soils, but also predicts the internal stability patterns, providing a reference for practical engineering. The stress conditions considered in internal stability discrimination methods mainly include confined compression, isotropic compression, and triaxial shear.

4.1. State of Confined Compression

Internal erosion under confined compression refers to the phenomenon of internal erosion occurring in soils under vertical load and lateral confinement. Stress levels significantly influence the critical hydraulic gradient of soils; the higher the stress level, the greater the corresponding critical hydraulic gradient. This is because the stress level affects the contact degree between coarse and fine particles. The higher the stress level, the greater the contact area and number of contact points between coarse particles, leading to a more stable skeletal structure. As a result, soil particles are less likely to be lost from the skeleton under the action of seepage.

Zhongming Jiang et al. [54] conducted internal erosion tests on gravelly soil under confined conditions with vertical stresses of 0.1, 0.3, 0.6, and 0.9 MPa. By analyzing the mechanical equilibrium of individual particles, a critical hydraulic gradient calculation formula that considers the friction between particles was derived.

$$i_c = (1 + 0.5K_0 \tan \alpha') \frac{\gamma'}{\gamma_w} + (1 + K_0 \tan \alpha') \frac{\sigma_s}{\gamma_w} \quad (4)$$

In the Formula (4), K_0 is the coefficient of the lateral earth pressure between soil particles, γ' is the self-weight stress of soil particles, α' is the effective internal friction angle of the soil, and σ_s is the additional stress acting on the particles. It can be seen from Formula (4) that the change in additional stress acting on the soil directly affects the change in the critical hydraulic gradient. Both experimental and theoretical studies have shown that the critical hydraulic gradient of soil increases linearly with the increase in stress level [54].

It is evident that a larger vertical stress increases the density of the soil, reduces porosity, and increases the initiation and failure hydraulic gradient of the soil [55–57], thereby enhancing the soil's erosion resistance. For landslide dams, the soil along the dam axis is in a confined compressive state due to gravity and lateral constraints, making it an ideal area for studying internal erosion behavior. Understanding the stress conditions

under which internal erosion is more likely to be triggered in landslide dam soil helps in comprehending the stability and erosion mechanisms of landslide dam soil [57].

4.2. Isotropic Pressure State

Isotropic pressure condition internal erosion refers to the phenomenon where internal erosion occurs in soil when it is subjected to equal stress in all directions. Under the influence of seepage, the magnitude of confining pressure affects the ability of water to pass through the soil pores [58,59]. For example, when the confining pressure applied to the soil sample is low, an increase in the confining pressure reduces the sample volume, decreases porosity, and lowers the permeability coefficient [60]. Consequently, the critical hydraulic gradient of the soil increases with the increase in the confining pressure [25,61–63].

Researchers have already obtained the relationship curve between the confining pressure and the critical hydraulic gradient through data fitting. For instance, Luo Yulong et al. [64] conducted five groups of experiments using an improved triaxial permeameter on discontinuously graded gravelly soil with confining pressures of 0, 0.2, 0.4, 0.6, and 0.8 MPa. A clear linear relationship between confining pressure and the critical hydraulic gradient was found, as shown in Formula (5).

$$i_c = 0.82P + 0.204 \quad (5)$$

In the equation, P represents the confining pressure applied to the soil sample. Although different scholars have fitted various forms of the relationship curve between the confining pressure and critical hydraulic gradient [25,64], the starting hydraulic gradient of the soil increases with the increase in the confining pressure, as shown in Figure 2.

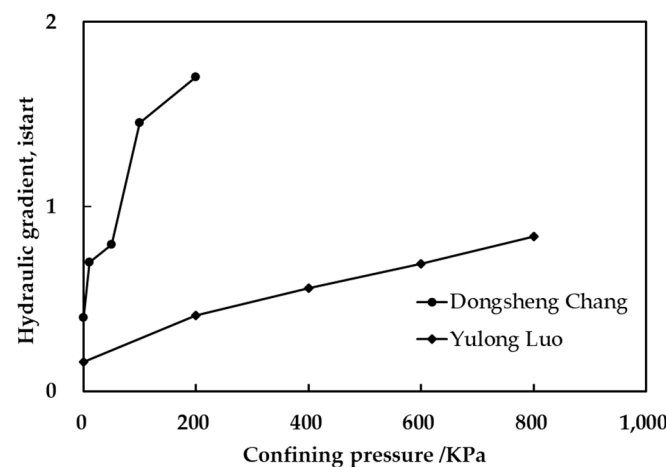


Figure 2. Effect of confining pressure on hydraulic gradient that initiates internal erosion [25,64].

However, under a high confining pressure, the stable structures formed between particles are more prone to failure or collapse, making the soil structure unstable. Even a small hydraulic gradient can cause internal erosion in the soil [61]. Under high hydraulic gradient conditions, the loss of fine particles may lead to the collapse of the original force transmission structure [65]. When the soil fails, the fine particles that originally provided lateral support to the coarse particles in the skeleton are lost, resulting in severe deformation of the soil skeleton [25]. For soils with a fine particle content at or above the critical value, the loss of fine particles further damages the force transmission structure, causing more fine particles to become unstable and easily lost [66].

Therefore, it is necessary to conduct internal stability tests on dam soils under high stress conditions to comprehensively assess the impact of the stress state on the internal stability of the soils.

4.3. Triaxial Compression State

In nature, soils are often subjected to triaxial stress states. To realistically simulate the internal stability of soil under complex stress conditions, researchers use an improved triaxial permeameter to simulate the stability behavior of soil under these conditions.

The stress ratio, shear stress ratio, and deviatoric stress are used to reflect the relationship between the major and minor principal stresses applied to the sample [67–69]. These are often correlated with the hydraulic gradient and volumetric deformation to reflect the influence of seepage and stress coupling on the internal erosion of soil. Some researchers [68] have found through experiments that when the shear stress ratio is less than the critical value, the starting hydraulic gradient of the soil increases with the increase in the shear stress ratio, as shown in Figure 3. Similarly, other researchers [69] have found similar situations through experiments, as shown in Figure 4. However, some researchers [70] have found that the relationship between the shear stress ratio and the critical hydraulic gradient is piecewise linear. This is because the porosity reaches a minimum value near the critical shear stress ratio, but suddenly increases after the shear stress ratio slightly exceeds the critical value [70], as shown in Figure 5.

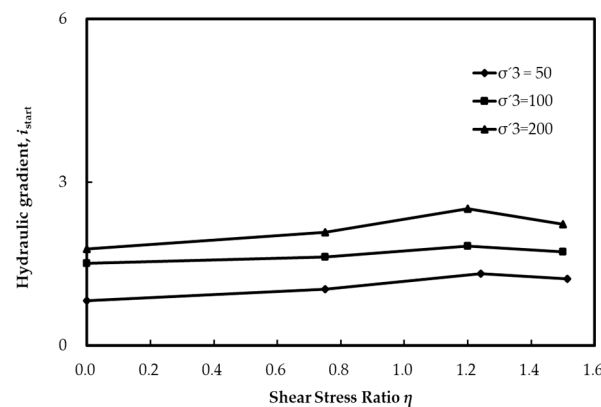


Figure 3. Effect of shear stress ratios on hydraulic gradient that initiates internal erosion [68].

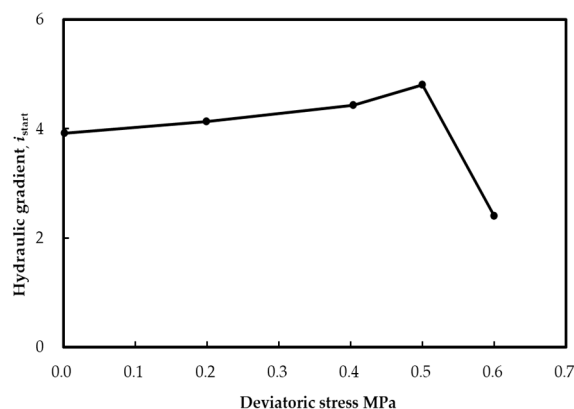


Figure 4. Effect of deviatoric stress on hydraulic gradient that initiates internal erosion [69].

In Figures 3 and 4, the initial increase in the shear stress ratio or deviatoric stress leads to an increase in the critical hydraulic gradient of the soil. This indicates that when the shear stress ratio or deviatoric stress applied to the sample does not reach the critical value, the soil's density and stability continue to increase. This means that a higher hydraulic gradient is required to induce internal erosion in the soil. Once the shear stress ratio or deviatoric stress exceeds the critical value, the hydraulic gradient in both figures shows a significant decrease. This is because excessive deviatoric stress may lead to the weakening of the soil structure and an increase in porosity, possibly forming shear bands or elongated

pores, which reduce soil stability. Fine particles are more easily carried away by water flow, forming stable seepage channels, thereby reducing the starting hydraulic gradient.

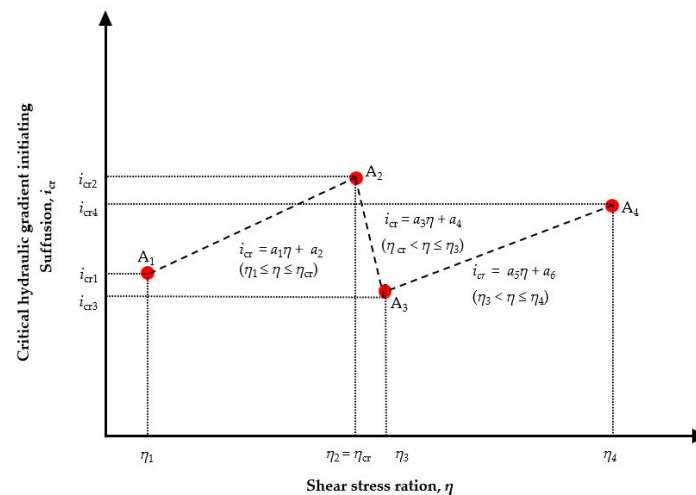


Figure 5. Critical starting gradient under different stress ratios [70].

Additionally, there is a complex interrelationship between the stress ratio, volumetric change (dilatancy or contraction), and particle loss [71–73]. Under low shear stress ratio conditions, the frictional force between soil particles is relatively small, making particles more prone to relative sliding and dispersion. In this case, as stress increases, the soil sample exhibits dilatant behavior [74]. During dilatancy, the soil becomes loose, internal pores increase, interparticle connections weaken, and particles are more easily carried away by water flow, leading to increased particle loss. Increased particle loss raises the void ratio of the soil, reducing its density, making the soil looser, and increasing the deformation potential. Therefore, after a significant loss of fine particles in the soil sample, the sample transitions from dilatancy to contraction [72,75–78], thereby altering the stress distribution in the soil test [78]. However, under high shear stress ratio conditions, the contact force and frictional force between soil particles increase, leading to the tighter packing of particles within the sample, resulting in contraction [79]. In this process, the soil becomes denser, internal pores decrease, particles are locked within the soil, and particle loss is reduced.

4.4. Discussions for the Influence of Stress State

Although existing studies have considered the impact of the stress state on the internal stability of soil, the stress levels are relatively low. Taking the soil element on the dam axis as an example, the stress level increases with depth. For landslide dams, the dam heights commonly range from tens of meters to over a hundred meters [1], with stress levels in some areas possibly exceeding 1 MPa. Therefore, the stress levels used in the afore-mentioned tests are insufficient for the actual conditions of a landslide dam. Furthermore, existing methods for assessing the seepage stability do not involve triaxial compression stress states, and thus cannot fully reflect the true conditions of landslide dam soil.

To compensate for the limitation of indoor experiments in observing the microscopic structural changes in soils, scholars [80–82] have used numerical methods to simulate the continuous changes in the microstructure within a sample during seepage. Currently, mesoscopic simulations are mainly conducted to verify the geometric(gradation) and hydraulic criteria. Some scholars [83–85] have simulated the internal seepage erosion process in soil samples by changing the soil gradation and fine particle content. Some scholars [86–88] have simulated the internal erosion process in samples. However, most studies focus on isotropic stress states, and fewer studies focus on complex stress states such as triaxial compression stress states.

Furthermore, stress anisotropy in dam soil has an impact on the seepage erosion in soil. It is necessary to conduct studies on the influence of stress anisotropy on internal erosion.

Studies have shown that under the anisotropic stress state, the internal erosion behavior of soil differs from that under the isotropic stress state. For instance, when the hydraulic gradient is large and the cumulative erosion reaches a critical value, significant volumetric deformation and collapse occur in the sample under anisotropic stress. While under isotropic stress, the sample remains stable without structural failure [89]. Additionally, anisotropic stress state may lead to stress concentration in the direction of the deviatoric stress state, but may also cause more deformation in that direction [90]. Furthermore, when the seepage direction is parallel to the maximum principal stress, the sample is more prone to seepage erosion, with a significant loss of fine particles and volume shrinkage [65].

5. Conclusions and Prospects

5.1. Conclusions

Based on the comprehensive analysis above, the conclusions are as follows:

- (1) The qualitative identification methods used to assess internal stability based on the physical properties of soils have two types. One type of method is based on characteristic particle sizes such as d_{10} , d_{60} , d_{15} and d_{85} . The other method is based on the degree of filling by fine particles, and includes theoretical formulas, empirical formulas, and semi-theoretical semi-empirical formulas.
- (2) The parameters of the hydraulic criteria used to assess internal stability mainly include the critical hydraulic gradient and critical seepage velocity. These criteria focus on the balance and movement of fine particles, and corresponding formulas are derived based on force balance. The difference lies in their starting points and expressions.
- (3) The criteria used to assess internal stability considering the stress state include three categories: the lateral confined state, isotropic compression state, and triaxial compression state. Under the lateral confined and isotropic compression states, the greater the vertical or confining pressure, the less potential there is for internal erosion. Under a high confining pressure, particle loss can lead to the collapse of the soil skeleton. The triaxial compression state can simulate the internal erosion of soils under complex stress states in practical engineering.

5.2. Prospects

- (1) Currently, most internal erosion tests under the triaxial compression state use vertical seepage, which does not match the actual situation in landslide dams where horizontal seepage is predominant. Therefore, it is necessary to conduct internal erosion tests of landslide dam soils under horizontal seepage conditions.
- (2) At present, there is limited research on the internal stability of landslide dam soils. The existing criteria used to assess the internal stability of these soils are lacking, especially under the complex stress state. Therefore, it is necessary to conduct laboratory tests considering the coupling of seepage and stress, and to establish methods to assess the internal stability of landslide dam soils based on particle initiation under the complex stress state.
- (3) One major issue with landslide dam soils is their natural vertical and horizontal heterogeneity. Landslide dams are usually formed by landslides or debris flows. During the deposition process, these soils are subjected to complex stratification and mixing, resulting in significant differences in their physical properties (such as particle size, density, porosity, and permeability) in the vertical and horizontal directions at the same location. This heterogeneity makes the evaluation and prediction of the seepage behavior and stability of dam soils difficult. Therefore, it is necessary for the differences in the physical properties of landslide dam soils to be thoroughly discussed to comprehensively assess and understand their internal erosion characteristics.

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