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Permeability evolution of compacted clay during triaxial compression

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Abstract: Triaxial seepage tests were conducted on the core-wall clay of a high rock-fill dam to investigate the change of hydraulic conductivity with axial strain. It was found that the compaction density and confining pressure were the two main factors influencing the change of hydraulic conductivity during triaxial compression. When the current confining pressure was greater than the pre-consolidation pressure of a compacted specimen, the specimen kept being compressed and became denser during the compression process, resulting in a decreasing trend in its hydraulic conductivity until it eventually reached a stable state. On the other hand, if the current confining pressure was far less than the pre-consolidation pressure of the specimen, the specimen deformed in a localized shear band which weakened the impermeability of the specimen, and as the shear band continued to dilate, an increasing trend in the overall hydraulic conductivity was observed. This study highlighted an important fact that heavily compacted clay under low confining pressures had a high susceptibility to localized shear bands of high permeability, which could be used to explain many historical leakage problems observed in clay-core dams.

Ke words: triaxial seepage test; hydraulic conductivity; strain localization; shear-induced leakage band

1 Introduction

China has built several 300-meter super-high dams: Nuozhadu (261.5 m), Lianghekou (295.0 m), Shuangjiangkou (312.0 m) and Rumei (315.0 m)，all of which are clay core-wall dams. The clay core-wall is the key part to the seepage-proofing of a high rock-fill dam [1]. It is required in the design that the core-wall should have a high degree of compaction to ensure higher modulus, strength, lower permeability and higher ability to resist seepage damage. When evaluating the permeability of clay core, a traditional permeameter (such as Nan-55[2]) is generally used to measure the hydraulic conductivity of a sample made according to the design compaction density[1]。In the process of earth dame construction, the dam will undergo deformation, which will cause the change of density and structure of the compacted clay, and result in the change of hydraulic conductivity and the ability to resist seepage failure of clay core. The distribution of density is non-uniform, and it is unreasonable to evaluate the seepage safety of the dam only by using the same hydraulic conductivity measured by the sample of the initial compaction density. Therefore, it is of great significance to study the variation of permeability characteristics of the core-wall (especially those in contact with embankment fills) during deformation[3-4]。

Triaxial compression test is the most commonly used method to study the stress-strain characteristics of core-walls, and it is also a commonly used method to measure the hydraulic conductivity. It can easily prevent side wall leakage in conventional seepage test, but also simulate different stress states. Zhu[5]conducted triaxial stress-controlled seepage tests on the undisturbed soil of the core-wall of the Chaihe dam. He studied the permeability and permeability anisotropy of the core soil with the change of stress conditions, and found that the permeability of the core soil decreased with the increase of effective stress. Lei et al. [6] conducted the seepage tests on the core-wall clay in the process of triaxial compression by using the improved triaxial penetration test device, and analyzed the effect of axial strain, confining pressure and seepage pressure on the hydraulic conductivity. The main conclusion from their research is that the increase of confining pressure and shear deformation will lead to the decrease of hydraulic conductivity. The above test results seem to indicate that the permeability of the core-wall will not decrease, on the other hand, it might increase after considering the actual deformation of the core-wall. However, Wang[7-10], Wei [11], et al. have successively developed torsional shear seepage test devices and ring shear seepage test devices, and observed the change of permeability of core-wall clay during large shear deformation. The results indicate that the change of permeability of the compacted clay shear band is related to the pressure acting on it. When under higher pressure, the permeability of the shear band decreases with the increase of shear deformation; at low consolidation pressure, however, the permeability of the shear band increases sharply. Based on this observation, they believed that the shear

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leakage zone of compacted clay under low consolidation pressure is the main cause of dam leakage during initial impoundment.

Since the above-mentioned torsional shear and ring shear test devices[7-8] restrict the large shear deformation to a thin concentrated shear band, but the boundary constraint is not strong in the actual dam core wall, will it produce the shear permeability zone in the test device? In this paper, the permeability test of a core-wall clay during triaxial compression is performed, and the development of the stress-strain curve, deformation form and hydraulic conductivity of the sample is investigated. We further explained that even in the process of triaxial compression, the compacted clay sample may produce high permeability shear band, and the stress conditions and engineering significance of producing high permeability shear band are preliminarily discussed.

2 Test method

The principle of triaxial seepage test is shown in Fig.1. The saturated sample is placed in a triaxial pressure chamber. If the initial height of the sample is h_0 and the initial diameter is d_0, then the initial cross-sectional area is $A_0 = \pi d_0^2 / 4$. After the sample is isotropically consolidated under the confining pressure of σ_3, increase the axial pressure to σ_1, the sample is compressed to the axial strain, then the height h of the sample *h* is:

$$h = h_0(1-\varepsilon_1) \quad (1)$$

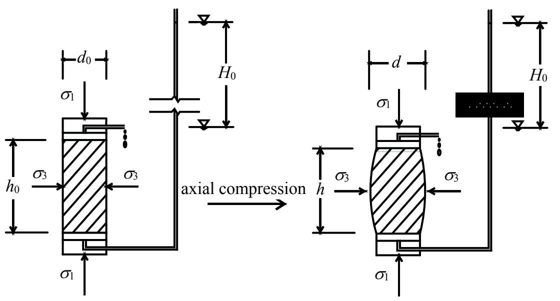

Fig.1 Schematic diagram of triaxial seepage test

If the bulk strain is ε_v, and the average diameter of the sample becomes *d*, the average cross-sectional area A of the sample is:

$$A = \pi d^2 = A_0(1-\varepsilon_v)/(1-\varepsilon_1) \quad (2)$$

In order to observe the change of the hydraulic conductivity of the sample with the axial strain, when shearing to the specified axial strain, stop shearing, adjust the liquid level height of the upper and lower drainage pipes, and form the water head H_0 between the upper and lower drainage pipes, thus the seepage water enters the sample from the lower drainage pipe and is discharged from the upper drainage pipe. Because the hydraulic conductivity of clay is very small, the variable water level method is used to measure the hydraulic conductivity of the sample. If the liquid level drop of the water inlet pipe is ΔH in the period of Δt, the calculation formula of the axial hydraulic conductivity *K* of the sample is as follows:

$$k = \frac{a h_0(1-\varepsilon_1)}{A_0(1-\varepsilon_v)/(1-\varepsilon_1)\Delta t} \ln \frac{H_0}{H_0 - \Delta H} \quad (3)$$

where *a* is the internal sectional area of the measuring tube. In this way, the change process of the hydraulic conductivity with the axial strain can be obtained by the seepage test.

3 Materials and experimental methods

The soil used in the test is the proposed core-wall clay material obtained from a high rock-fill dam, and its basic geotechnical properties are shown in Table 1. The clay is lean clay with the maximum dry density $\rho_{max} = 1.74$ g/cm^3 and the optimum water content w_{opt}=18.2% under the standard compaction energy.

Table 1 Geotechnical preperties of the tested soil

Liquid limit w_L/%	Plastic limit w_P/%	Relative density of particle G_s	Optimum water content w_{opt}/%	Maximum dry density ρ_{dmax}/(g • cm^{-3})
37.1	15.3	2.72	18.2	1.74

Triaxial compression test and axial seepage test were carried out with GDS unsaturated soil triaxial apparatus. The equipment includes a double-layer pressure chamber, which can not only measure the volume change of the sample through the volume of water discharged from the saturated sample, but also measure the volume change of the sample according to the water level change of the inner pressure chamber, therefore, ensure the high accuracy of the measured volume change. The initial height of the sample $h_0 = 8.0$ cm, and the initial diameter $d_0 = 3.91$ cm.

Degree of compaction has a great influence on the mechanical properties and permeability of compacted clay. Two kinds of compaction density are selected to prepare the sample: $\rho_d = 1.74$ g/cm^3, i.e. the maximum dry density under the standard compaction energy, and the corresponding compaction degree is 100%; $\rho_d = 1.58$ g/cm^3, which is a smaller dry density. The microstructure and mechanical properties of compacted clay are related to the compacted water contents[12]. The water content of the samples with two compacted densities was 18.2%, which is the optimal water content under standard compaction energy, and the samples with different dry densities are obtained by applying different compaction energy (blow counts) in the process of stratified sample preparation.

Preconsolidation pressure is an important index to characterize the mechanical properties of compacted clay. Different compaction density corresponds to different preconsolidation pressure. Fig.2 shows the isotropic consolidation compression curves of the samples with two compaction densities, according to which the preconsolidation pressure p_0 of the samples can be roughly estimated by Casagrande method. When $\rho_d = 1.74$ g/cm^3, $p_0 \approx 100$ kPa; when $\rho_d = 1.58$ g/cm^3, $p_0 \approx 25$ kPa.

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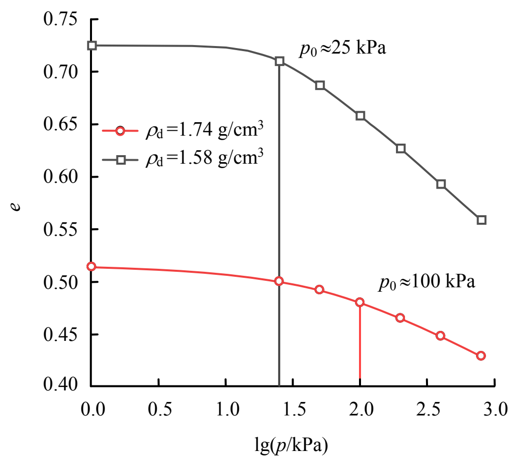


Fig.2 Compression curves of samples with two different densities

In order to analyze the influence of the consolidation pressure p on the change of the permeability characteristics of the compacted clay, four confining pressures (25, 50, 100, and 200 kPa) were selected for testing. Firstly, the sample is isotropically consolidated under the corresponding confining pressure, then it is compressed axially under the drainage condition, and the seepage test is carried out in stages during the compression process (the initial head difference of the upper and lower drainage pipes $H_0 = 100$ cm). Therefore, before the start of axial compression, the sample of $\rho_d = 1.74$ g/cm^3 is in the state of overconsolidation under 25、50 kPa confining pressures, while the sample of $\rho_d = 1.58$ g/cm^3 is in the state of normal consolidation under four confining pressures.

4 Hydraulic conductivity change

Fig.3 shows the change of hydraulic conductivity k versus axial strain of samples at a compaction density of $\rho_d = 1.58$ g/cm^3 under different consolidation pressures. It can be seen that: (a) the larger the confining pressure, the smaller the initial hydraulic conductivity of the sample, because the density of the sample after consolidation increases with the increase of confining pressure, and the pores in the sample are smaller; (b) in the axial shear process under four confining pressures, the hydraulic conductivity of the sample decreases with the increase of the axial strain, however, the rate of decrease gradually slows down, and finally tends to be stable; (c) the difference of the final hydraulic conductivity under each confining pressure is much smaller than the difference of the initial hydraulic conductivity. The behaviors observed in Fig.3 is consistent with the variation characteristics of hydraulic conductivity found by Zhu[5] and Lei[6].

The test results of the sample of $\rho_d = 1.74$ g/cm^3 are given in Fig.4. The behaviors are slightly different from those observed in Fig.3. As can be seen from figure 4, the hydraulic conductivities under low confining pressures (25 and 50 kPa) and high confining pressures (100 and 200 kPa) show distinct development trends with the development of axial strain. Under the confining pressures of 100 and 200 kPa, the hydraulic conductivity gradually decreases with the increase of axial strain and eventually tends to stable. However, under confining pressure of 25 kPa, the hydraulic conductivity first slightly decreased and then significantly increased with the increase of axial strain; under confining pressure of 50 kPa, the hydraulic conductivity first slightly decreased and then slightly increased.

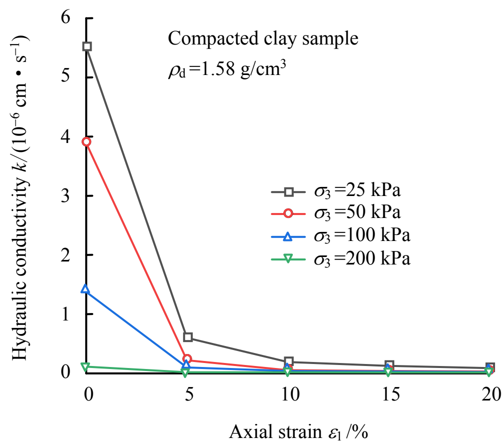


Fig.3 Change of hydraulic conductivity with axial strain(ρ_d =1.58 g/cm^3)

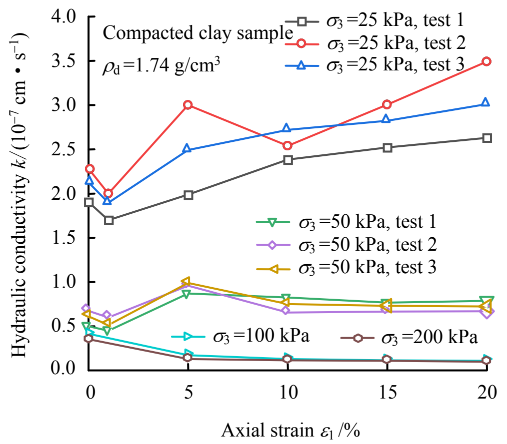


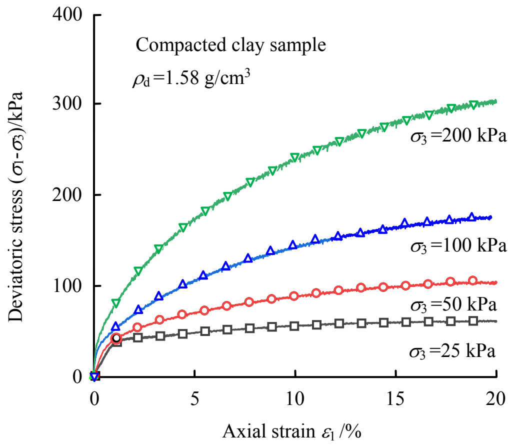
Fig.4 Change of hydraulic conductivity with axial strain(ρ_d =1.74 g/cm^3)

In order to verify that this phenomenon is not accidental, three parallel experiments were repeated for both confining pressures. As can be seen in Fig.4, although the specific values of the hydraulic conductivity changes in the three parallel experiments are slightly different, the trends are consistent, indicating that the increase in hydraulic conductivity measured under low confining pressure is true.

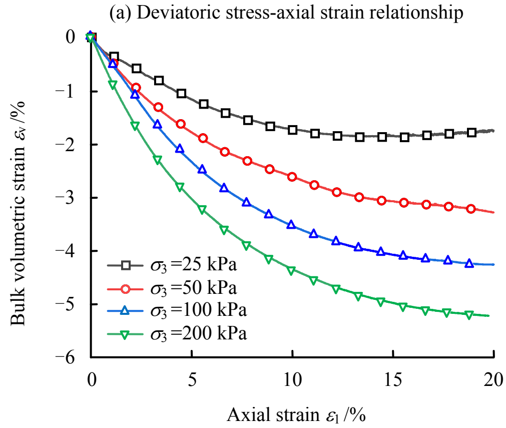
5 Mechanisms associated with permeability change

The variation of hydraulic conductivity in Fig. 3 and Fig.4 can be explained by the deformation behavior and deformation morphology of samples during triaxial compression.

Fig.5 shows the deviatoric stress curve and bulk volumetric strain curve of the sample at $\rho_d = 1.58$ g/cm^3 . The curves of deviator stress under four confining pressures showed strain hardening. The bulk volumetric strain curves with confining pressures of 50, 100 and 200 kPa show continuous volume shrinkage, and the volumetric strain tends to be stable with the increase of shear stress. The bulk volumetric strain curve of $\sigma_3 = 25$ kPa showed slightly dilation after $\varepsilon_1 = 13\%$, but the total bulk volumetric strain is still contracted. The behavior the deviatoric stress curves and bulk volumetric strain curves above indicated that the samples are normally consolidated under confining pressures of 25, 50, 100 and 200 kPa, respectively.



(a) Deviatoric stress-axial strain relationship



(b) Bulk volumetric strain-axial strain relationship

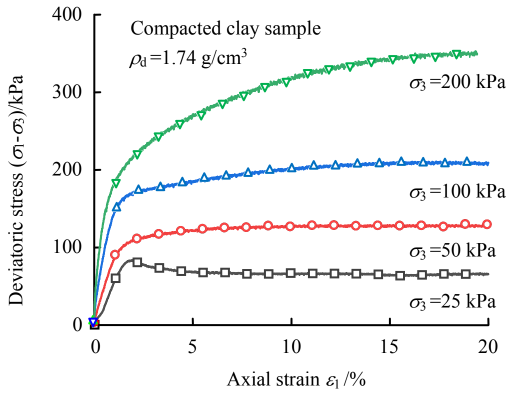
Fig.5 Stress-strain response (ρ_d =1.58 g/cm^3)

The failure shape of samples after triaxial tests under four consolidation pressures are shown in Fig.6. It can be seen that all samples show drum-like (bulging) deformation without obvious strain localization, indicating that the samples have a good ability to adapt to deformation. The bulk volumetric strain curve and the failure shape of the sample explain the reason for the decrease of the hydraulic conductivity. During the shearing process, the sample presents a continuous volume shrinkage, and the pores in the sample are compressed to be smaller and smaller, making it more difficult for permeable water to pass through the pores in the sample.

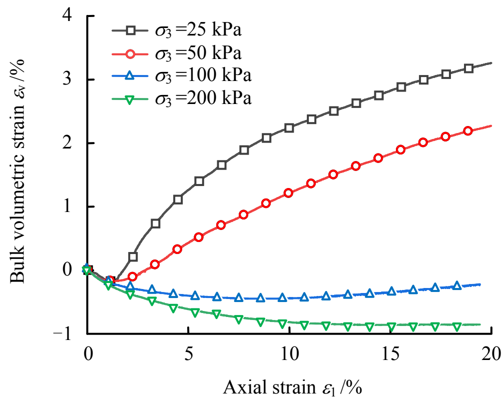
(a) σ_3 =25 kPa (b) σ_3 =50 kPa (c) σ_3 =100 kPa (d) σ_3 =200 kPa

Fig.6 Specimen shapes after triaxial compression (ρ_d =1.58 g/cm^3)

Fig.7 shows the deviatoric stress curve and bulk volumetric strain curve of the sample at $\rho_d = 1.74$ g/cm^3. The deviatoric stress curves of σ_3 = 100, 200 kPa show continuous hardening, but the bulk volumetric strain curves show continuous volume decrease, and tend to be stable with the increase of axial strain, which is in consistent with the deformation characteristics of normally consolidated clay. However, the stress-strain curves of $\sigma_3 = 25$ kPa and $\sigma_3 = 50$ kPa show the typical deformation characteristics of overconsolidated clay: two deviatoric stress curves reach the peak value quickly ($\varepsilon_1 = 2\%$), but the deviatoric stress curve of $\sigma_3 = 25$ kPa shows obvious softening; the two bulk volumetric strain curves first show slightly shear softening ($\varepsilon_1 = 1\% \sim 2\%$), and then obvious and strong shear dilation. The test phenomenon is consistent with the sample of $\rho_d = 1.74$ g/cm^3 with an initial consolidation pressure of about 100 kPa.



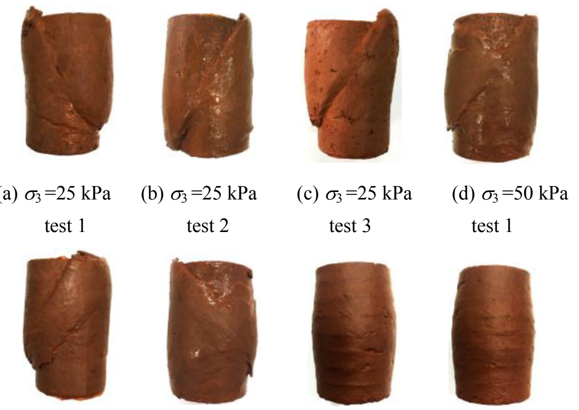
(a) Deviatoric stress-axial strain relationship



(b) Volumetric strain-axial strain relationship

Fig.7 Stress-strain response (ρ_d =1.74 g/cm^3)

Fig.8 shows the specimen's shape after triaxial compression deformation when $\rho_d = 1.74$ g/cm^3. It can be seen that the overconsolidated specimens (σ_3 = 25, 50 kPa) and normal consolidated (σ_3 = 100, 200 kPa) specimens show different failure shapes. The deformation of overconsolidated specimen is no longer uniform after dilation. The strain localization occurs, and the deformation concentrates on the shear plane caused by the maximum shear stress (the angle between the plane and the horizontal direction is about $45^\circ + \varphi / 2$, φ is the internal friction angle). The deformation form of normal consolidated specimen shows drum shape (bulging) without obvious strain localization. By comparing the deformation shape of samples under different confining pressures in figure 8, it can be seen that the ability of compacted clay samples to adapt to deformation decreases with the decrease of confining pressure. When the confining pressure is much less than the preconsolidation pressure, the sample may have strain localization under a small shear strain ($\varepsilon_1 = 1\% - 2\%$).



(a) σ_3 =25 kPa test 1 (b) σ_3 =25 kPa test 2 (c) σ_3 =25 kPa test 3 (d) σ_3 =50 kPa test 1

(e) σ_3 =50 kPa test 2 (f) σ_3 =50 kPa test 3 (g) σ_3 =100 kPa (h) σ_3 =200 kPa

Fig.8 Specimen shapes after triaxial compression (ρ_d =1.74g/cm^3)

Due to two distinct deformation forms, two different trends of the hydraulic conductivity change have appeared. For normally consolidated samples, the variation of hydraulic conductivity is consistent with that of the aforementioned sample of $\rho_d = 1.58$ g/cm^3 . For the overconsolidated specimen, there is obvious strain localization in triaxial compression, and the deformation mainly develops along the concentrated shear band. Because of the dilatation and expansion of the shear band in the shear process, the leakage channel is formed, so the measured axial seepage flow is significantly increased. It should be pointed out that the actual measured axial hydraulic conductivity is only an apparent homogeneous hydraulic conductivity due to strain localization. Because the rate of flow through the specimen and the shear band cannot be distinguished, it is impossible to accurately estimate the hydraulic conductivity of the shear band. Considering that the ratio of the cross-sectional area of the shear band to the cross-sectional area of the specimen is very small, it can be considered that the hydraulic conductivity of the shear band has increased by an order of magnitude. Due to the randomness of the position of the shear band in the sample, and the position of the shear band under the same consolidation pressure is slightly different. Influenced by the end restriction of the sample, the specific value of the apparent hydraulic conductivity measured by the parallel sample shown in Fig.4 is slightly different. It is worth noting that although only a few shear bands run through the upper and lower top surfaces of the specimen in the test, the existence of shear bands still increases the apparent hydraulic conductivity of the specimen, which indicates that the water flow will preferentially pass through the shear band, resulting in the shortening of the permeability path of the specimen and the increase of the seepage gradient, which also confirms that the permeability coefficient of the shear band has changed significantly, and shear band has a great influence on the permeability of the sample.

6 Engineering implications

The above test results reveal an important fact: heavily compacted (relative compaction degree is close to or more than 100%) clay has a high preconsolidation pressure, and its ability to adapt to deformation is reduced under low consolidation pressure, thus strain localization is easy to occur under shear, resulting in concentrated shear zone; more importantly, the soil in the shear bands continues to dilate, which makes its permeability significantly increased.

During the initial impoundment of embankment dams, leakage is often observed at the mid-to-upper elevation where the dam body contacts the slopes on both sides of the bank[13-16], and hydraulic fracturing has been generally considered to be the cause of leakage[17-19]. The criterion of total stress method for hydraulic fracturing is that the water pressure on one side of the soil exceeds the total stress on that side (and the sum of its tensile strength with the total stress); the criterion of effective stress method for hydraulic fracturing is that the minimum effective principal stress in the soil reaches the tensile strength of the soil. The triaxial permeability test in this paper shows that the stress condition of the concentrated shear zone with high permeability is easier to satisfy than that of the traditional hydraulic fracturing criterion, therefore, more attention should be paid to the evaluation and disposal of the concentrated shear zone with high permeability in the actual engineering design.

Engineers suspected hydraulic fracturing happened in some projects in history (such as Teton dam[13], Hyttejuvet dam[14] and Viddalsvatn dam[15]), it is difficult to reasonably reproduce the stress conditions of hydraulic fracturing in the core wall, either in terms of engineering monitoring results or various simulation analysis results[20]; however, it can be determined that the location of their initial leakage is generally located in the middle and upper part of the bank slope dam section, especially near the steep and gentle junction of the bank slope[15]. According to the test observations in this paper, the leakage and hydraulic failure of these typical projects can be reasonably explained. At the middle and upper part of the dam body, because the modulus of the core wall and rockfill at the lower part of the valley is far less than that of the bedrock at the lower part of the dam slope section, and the settlement at the middle part of the valley is far greater than that at the dam slope, it makes the dam section of slope bank move and incline to the middle of river valley, and produces great shear deformation at the contact part of dam body and bank slope. The overburden pressure of the middle and upper parts of the dam slope section is not high, and the lateral movement and deformation will also reduce the lateral restraint stress. In addition, due to adverse slope topography (such as slope change and local channel), it makes the small principal stress of these parts likely to be less than the preconsolidation pressure of the heavily compacted core wall clay. Therefore, it is easy to produce the concentrated shear zone with high permeability. Modern clay-core dams are protected by well-designed upstream and downstream anti-filter materials [1]. In addition, observations during initial water storage and control measurements for water storage speed, the concentrated shear band will heal itself after a certain period of time, which generally will not cause as serious consequences as the Teton dam failure.

Design specification for rolled earth-rock fill dam(DL/T 5395-2007)[1] has the following requirements for the degree of compaction of clay core walls: "The compaction degree of class 1, 2 and high dams shall be 98% - 100%, and that of class 3 medium, low and medium dams below class 3 shall be 96%-98%". In large projects, due to the progress of construction machinery and strict quality control measures, the degree of compaction of core-wall clay often exceeds the design requirements. According to the above analysis, the control index of the degree of compaction should be combined with the specific parts of the core wall. In the area of core wall with small overburden pressure or consolidation pressure (such as the upper bank slope), high degree of compaction will increase the degree of overconsolidation of core wall, thus increasing the risk of high permeability concentrated shear bands. In the upper area of the core wall, especially the upper bank slope area, the degree of compaction should be controlled within the reasonable design range to avoid over compaction.

7 Conclusions

The triaxial compression test and seepage test were conducted on the core-wall clay of a high rock-fill dam. The changes of axial stress, volumetric strain and axial hydraulic conductivity with axial strain were investigated under different confining pressures for samples with different compaction densities. The following conclusions can be drawn:

(1) The change of hydraulic conductivity during triaxial compression on compacted clay is related to confining pressure. Under high confining pressure, the hydraulic conductivity decreases with the increase of axial strain and finally becomes stable. However, under low confining pressure, the measured hydraulic conductivity may increase significantly with the increase of axial strain. The latter is important but has been overlooked in previous studies.

(2) The change of hydraulic conductivity of compacted clay during triaxial compression is closely related to its deformation behavior. Samples with different compacting densities have different preconsolidation pressures. When the confining pressure is greater than the pre-consolidation pressure, the sample normally consolidates under the confining pressure. During the process of triaxial compression, the sample presents a continuous volume contraction, and the deformation of the sample is relatively uniform, the hydraulic conductivity becomes smaller as the pore ratio becomes smaller. When the confining pressure is far less than the pre-consolidation pressure of sample, the clay's ability to adapt to deformation decreases, and the sample quickly undergoes strain localization during triaxial compression, the deformation develops along the concentrated shear band, and the concentrated shear band presents continuous dilatation, the shear band becomes a channel for water flow, so the measured apparent hydraulic conductivity increases significantly.

(3) Triaxial compression seepage test shows that the highly compacted clay will produce concentrated shear band with high permeability during large deformation under low confining pressures. The stress condition of the concentrated shear band that triggering high permeability is easier to meet than that of the traditional hydraulic fracturing criterion, therefore, the evaluation and management of the concentrated shear band should be paid more attention in the engineering design.

References

[1] National Development and Reform Commission of the People's Republic of China. DL/T 5395－2007 *Design specification for rolled earth-rock fill dams*[S]. Beijing: China Electric Power Press, 2008.

[2] Ministry of Water Resources of the People's Republic of China. SL237－1999 *Specification of soil test*[S]. Beijing: China Water & Power Press, 1999.

[3] CHEN Sheng-shui. Key technological issues in construction and safety assurance of super-high earth and rockfill dams[J]. Yangtze River, 2018, 49(5): 74-78.

[4] WANG Xiao-gang. Discussion on some problems observed in high earth-rockfill dam[J]. Chinese Journal of Geotechnical Engineering, 2018, 40(2): 203-222.

[5] ZHU Jian-hua. Triaxial permeability test on undisturbed soils of earth dam core[J]. Chinese Journal of Geotechnical Engineering, 1989, 11(4): 57-63.

[6] LEI Hong-jun, BIAN Feng, YU Yu-zhen, et al. Experimental study of permeability of clayey soil during process of large shear deformation[J]. Rock and Soil Mechanics, 2010, 31(4): 1130-1133.

[7] WANG Gang, WEI Xing. A torsional shear penetration

testing device and corresponding testing method for soils[P]. China: CN104142277A, 2014-11-12.

[8] WANG Gang, WEI Xing, JIANG Li. A ring shear seepage testing device for soils and the soil-structure interfaces[P]. China: CN105606508A, 2015-11-24.

[9] WANG G, WEI X, ZOU T. A hollow cylinder radial-seepage apparatus for evaluating permeability of sheared compacted clay[J]. Geotechnical Testing Journal, 2018, 42(5): 1133-1149.

[10] WANG Gang, YOU Ke-qin, WEI Xing, et al. Experimental study on the permeability of shear bands in compacted clay[J]. Chinese Journal of Geotechnical Engineering, 2019, 41(8): 1530-1537.

[11] WEI Xing, ZOU Ting, WANG Gang. Experimental study on permeability of clay during coupled compression and shear[J]. Chinese Journal of Rock Mechanics and Engineering, 2017, 36(Suppl.1): 3561-3568.

[12] SEED H B, CHAN C K. Structure and strength characteristics of compacted clays[J]. Journal of the Soil Mechanics and Foundations Division, ASCE, 1959, 85(6): 31－47.

[13] U. S. Department of the Interior, Teton Dam Failure Review Group. Failure of Teton dam: a report of findings[R]. Washington: U.S. Department of the Interior, 1977.

[14] KJARNSLI B, TORBLAA I. Leakage through horizontal cracks in the core of Hyttejuvet dam[J]. Norwegian Geotechnical Institute Publication, 1986(80): 39-47.

[15] PENMAN A D M. On the embankment dam[J]. Géotechnique, 1986, 36(3): 303-348.

[16] FOSTER M, FELL R, SPANNAGLE M. The statistics of embankment dam failures and accidents[J]. Canadian Geotechnical Journal, 2000, 37(5): 1000-1024.

[17] HUANG Wen-xi. Several views on research of embankment dams[J]. Water Resources and Hydropower Engineering, 1982(4): 23-27.

[18] SHERARD J L. Hydraulic fracturing in embankment dams[J]. Journal of Geotechnical Engineering, 1986, 112(10): 905-927.

[19] LO K Y, KANIARU K. Hydraulic fracture in earth and rock-fill dams[J]. Canadian Geotechnical Journal, 1990, 27(4): 496-506.

[20] SHEN Zhu-jiang, YI Jin-dong, ZUO Yuan-ming. Centrifuge model test of hydraulic fracture of earth dam and its analysis[J]. Journal of Hydraulic Engineering, 1994(9): 67-78.