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Methods for estimating permanent deformation of unsaturated aging railroad ballast under cyclic loading

繰返し荷重下の不飽和経年劣化鉄道バラストの永久変形量の推定法に関する研究

By

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ABSTRACT

The ballasted track structure is one of the typical track structures for traditional railways in Japan. In general, track deteriorations with a continuous decrease in the load bearing capacity or increase in the permanent deformation are always observed at ballasted tracks mainly due to the degradation of ballast particle size and shape, referred to as “aging” in this study, induced by repeated train loads. In addition, the increase in fine fraction content caused by aging also affects the permeability and water retentivity of the ballast. However, the current Japanese design standard adopts a semi-empirical formula to estimate the permanent deformation of ballast for the construction of new lines, any method for calculating permanent deformation of ballast observed at aged lines has not been proposed by considering the effects of water content and aging, which may reduce the prediction accuracy of the maintenance cycle of ballasted tracks. Therefore, it is meaningful to evaluate the impact of aging on hydro-mechanical behavior of unsaturated ballast, and to further predict the permanent axial deformation by some estimation models, so as to improve the track structure design and reduce maintenance costs.

For this purpose, this study first examines the validity of the Los Angeles Abrasion (LAA) test in the preparation of aged ballast. It indicates that the LAA test can reproduce the grain size distribution and particle shape for in-situ aged ballast obtained from the actual railway track. To evaluate the influence of aging on the hydraulic properties of unsaturated ballast, the water retentivity and coefficient of permeability for fresh and aged ballasts are determined by water retention tests and saturated/unsaturated permeability tests, respectively. The results of water retention tests prove that the aging induces significant increase in water retentivity compared with fresh ballast, due to the presence of fine fraction content. Besides, the results of saturated/unsaturated permeability tests show that the coefficient of permeability of aged ballast is lower than that of fresh ballast, while the relative coefficient of permeability of fresh ballast is more significantly affected by degree of saturation.

In terms of shear strength, the influence of aging on the shear strength behavior of unsaturated ballast is examined through a range of monotonic loading triaxial compression tests. Test results show that the effective shear strength parameters of fresh ballast under fully drained conditions (CD test) are close to those under fully undrained conditions (CU test), regardless of water content. However, for the aged ballast, its effective shear strength parameters are more suitable to be determined through the CU test due to low permeability. Besides, the aging reduces the effective internal friction angle of ballast, while it increases the total cohesion as compared with fresh ballast, especially under the high saturation condition. Regarding the effect of water content, the shear

strength of both samples has a decreasing tendency as water content increases, and the declining trend of shear strength is more noticeable for aged ballast.

On the other hand, it is of great significance to predict the permanent deformation of the ballasted track under repeated train loads. Thus, the influence of aging on the permanent axial deformation of unsaturated ballast is evaluated through a series of cyclic loading triaxial compression tests. It indicates that the permanent axial deformation of ballast under cyclic loading is seriously affected by water content and aging, and the increasing trend of permanent axial deformation becomes more remarkable in such a case where both water content and fine fraction content increase. Besides, the results of the CU test indicate that the aged ballast may have a greater positive excess pore water pressure as compared with those of the CD test, which further boosts the development of permanent axial deformation, especially under the saturated condition. Furthermore, the applicability of two types of estimation models (i.e., UIUC model and SSE model) is verified to the prediction for permanent axial deformation of unsaturated ballast in this study. These results indicate that both the UIUC model and SSE model can well predict the permanent axial deformation of clean and aged ballasts in CD tests. However, the applicability of the UIUC model to the prediction for the permanent axial deformation of aged ballast is limited, since it is difficult to predict the excess pore pressure under cyclic loading. The findings of this study show that aging has adverse effects on the hydro-mechanical behavior as well as permanent axial deformation of unsaturated ballast. Therefore, the effects of aging, drainage condition, and water content on the permanent axial deformation of the ballasted tracks should be considered comprehensively in the actual railway design, which is of great significance to improve the track structure design and reduce maintenance costs.

The thesis includes 8 Chapters. Chapter 1 introduces the background, literature review, objectives, and organization of this study. Chapter 2 introduces the features of test apparatus, like water retention apparatus, saturated/unsaturated permeability apparatus, and medium-size triaxial apparatus used in this study. Chapter 3 describes the test samples and test methods. Chapter 4 evaluates the hydraulic properties of unsaturated ballast based on water retention test and saturated/unsaturated permeability tests. Chapter 5 discusses the influence of aging on shear strength of unsaturated ballast based on monotonic loading triaxial compression test. Chapter 6 discusses the influence of aging on permanent deformation of unsaturated ballast based on cyclic loading triaxial compression test. Two types of methods for estimating the permanent deformation of unsaturated ballast are presented in Chapter 7. Lastly, Chapter 8 summarizes the conclusions of this study and gives the recommendations for future studies.

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2. Ishikawa, T., Lin, T.S., Yang, J.Q., Tokoro, T., and Tutumluer, E. (2019). Application of the UIUC model for predicting ballast settlement to unsaturated ballasts under moving wheel loads. *Transportation Geotechnics*, 18: 149-162. (doi: <https://doi.org/10.1016/j.trgeo.2018.12.003>)
3. Yang, J.Q., Ishikawa, T., Matsutani, S., Tokoro, T., Nakamura, T., and Momoya, Y. Influences of water content and fine fraction content on permanent deformation of ballasts. *16th Asian Regional Conference on Soil Mechanicals and Geotechnical Engineering*, 2019 Oct 14-18, Taipei, Taiwan.
4. Yang, J.Q., Ishikawa, T., Okayasu, T., Nakamura, T., and Kijiya, I. Applicability of SSE model for railroad ballasts with various water contents and fine fraction contents. *IS-Atlanta 2018*, 2018 Sept 10-13, Atlanta, USA.
5. Yang, J.Q., Ishikawa, T., Tokoro, T., Nakamura, T., and Momoya, Y. An estimation shear strength equation for unsaturated ballast with different fine fraction contents. 251-258. *59th Annual Meeting of Hokkaido Branch Japanese Geotechnical Society*, 2019 Jan 25-26, Tomakomai, Japan.

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1 INTRODUCTION

1.1 Background

The ballasted track structure (Figure 1-1) is one of the typical track structures for traditional railways in Japan. In general, it is known that the railroad ballast is gradually contaminated with fouling materials during the train operation period. These fouling materials, which are finer than fresh ballast aggregates, are commonly produced from the ballast abrasion, breakdown, and external contamination (Indraratna et al., 2011; Selig and Waters, 1994). For example, Selig and Waters (1994) illustrated that the primary source of fouling materials is the breakdown and abrasion of ballast particles induced by repeated train loads.

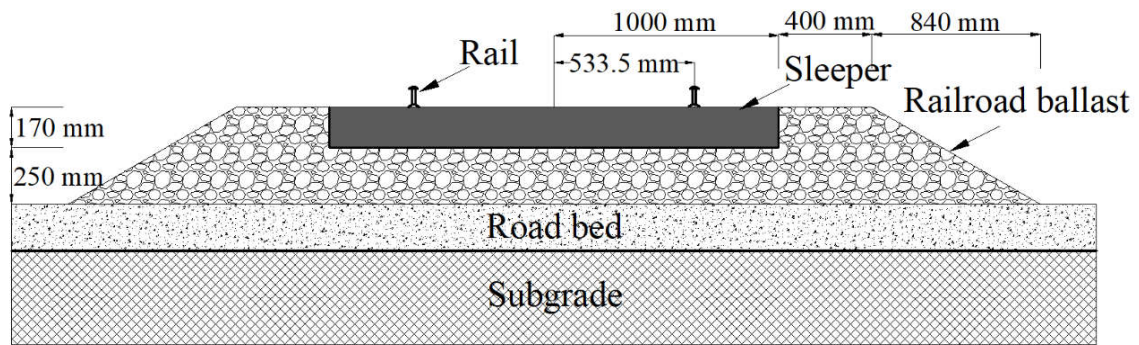


Figure 1-1 Schematic diagram of a typical ballasted track structure in Japan.

In general, track deteriorations with a continuous decrease in the load bearing capacity or increase in the permanent deformation are always observed at ballasted tracks mainly due to the degradation of ballast particle size and shape, referred to as “aging” in this study, induced by repeated train loads. In addition, the increase in fine fraction content caused by aging also affects the permeability and water retentivity of the ballast, which may cause the generation of mud from under the sleepers and further reduce the load bearing capacity or increase the development of permanent deformation of ballasted track, as shown in Figure 1-2. However, the current Japanese design standard (RTRI, 2012) adopts a semi-empirical formula (Eq. (1-1)) to estimate the permanent deformation of ballast for the construction of new lines, any method for calculating permanent deformation of ballast

observed at aged conventional lines has not been proposed by considering the effects of water content and aging, which may reduce the prediction accuracy of the maintenance cycle of ballasted tracks.



Figure 1-2 Fouled ballast in the aged existing line.

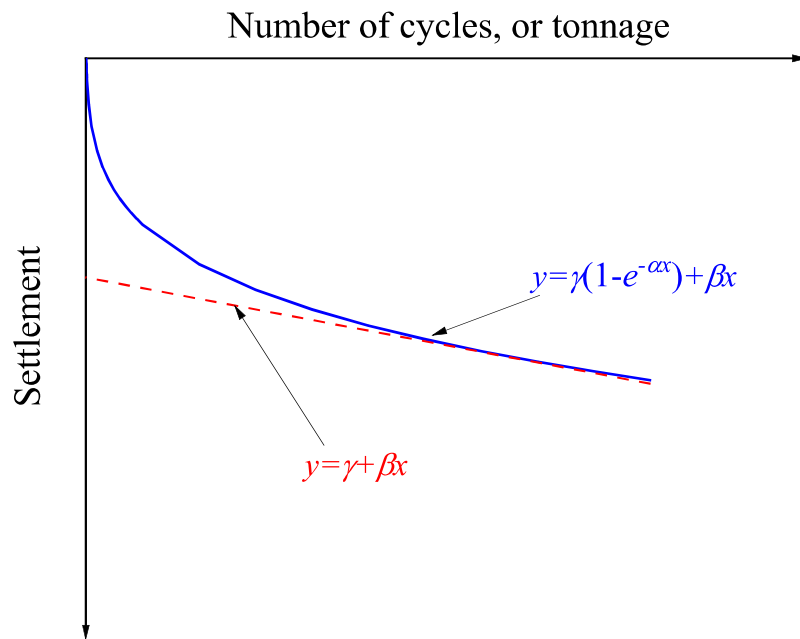


Figure 1-3 Estimation method of ballasted track settlement in the current standard.

$$y = \gamma(1 - e^{-\alpha x}) + \beta x \quad (1-1)$$

where, x represents the number of loading cycles or tonnage; γ gives the severity of the settlement; and α indicates how quickly the initial part of the settlement attenuates; the severity of the long-term settlement is given by the parameter β .

Figure 1-3 shows estimation method of ballasted track settlement in the current standard. It indicates that the settlement process of the ballast track includes two parts: initial settlement process and progressive settlement process. The settlement of the ballasted track has a great rapid growth trend during the initial loading process, and then it increases linearly as the number of loading cycles increases. Although Eq. (1-1) can simply predict the settlement of the ballasted track by using field measurement data to calibrate its input parameters, the settlement is sometimes difficult to interpret because the analysis based on field measurement involves many factors, such as load amplitude, load frequency, moisture content, ballast aging (particle size and shape), and confining pressure, etc. To separate the influence of different factors and find out the controlling parameters has been the major objective of many research studies. In this case, previous studies have indicated that ballast fouled and water content are the main factors affecting the settlement of ballasted tracks. However, the Eq. (1-1) used in the current design standard cannot clearly reflect the impact caused by these factors. Therefore, it is meaningful to evaluate influence of aging on the hydro-mechanical behavior and permanent deformation of unsaturated ballast, and to further predict the permanent deformation using some estimation models, so as to improve the track structure design and reduce maintenance costs.

1.2 Literature Review

1.2.1 Monotonic triaxial compression tests for coarse granular materials

Many past studies (e.g., Cabalar, 2011; Ebrahimi et al., 2015; Giannakos, 2010; Indraratna and Salim, 2002; Kim et al., 2005; Tutumluer et al., 2008) indicated that the increase in fine fraction content affects shear strength properties for coarse granular materials. However, the favorable or adverse impact of fine fraction content on the shear strength behavior depends on soil properties of the fouling content (Indraratna et al., 2013a). For example, the sand-size and fine grain-size fouling materials are beneficial to enhance the shear strength of ballast, owing to a wider range of gradation and better compaction performance (Selig and Waters, 1994). On the contrary, the fine particles such as clay and

coal can decrease the shear strength by reducing the interlock of coarse aggregates. For example, [Huang et al. \(2009\)](#) revealed that the shear strength of the coal-fouled ballast steadily decreases with increase in fouling content. Similar findings are also confirmed by [Indraratna et al. \(2013b\)](#). In addition, ballasted tracks are subject to the rainfall infiltration and seasonal changes in groundwater levels during the melting season, which causes a fluctuation in water content of the ballast layer. The key role of water content in the shear strength of coarse granular materials was also confirmed in previous research ([Duong et al., 2013](#); [Ekalad, 2006](#)). More importantly, in cases where the fouling content of the ballast is noticeable, it would reduce the drainage capacity of the ballasted track and thus decrease the shear strength of the ballasted track ([Trinh et al., 2012](#)).

Although severe issues related to ballast fouling and water content have been studied, previous findings ([Ishikawa et al., 2019](#); [Kashani et al., 2017](#); [Thu et al., 2006](#)) were mainly based on the triaxial compression tests under fully drained conditions for pore air and water pressures, or the undrained condition for pore water pressure and the drained condition for pore air pressure. However, many engineering practices cannot achieve fully drained conditions for pore air and water pressures. For example, the air can be trapped in the ballast layer during heavy rains. In addition, for low-permeability soils, the air and water cannot be quickly removed from the soil during the earthquake or fast-moving train loads. Considering that the previous studies on undrained triaxial test (CU test) for unsaturated soils were mainly carried out under the constant water content condition (CW test), where pore water pressure was drained and pore air pressure was undrained. Consequently, it is meaningful to study the air-water-soil interaction in unsaturated soils under fully undrained conditions for pore air and water pressures.

1.2.2 Cyclic triaxial compression tests for coarse granular materials

Track deteriorations with a remarkable increase in permanent deformation are always observed at actual ballasted tracks mainly due to ballast aging. The permanent deformation of the aggregate materials was affected by stress level, moisture content, fine fraction content, grading and so on. ([Lekarp et al., 2000](#); [Le Pen et al., 2013](#)). For example, the increase in fine fraction content seriously alters the deformation-strength characteristics of the ballast, depending on the amount of fouling materials mixed with clean ballast ([Dareeju et al., 2015](#); [Lackenby et al., 2007](#)). Besides, [Indraratna et al. \(2013a\)](#) point out that with

the increase in the fine fraction content, the sleeper settlement of fouled ballasted track increases. [Ebrahimi et al. \(2010\)](#) also indicated that the water content of fouling materials (increasing from 5 to 25%) had a significant effect on accelerating the plastic deformation and changing the shape of deformational behavior of ballast from a small rate of plastic deformation (i.e., plastic creep) to a high rate of plastic deformation. [Ishikawa et al. \(2019\)](#) revealed that both water content and fine fraction content may have serious influences on permanent deformation of the ballast. However, the effect of particle shape changes and drainage conditions on permanent deformation was not evaluated in previous studies on fouled ballast.

1.2.3 Methods for estimating permanent deformation of soils

For semi-empirical model, many advanced phenomenological models for estimating the accumulative strain of geomaterials under cyclic loading have been proposed by past researchers. [Abadi et al. \(2016\)](#) reviewed some ballast settlement models by comparing the estimation results with the measured results obtained from full-scale model tests, and reported the weaknesses of the existing models that none of the models allow for sleeper properties, sleeper spacing, ballast type, ballast depth or shoulder dimensions. This suggests that there are some variables to incorporate into these models in order to improve the precision and the applicability. Besides, several rutting models have been proposed to estimate the permanent deformation of pavement materials under repeated traffic loads so far (e.g. [Lekarp et al., 2000](#); [Monismith et al., 1975](#)). To adequately characterize and predict the performance of pavements under repeated traffic loads, the UIUC model was established to estimate permanent deformation of pavement materials under repeated traffic loads considering the effects of applied stress levels and shear strength properties ([Qamhia et al., 2016](#)). Previous studies ([Chow et al., 2014a; 2014b](#)) have shown that the UIUC rutting model can well estimate permanent deformation of pavement materials. However, the applicability of the UIUC model on the estimation of permanent deformation for ballast materials under repeated train loads was not verified will be verified by considering some uncertain effects by considering some uncertain influencing factors, like the effects of ballast fouling and fluctuation in water content.

On the other hand, for the numerical simulations, various constitutive models for granular materials have been developed so far ([Hashiguchi and Ueno, 1977](#); [Habiballah](#)

and Chazallon, 2005; Iwan, 1967; Mroz et al., 1981; Niemunis et al., 2005). In these models, the subloading surface model showed a good ability to predict the cyclic plastic deformation behavior for metals (Hashiguchi and Ueno, 1977). For this model, it was assumed that a subloading surface with a similar geometry in the normal yield surface, and the applied stress is located on the subloading surface. Therefore, the loading criterion does not need to determine whether the stress is located on the yield surface, which can be used to describe the mechanical behavior as well as cyclic plastic deformation of materials such as not only smooth elastoplastic transition but also cyclic loading behavior. Compared with the conventional constitutive models which only have a normal yield surface, the plastic modulus of subloading surface model is expressed by the ratio of the subloading surface to the normal yield surface. After that, the subloading surface model was modified by assuming an existence of the elastic domain surface inside the subloading surface to realistically describe the inelastic deformation behavior of coarse granular material under cyclic loading conditions. The validity of subloading surface extension (SSE) model in predicting the permanent deformation of air-dried clean ballast under cyclic loading was verified by comparing the test results with the numerical results (Okayasu et al., 2014). However, the applicability of this model to predict the cyclic plastic deformation of the ballast with various water contents or fine fraction contents has not been discussed so far.

1.3 Thesis Objectives

To improve the track structure design and reduce maintenance costs, the objectives of this study are to comprehensively investigate the effects produced by aging on the hydro-mechanical behavior and permanent deformation of unsaturated ballast, and to further predict the permanent deformation of fresh and aged ballasts with different water contents by some estimation models.

For this purpose, this study first examines the validity of the Los Angeles Abrasion (LAA) test in the preparation of aged ballast in terms of grain size distribution and particle shape. Then, the water retention tests and saturated/unsaturated permeability tests are conducted for fresh and aged ballasts to evaluate the influence of aging on hydraulic properties of ballast (i.e., water retentivity, permeability). Moreover, the effects of water content, drainage condition, and confining pressure on the shear strength behavior for fresh

and aged ballasts are examined through a range of monotonic triaxial compression tests. In this case, the variations of shear strength parameters for fresh and aged ballasts are discussed according to unsaturated soil mechanics.

On the other hand, it is of great significance to evaluate and further predict the permanent deformation of the ballasted track under the repeated train loads. Therefore, the influence of aging on permanent axial deformation of unsaturated ballast is also evaluated through a series of cyclic loading triaxial compression tests for fresh and aged ballast. Moreover, the applicability of two types of estimation models (i.e., UIUC model and SSE model) is verified to the prediction for the permanent deformation of fresh and aged ballasts with different water contents in this study.

1.4 Thesis Organization

This dissertation is divided into eight chapters as follows:

Chapter 1 introduces the background, literature review, objectives, and organization of this study.

Chapter 2 introduces the features of test apparatus, like water retention apparatus, saturated/unsaturated permeability apparatus, and medium-size triaxial apparatus used in this study.

Chapter 3 describes the test samples and test methods.

Chapter 4 evaluates the hydraulic properties of unsaturated ballast based on water retention test and saturated/unsaturated permeability tests.

Chapter 5 discusses the influence of aging on shear strength of unsaturated ballast based on monotonic loading triaxial compression test.

Chapter 6 discusses the influence of aging on permanent deformation of unsaturated ballast based on cyclic loading triaxial compression test.

Chapter 7 presents two methods for estimating the permanent deformation of unsaturated ballas.

Lastly, Chapter 8 summarizes the conclusions of this study and gives the recommendations for future studies. Figure 1-4 shows the flowchart of this dissertation.

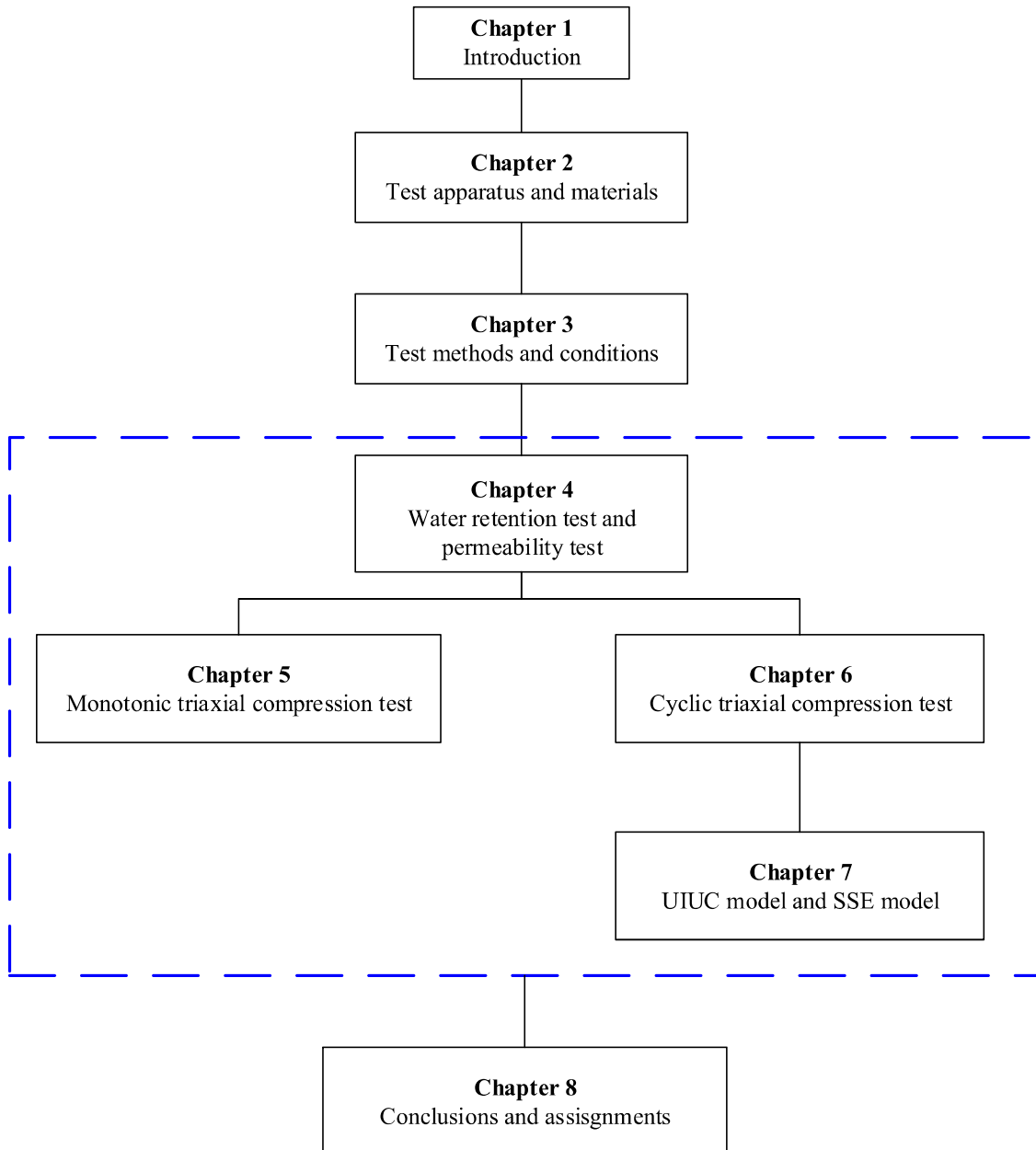


Figure 1-4 Flowchart of this study.

2 TEST APPARATUS

2.1 Water Retention Apparatus

In this study, the water retention test was carried out on a newly developed water retention test apparatus for coarse granular materials. [Figure 2-1](#) shows the schematic diagram of this test apparatus. The apparatus has the following features: (1) It can be used for the water retention test based on the standard of the Japanese Geotechnical Society ([JGS 0151-2009](#)). (2) Test time can be shortened by reducing the specimen size and setting two drainage paths at the pedestal. (3) Water retention tests can be carried out by the suction method and the pressure method.

For this apparatus, since the cap is a perforated loading plate, both the confining pressure and the pore air pressure (u_a) of the specimen are equal. Besides, since the lateral direction of the specimen was surrounded by a rigid collar, the sample was one-dimensionally consolidated under a specified overburden pressure (σ_v), which can be controlled using a BF cylinder. As shown in [Figure 2-1\(b\)](#), two drainage paths in the pedestal are designed to be capable of handling a wide range of suction by using a versapor membrane on one side and a ceramic disk on the other side. During the water retention test, the drainage volume with the applied matric suction (s) can be measured by using an electronic balance in the suction method, while it can be measured by a double tube burette and a differential pressure gauge in the pressure method. Here, matric suction (s) is defined as $s = u_a - u_w$, u_a is pore air pressure, and u_w is pore water pressure.

The size of a cylindrical specimen in this apparatus was 100 mm in height (H) and 150 mm in diameter (D). It is noted that the specimen height (H) in this apparatus was lower than that in the medium-size triaxial compression apparatus ([Figure 2-5](#)). In this case, the validity of the water retention test results obtained by this apparatus is proved by comparing test results obtained by the medium-size triaxial test apparatus in previous research ([Aoyagi et al., 2019](#)). For more information about this test apparatus, [Aoyagi et al. \(2019\)](#) is to be referred.

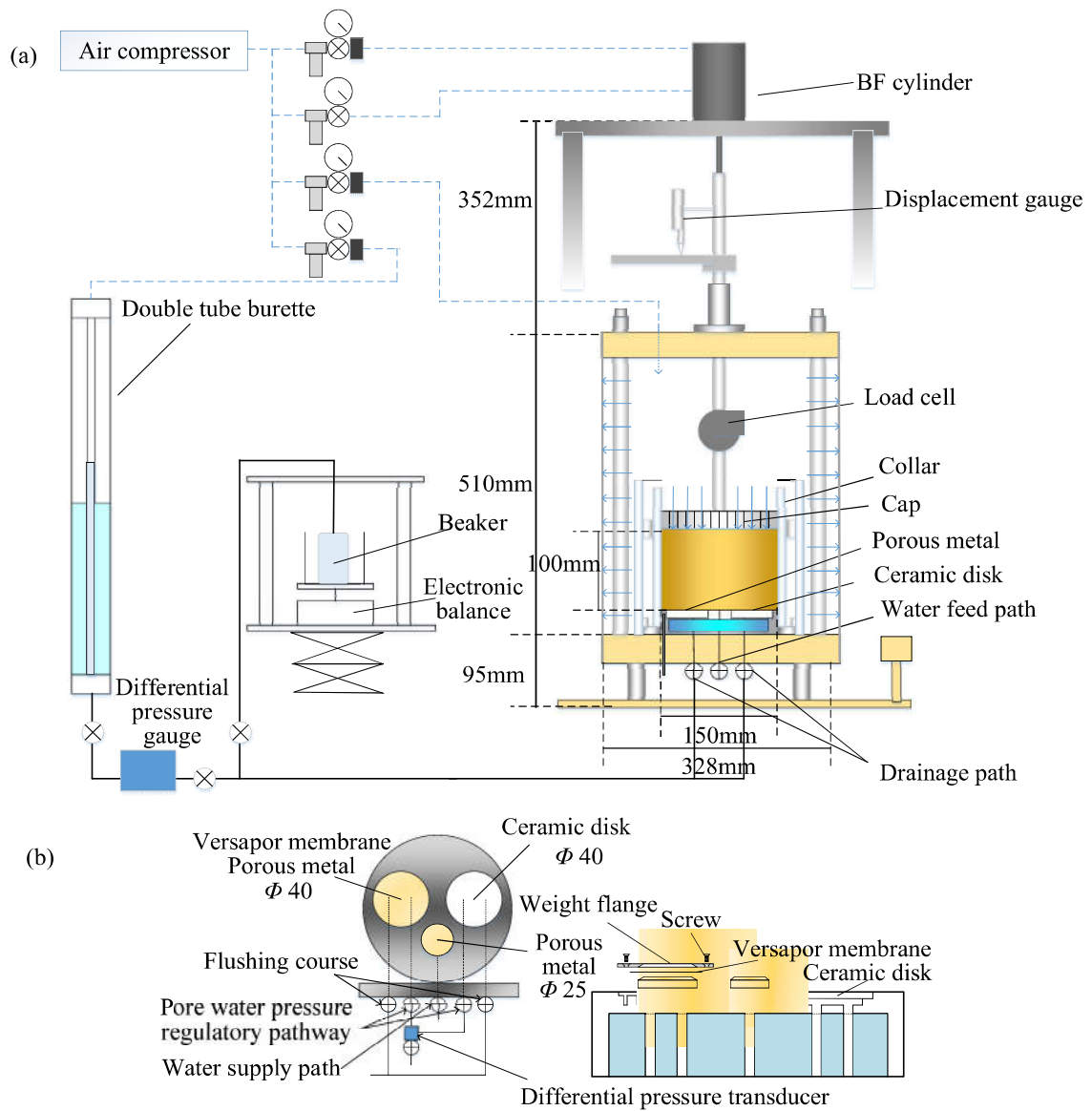


Figure 2-1 Water retention test apparatus: (a) schematic diagram; (b) structure of pedestal (after Aoyagi et al., 2019).

2.2.2 Unsaturated permeability apparatus

Similarly, Figure 2-3 shows the schematic diagram of unsaturated permeability test apparatus for granular coarse materials. The size of a cylindrical specimen in the apparatus was 125 mm in the height (H) and was 150 mm in the diameter (D). For the unsaturated permeability test, the instantaneous profile method was employed to calculate the unsaturated coefficient of permeability (k_{us}) of soil. It should be noted that in this study, the structures of the unsaturated permeability apparatus have some difference in order to consider the different rainfall intensity (RI). As shown in Figure 2-3(a), it can perform the unsaturated permeability test for soils with the rainfall intensity (RI) ranged from 5 to 200 mm/h. Besides, a spray head was installed on the right apparatus (Figure 2-4), which can apply a higher water head value to consider the greater rainfall intensity (RI) ranged from 300 to 2500 mm/h.

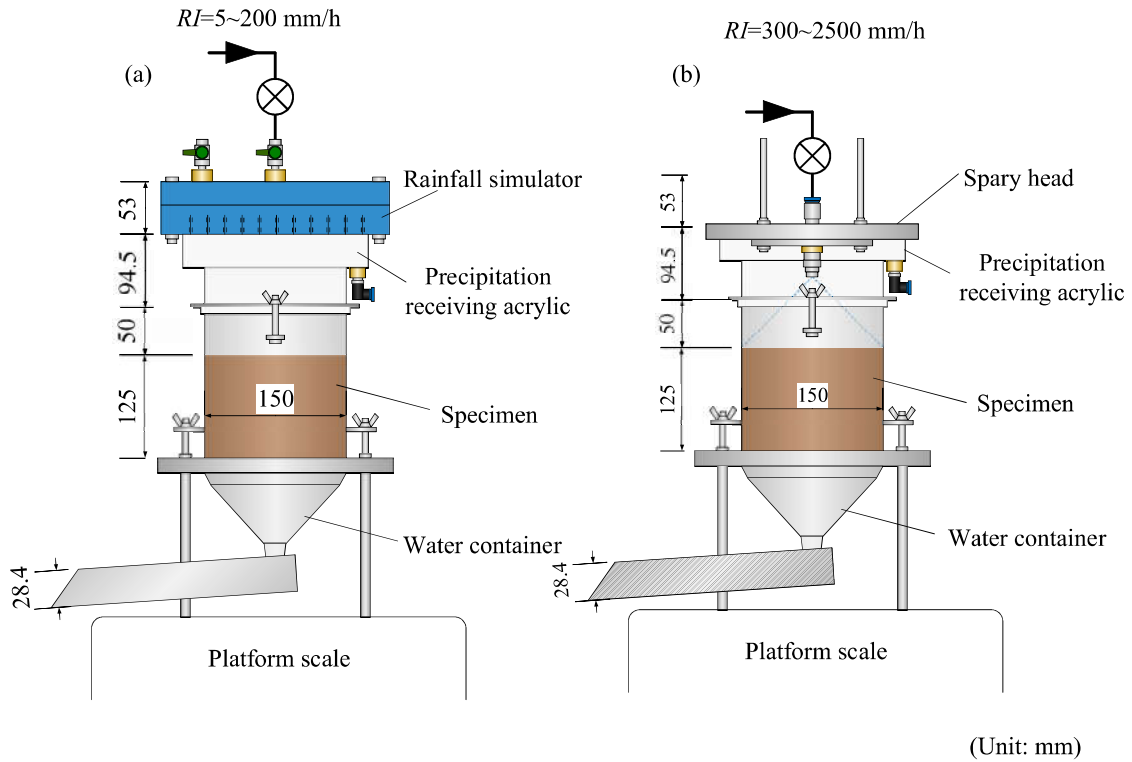


Figure 2-3 Unsaturated permeability apparatus to simulate different rainfall intensity (RI): (a) $RI=5\sim200$ mm/h; (b) $RI=300\sim2500$ mm/h.

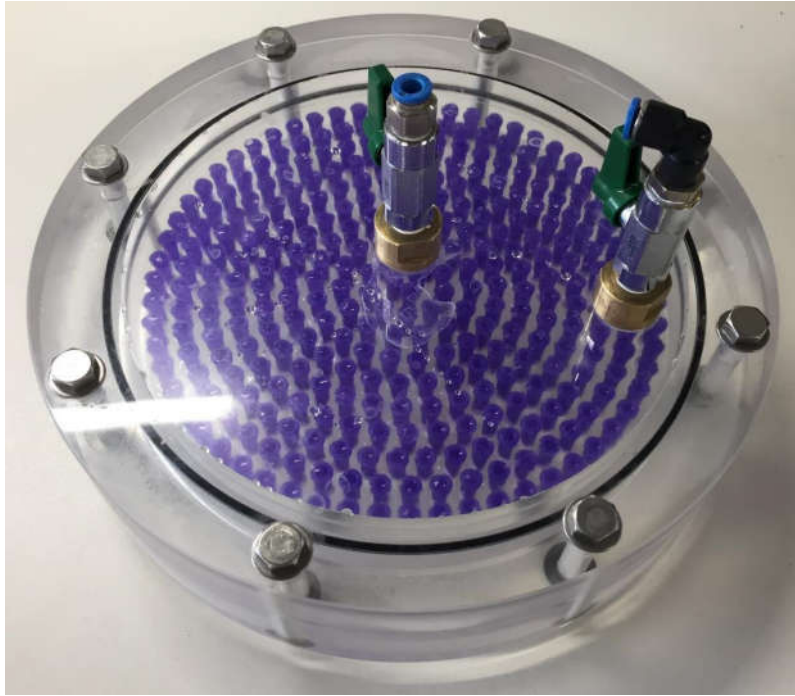


Figure 2-4 Photo of a rainfall simulator.

2.3 Medium-Size Triaxial Apparatus

A schematic diagram of the medium-size triaxial apparatus for the unsaturated granular coarse materials was shown in [Figure 2-5](#). The pressure membrane method was adopted for the unsaturated soil to apply the matric suction. One key feature of the apparatus is the structural design on the cap and the pedestal as shown on [Figure 2-5\(b\)](#) and [Figure 2-6](#). The designs for the cap and the pedestal are the same, which can control the pore water pressure and the pore air pressure independently. Therefore, the apparatus can apply the matric suction from both top and bottom ends of the specimen. For the cap and the pedestal, the pore water pressure is applied to the specimen through a versapor membrane filter attached to the water plumbing path, while the pore air pressure is applied through a hydrophobic polyflon filter attached to the air supply path as shown in [Figure 2-5\(b\)](#). The photos of the versapor membrane filter and the hydrophobic polyflon filter are presented in [Figure 2-7](#), respectively. Here, the versapor membrane filter is a kind of microporous membrane filters made from a hydrophilic acrylic copolymer. Physical properties of the filters are shown in [Table 2-1](#).

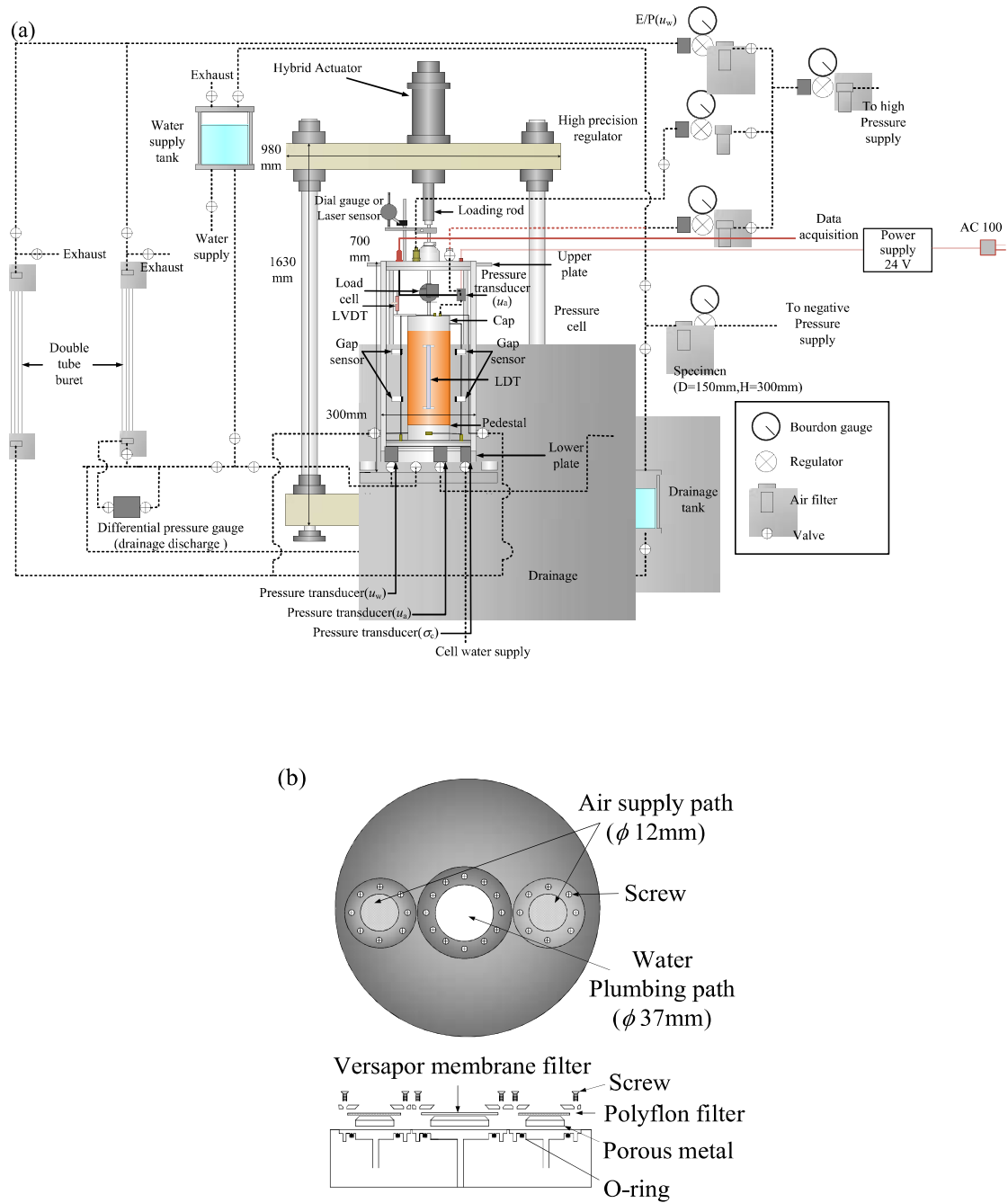


Figure 2-5 Medium-size triaxial test apparatus for unsaturated soils: (a) schematic diagram; (b) structures of cap and pedestal (revised after [Ishikawa et al., 2014](#)).

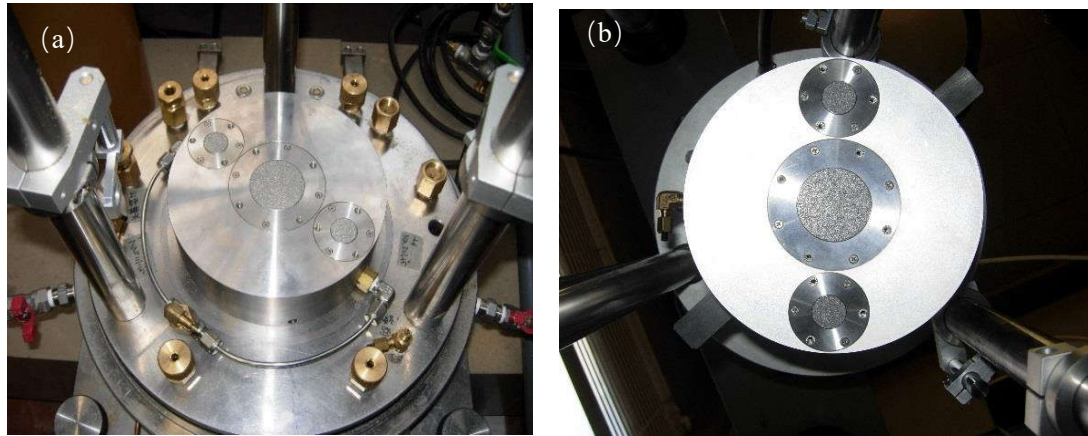


Figure 2-6 Photos of cap and pedestal of medium-size triaxial compression apparatus: (a) the cap; (b) the pedestal.

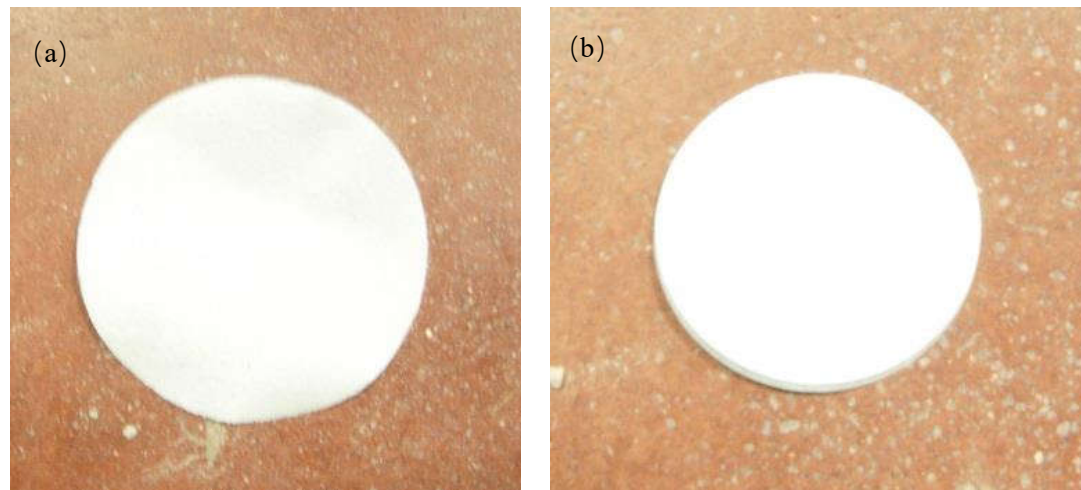


Figure 2-7 The photos of (a) versapor membrane filter and (b) hydrophobic polyflon filter.

Table 2-1 Physical properties of filters.

Name	Thickness (μm)	Pore size (μm)	AEV/WEV (kPa)	Water flow (ml/min/cm ²)
Versapor membrane filter	94.0	0.8	60.0	142
Hydrophobic polyflon filter	540.0	-	17.5	-

AEV is the air entry value of the versapor membrane filter, while WEV is the water entry value of hydrophobic polyflon filter; Water flow is the maximum flow of versapor membrane filter under the differential pressure of 70 kPa.

In addition, the apparatus can perform both monotonic and cyclic loading triaxial compression tests with a medium-size cylindrical specimen with initially 300 mm in height (H) and 150 mm in diameter (D) in accordance with the standards of the Japanese Geotechnical Society ([JGS 0523-2009](#); [JGS 0524-2009](#); [JGS 0527-2009](#)). In this case, the apparatus can apply axial load to a specimen with high accuracy by both the strain control method and the stress control method with only one hybrid actuator. Moreover, the apparatus can perform both monotonic loading tests with very slow strain rate and cyclic loading tests in which the maximum frequency of cyclic loading is up to about 10 Hz.

The measurements of stress and strain in a specimen for monotonic and cyclic triaxial compression tests are performed as follows. Axial stress (σ_a) of both tests was measured by a load cell installed inside the triaxial cell. Volumetric strain (ε_v) was mainly calculated by the lateral displacements of the specimen, namely the change in the specimen diameter, measured by two sets of two proximity transducers (gap sensors) attached at the points of 1/4 and 3/4 of the specimen height diagonally opposite to each other around the specimen diameter, respectively. For the saturated specimen, the volume of water drainage during test was also measured with a double tube burette. It is noted that for monotonic loading triaxial compression tests, axial strain (ε_a) was mainly obtained by measuring the displacement of the loading position with an external displacement transducer and a linear variable differential transducer (LVDT), and for cyclic loading triaxial compression test, the axial strain (ε_a) was obtained by measuring the displacement of the loading position with a laser sensor. For more information about this triaxial apparatus, [Ishikawa et al. \(2014\)](#) can be referred.

On the other hand, it is worth mentioning that in this study, this apparatus was improved by installing a new electrical pressure transducer ([Figure 2-5\(a\)](#)) on the cap to measure the pore air pressure (u_a) in the CU test. To be specific, the pore water pressure (u_w) in the CU test was measured by a pressure transducer installed on the pedestal, and the pore air pressure (u_a) in the CU test was measured by this new electrical pressure transducer mounted on the cap to minimize the effect of air diffusion, as mentioned in [Fredlund et al. \(2012\)](#). The reliability of the measurement system for the pore air pressure (u_a) will be examined in the Chapter 5.

2.4 Calibration of Measurement Transducers

2.4.1 System compliance (SC) and bedding errors (BE)

The system compliance (SC) and bedding errors (BE) have been addressed as problems encountered in accurately measuring the axial displacement with an external displacement transducer, especially in triaxial compression tests for hard geomaterials such as gravel (Tatsuoka and Shibuta, 1992). In order to check the effects of system compliance (SC) and bedding errors (BE), one air-dried specimen was sheared at small axial strain ranging from 0 to 2 %, meanwhile, the axial displacement was measured using the external displacement transducer (EXT), the two linear variable differential transformers (LVDTs) installed on the top of the cap inside the triaxial cell and two LDTs attached at the center of the specimen, in parallel with two side lines located at the diagonal position.

Figure 2-8 to Figure 2-11 present the axial strain measured from different transducers obtained from the ML tests results of the air-dried specimen of 1/2CB and 1/2AB at axial strain ranging from 0 to 2 %. In this case, the difference between ε_a (LVDT) and ε_a (EXT) mainly indicates the effect of SC, while the difference between ε_a (LDT) and ε_a (LVDT) mainly explains the effect of BE. the relationship between ε_a (LVDT) and ε_a (EXT) at the axial strain ranges from 0 to 2 %. The value of ε_a (LVDT) is almost 80 % and 90% of ε_a (EXT) for 1/2CB and 1/2AB, respectively. Therefore, it can be considered that the measuring error caused by system compliance (SC) is approximately 10~20 % of ε_a (EXT) in this study. Besides, the result shows that BE appears strongly during the initial loading stage and then the increasing rate of ε_a (LVDT) is almost equal to that of ε_a (LDT) at the axial strain larger than axial strain of 2 %. On the other hand, for the CL tests, Figure 2-12 and Figure 2-13 shows the relationship between ε_a (LDT) and ε_a (Laser sensor) at the axial strain ranges from 0 to 2 %. The value of ε_a (LDT) is almost 70 % of ε_a (Laser sensor) and the effect is nearly stable regardless of the strain level. Therefore, it can be considered that the measuring error caused by system compliance and bedding errors can be considered in the CL tests in this study.

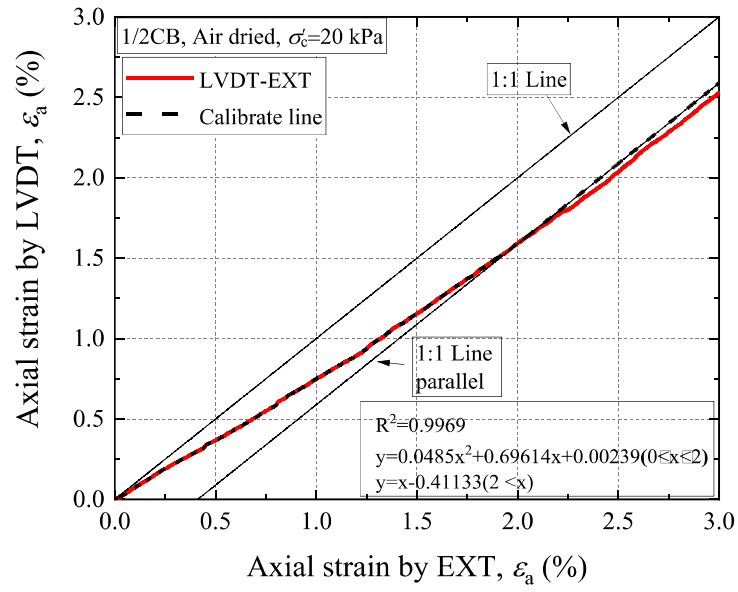


Figure 2-8 Relationship between LVDT and EXT of 1/2CB.

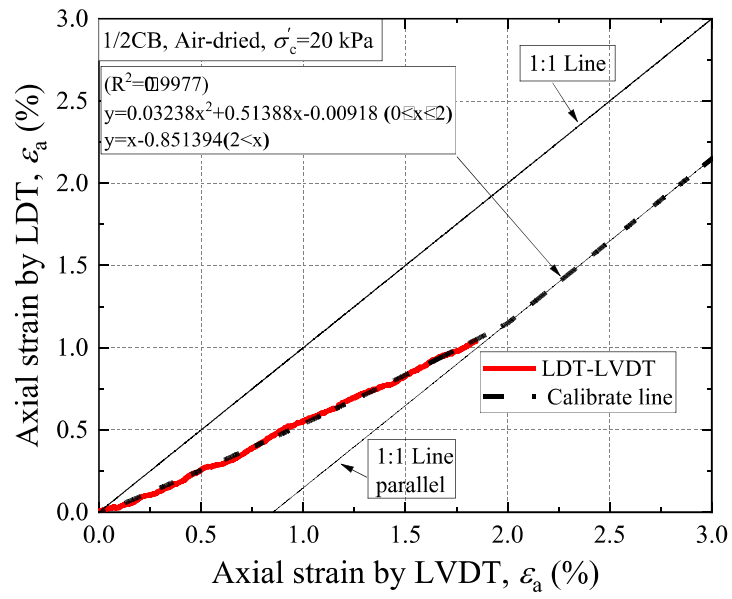


Figure 2-9 Relationship between LDT and LVDT of 1/2CB.

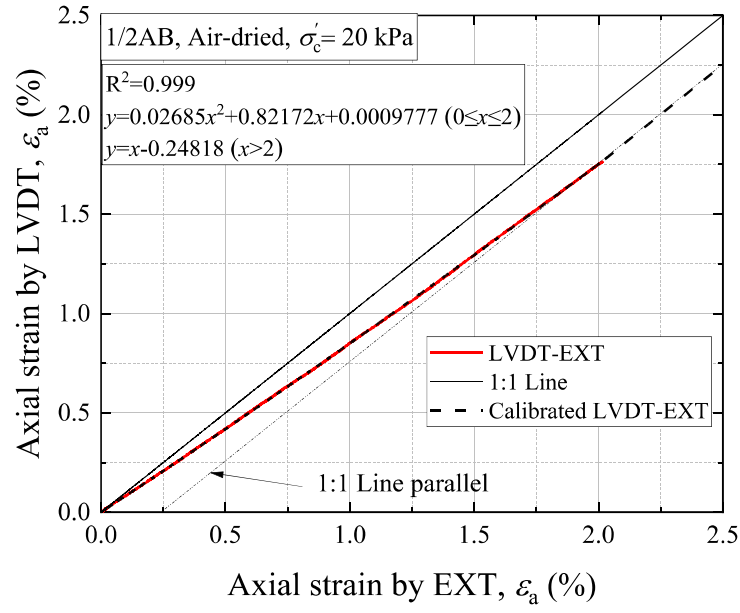


Figure 2-10 Relationship between LVDT and EXT of 1/2AB.

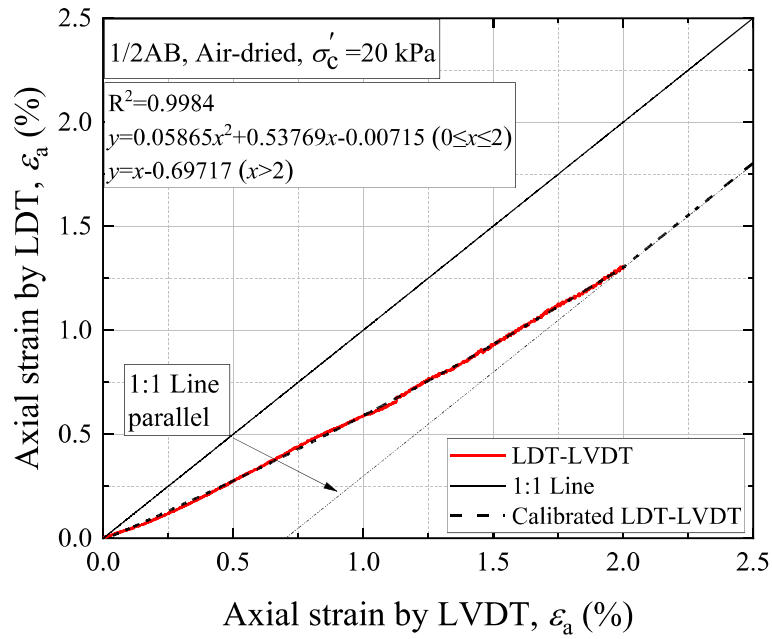


Figure 2-11 Relationship between LDT and LVDT of 1/2AB.

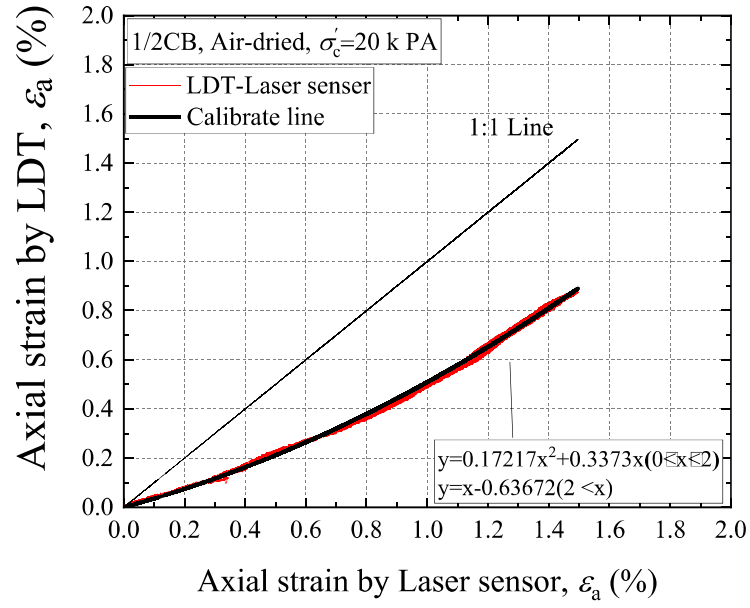


Figure 2-12 Relationship between LDT and Laser sensor of 1/2CB.

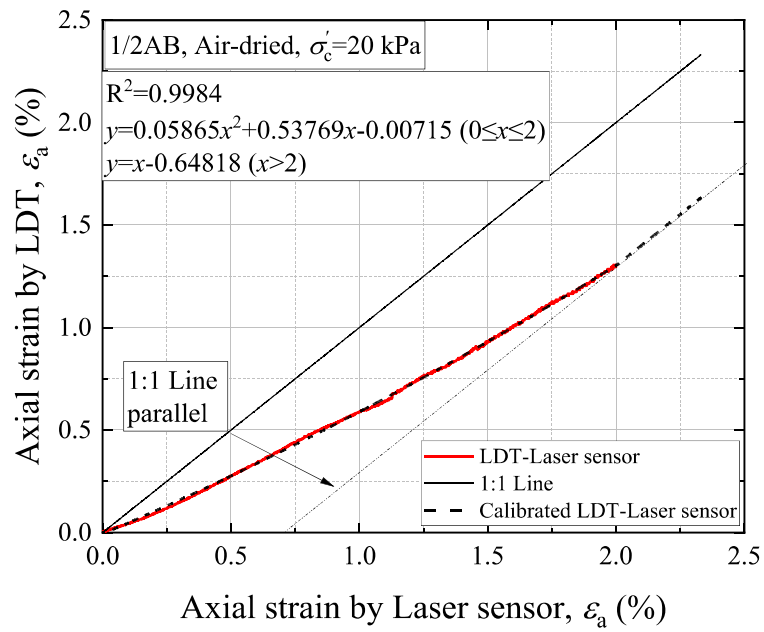


Figure 2-13 Relationship between LDT and Laser sensor of 1/2AB.

2.4.2 Measuring accuracy of volumetric strain in CU tests

The gap sensors were used to measure the volumetric strain for specimens on monotonic triaxial compression tests. Note that a double tube burette was also used to measure the volumetric strain for saturated specimens. The volumetric strain (ε_v) measured by gap sensors is the average value of volumetric strains (ε_{v1}) and (ε_{v2}) derived from two different calculation methods shown in Figure 2-14. The ε_{v1} was calculated supposing that the specimen after shear has a vertical cross section, like the shape of a beer barrel, whose curved boundary is approximated by a parabola as shown Eq. 2-1 (Kato and Kawai, 2000). While the ε_{v2} was calculated supposing the specimen diameter uniformly spreads like a cylinder regardless of the height during compression. In ε_{v1} calculation method, the specimen was separated into two parts, i.e., the upper and lower parts. The volumetric strain of upper part and lower part specimen were determined by two parabola functions from the lateral displacement by gap and height at the position of the gap sensor (e.g., point B in Figure 2-14), and the end of the specimen (e.g., point A in Figure 2-14).

$$y = ax^2 + c \quad (2-1)$$

Here, a and c are variables.

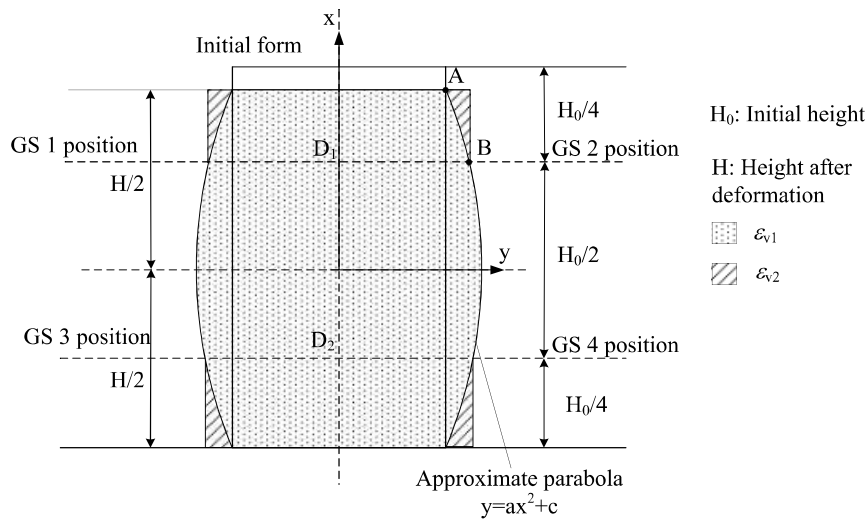


Figure 2-14 Calculation methods for volumetric strain.

In order to evaluate the applicability and the usefulness of gap sensors, Figure 2-15 and Figure 2-16 show the volumetric strain (ε_v) obtained by different methods for the air-

dried specimen under effective confining pressure of 20 kPa, respectively. Noted that here the ε_v (GS) was volumetric strain measured by gap sensors, while the ε_v (PV=NRT) was volumetric strain measured this equation. As shown in these figures, the measurements by the two methods are in reasonable agreement with each other in the change of volumetric strain during shear up to the axial strain of about 1~3 %. However, the ε_v (GS) - ε_a relation after axial strain of 1% is extremely different from ε_v (PV=NRT) - ε_a relation due to large deformation of the specimen. Accordingly, in CU tests, we do not adopt the ε_v calculated with the lateral displacements of the specimen measured by gap sensors as an experimental data in the range of ε_a of 1% or more from the viewpoint of the measurement precision.

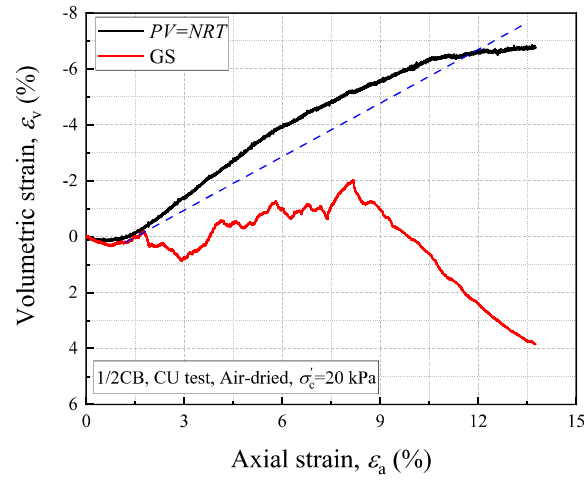


Figure 2-15 Comparison of axial strain for 1/2CB from different measuring methods in CU test.

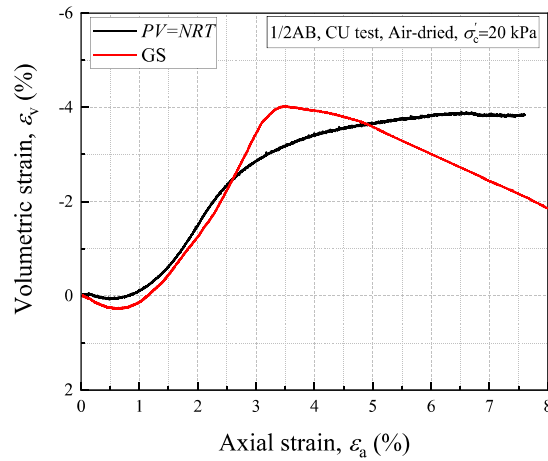


Figure 2-16 Comparison of axial strain for 1/2AB from different measuring methods in CU test.

3 TEST SAMPLE AND TEST METHODS

3.1 Test Sample

In general, most of Japanese railroad ballasts are composed of angular, crushed, and hard andesite stone. The grain size distribution of properly graded ballast in the Japanese railway standard ranges from 19 to 63 mm. Fresh ballast or called clean ballast (CB) and aged ballast (AB) are used as test samples, as shown in Figure 3-1. In Figure 3-2, the terms “1/1CB” and “1/1AB” refer to the full-scale fresh ballast and full-scale aged ballast obtained from actual railway tracks in Japan. Here, 1/1AB used in this study was extracted from heavily aged railway tracks. It should be noted that the field investigations on aged railway tracks in Japan showed that the fine fraction content (F_c) of in-situ aged ballast is generally between 0 and 15%. Therefore, the maximum fine fraction content (F_c) of 15% in the field investigation is used as the basis for the laboratory test condition. In this study, the small-scale fresh ballast (1/2CB) and the small-scale aged ballast (1/2AB) were employed as test samples due to the specimen size in the medium-size triaxial apparatus. As shown in Figure 3-2, both 1/2CB and 1/2AB have the half mean grain size of 1/1CB and 1/1AB with a parallel gradation.



Figure 3-1 Photos for test samples: (a) 1/2CB; (b) 1/2AB.

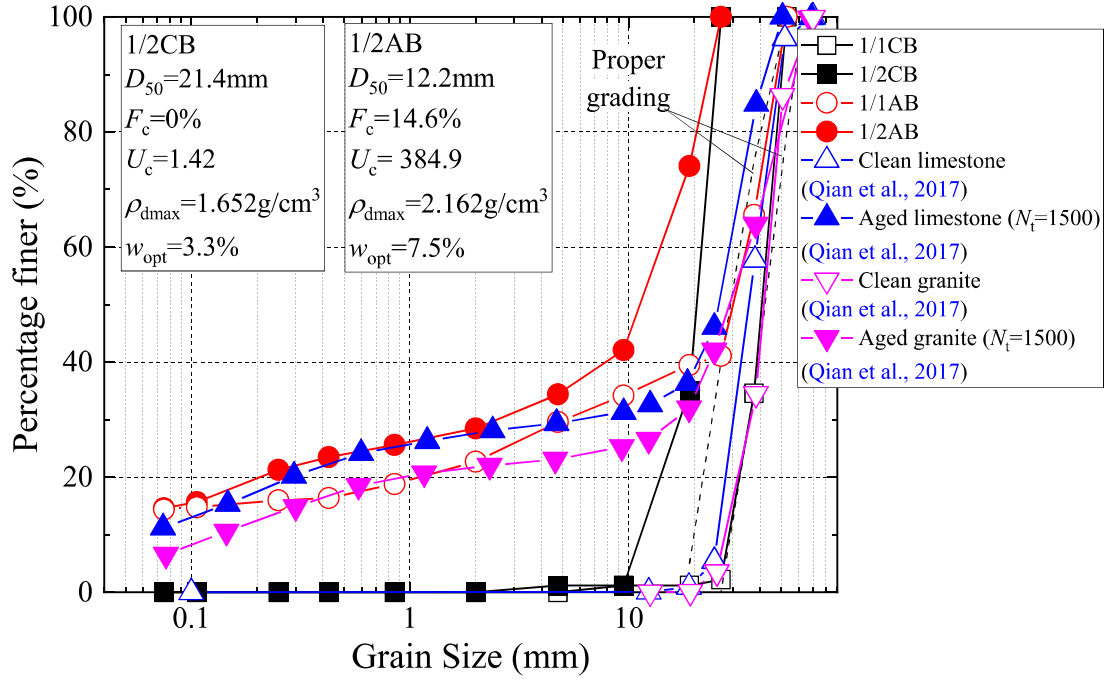


Figure 3-2 Grain size distributions of test samples.

3.1.1 Method of LAA test

In order to obtain the deteriorated ballast, the Los Angeles Abrasion (LAA) test was used. Generally, it is used for measuring durability and strength properties of granular samples. The LAA machine has revolving speed at 31~33 r/min, and is filled with samples and 12 steel balls. For the LAA test, previous studies (Guo et al., 2018; Qian et al., 2014) pointed out that the LAA test is a simple and valid method to reproduce aged ballast. Therefore, 1/2AB used in this study was prepared by the LAA test (Figure 3-3) to reproduce the grain size distribution and particle shape of 1/1AB. In this case, the fresh ballast was placed in the LAA equipment at a speed of 33 r/min for 60 minutes (1980 turns). The ballast particles were taken out and photographed every 10 minutes (330 turns) for image analysis. Here, the aged ballast with different number of turns was called N_{t0} (Number of turns (N_t) is equal to 0, and abrasion time (L) is equal to 0), N_{t330} (Number of turns (N_t) is equal to 330, and abrasion time (L) is equal to 10 mins), N_{t660} , N_{t990} , N_{t1320} , N_{t1650} , and N_{t1980} , respectively.



Figure 3-3 LAA test apparatus.

3.1.2 Method of particle shape analysis

The particle shape of the ballast before and after LAA tests was evaluated by the image analysis method. In this method, after converting the cross-section of the soil particles shot with a digital camera into X - Y coordinate values, the particle shape of the ballast was evaluated by two analytical methods, which used either the angularity index or the radius index (Kono et al., 2000). In general, the angularity index is regarded as an effective index for particle angularity, and the radius index is regarded as an effective index for sphericity. The angularity index is a function of the tangential line at P_x , which is the point advanced x from the reference point S , and the angle of reference line $\theta_{(x)}$, as shown in Figure 3-4. If the perimeter of the particle is L , the angularity index is normalized as shown in Eq. (3-1).

$$\theta_{N(x)} = \theta_{(x)} - 2\pi \left(\frac{x}{L} \right) \quad (3-1)$$

where $\theta_{N(x)}$ is the normalized angularity index. Since the normalized angularity index $\theta_{N(x)}$ is a periodic function of period L , it can be developed into a Fourier series, as shown in Eq. (3-2). The k -th amplitude spectrum can be expressed by Eq. (3-3):

$$\theta_{N(x)} = \frac{b_0}{2} + \sum_{k=1}^{\infty} \left\{ a_k \sin \left(\frac{2\pi kx}{L} \right) + b_k \cos \left(\frac{2\pi kx}{L} \right) \right\} \quad (3-2)$$

$$C_k = \sqrt{(a_k^2 + b_k^2)} \quad (3-3)$$

where a_k and b_k are

$$a_k = (2/L) \int_{x_0}^{x_0+L} f(x) \sin\left(\frac{2\pi kx}{L}\right) dx \quad k = 1, 2, \dots \quad (3-4)$$

$$b_k = (2/L) \int_{x_0}^{x_0+L} f(x) \cos\left(\frac{2\pi kx}{L}\right) dx \quad k = 1, 2, \dots \quad (3-5)$$

Kono et al. (2000) reported that the sum of the 3–20th amplitude spectra in Eqs. (3-4) & (3-5) indicates the degree of the angularity of the particle. Hence, this study assumed that the angularity of a particle shape was defined by the normalized angularity index, as given in Eq. (3-6):

$$\text{Angularity Index} = \sum_{k=3}^{20} C_k \quad (3-6)$$

On the other hand, the radius index is a function of the length $r_{(\alpha)}$ of the straight line that connects a certain point P_α on the contour and the center of gravity O , as shown in Figure 3-4. This is expressed by Eq. (3-7). The k -th amplitude spectrum is expressed by Eq. (3-8):

$$r_{(\alpha)} = \frac{b_0}{2} + \sum_{k=1}^{\infty} \{a_k \sin(k\alpha) + b_k \cos(k\alpha)\} \quad (3-7)$$

$$C_k = \sqrt{(a_k^2 + b_k^2)} \quad (3-8)$$

where a_k and b_k are

$$a_k = (1/\pi) \int_{\alpha_0}^{\alpha_0+2\pi} r_{(\alpha)} \sin(k\alpha) d\alpha \quad k = 0, 1, 2, \dots \quad (3-9)$$

$$b_k = (1/\pi) \int_{\alpha_0}^{\alpha_0+2\pi} r_{(\alpha)} \cos(k\alpha) d\alpha \quad k = 1, 2, \dots \quad (3-10)$$

Kono et al. (2000) reported that the ratio of the second-and zeroth-order amplitude spectra, shown in Eq. (3-8), indicates the degree of the aspect ratio of the particle. Then, the variation in aspect ratio was defined by the radius index, as given in Eq. (3-11):

$$\text{Radius Index} = \frac{C_2}{C_0} \quad (3-11)$$

For image analysis, Figure 3-5 shows the aged ballast particle extracted from the actual ballasted track, and Figure 3-6 shows the fresh and aged ballast particles before and after LAA test. In this study, the particle shape changes before and after the LAA test (Figure 3-6) will be analysis using angularity index and radius index as compared with those indexes of aged ballast particle extracted from the actual ballasted track (Figure 3-5).

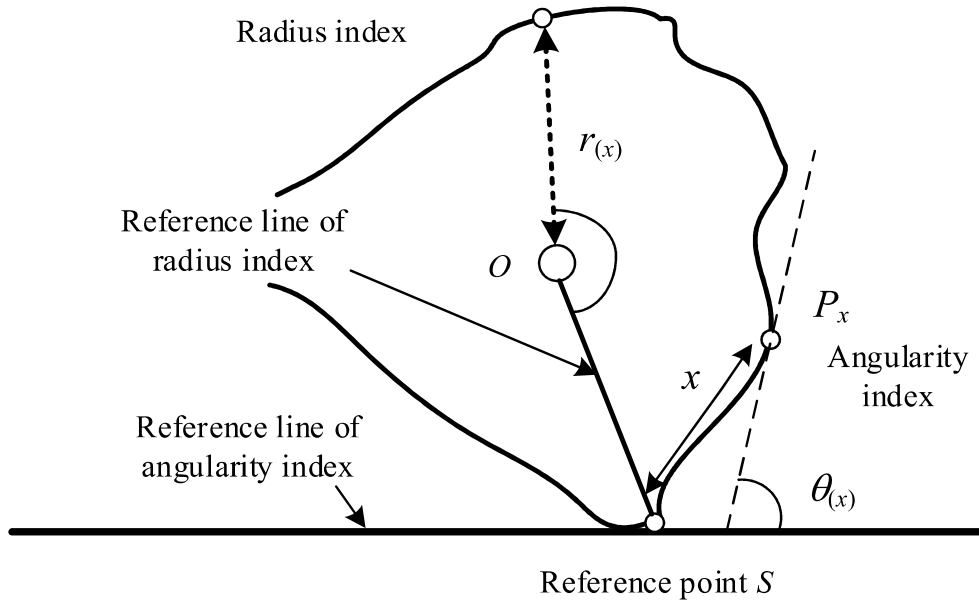


Figure 3-4 Figure of angularity and radius indexes (after Kono et al.(2000))



Figure 3-5 Aged ballast particle extracted from the actual ballasted track: (a) ballast particle in the track; (b) a typical ballast particle.



Figure 3-6 Fresh and aged ballast particles before and after LAA test: (a) ballast particle of L0;
(b) ballast particle of L60.

3.1.3 Effectiveness of LAA test for aged ballast

The effectiveness of the LAA test was verified by comparing the particle shape and fine fraction content (F_c) of 1/2AB prepared by the LAA test with those of 1/1AB ($F_c=4\%$) extracted from lightly aged railway tracks. In this case, the fine fraction content (F_c) of aged ballast was calculated by the sieve analysis method, and the particle shape was evaluated by the image analysis method.

3.1.3.1 Sieve analysis of particle size

The particle size of ballast was calculated by the sieve analysis method. For the particle size, Figure 3-7 shows the change in the fine fraction content (F_c) of 1/2AB with the number of turns (N_t) during the LAA test. It indicates that the fine fraction content (F_c) increases as the number of turns (N_t) increases. More importantly, the fine fraction content (F_c) of N_{1330} is slightly higher than that of 1/1AB ($F_c=4\%$) sampled from lightly aged ballasted tracks, while the fine fraction content (F_c) of N_{1650} is close to that of 1/1AB sampled from heavily aged ballasted tracks. Besides, it should be noted that the grain size distribution of 1/1AB in this study is close to that of the aged ballast (Limestone or granite, $N_t=1500$) produced from the original clean ballast with similar grain size distribution to 1/1CB by the LAA tests in Qian et al. (2017), as shown in Figure 3-2. It indicates that by changing the number of turns (N_t), the changes in fine fraction content (F_c) from fresh ballast to aged ballast with different degrees of aging can be effectively simulated by the LAA test.

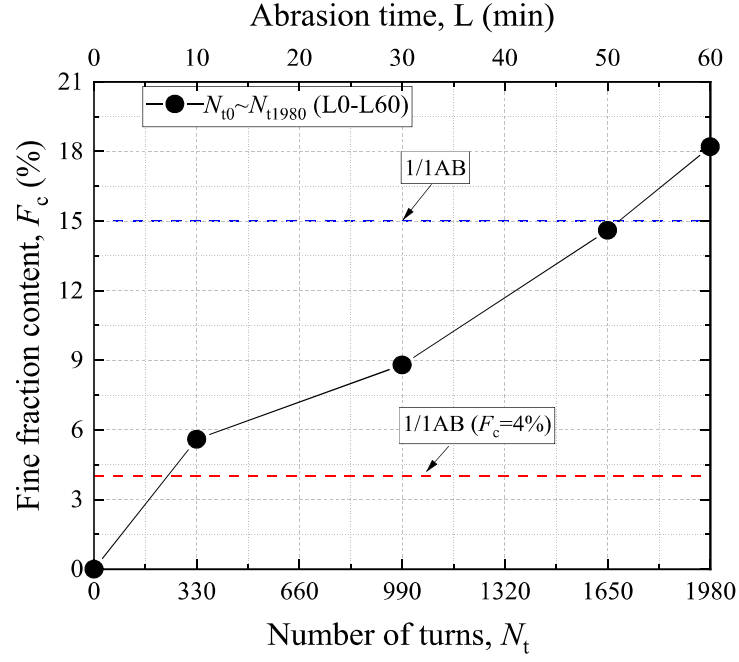


Figure 3-7 Comparisons between the fine fraction content of 1/2AB with number of turns (abrasion time) and those of 1/1AB.

In general, fresh ballast is a free draining material with large voids. However, the infiltration of fouling material reduces the voids and restricts water drainage. [Indraratna et al. \(2010\)](#) proposed the Void Contaminant Index (VCI) to quantitatively evaluate the degree of fouling for fouled ballast, which can consider the differences in void ratio, specific gravity, and gradation between the fresh ballast and the fouling material.

$$VCI = \frac{(1+e_f)}{e_b} \times \frac{G_{sb}}{G_{sf}} \times \frac{M_f}{M_b} \times 100 \quad (3-12)$$

where, e_b , G_{sb} , and M_b are void ratio, specific gravity, and dry mass of fresh ballast, respectively; e_f , G_{sf} , and M_f are void ratio, specific gravity, and dry mass of fouling material, respectively. Based on this definition, Figure 3-8 shows the relationships between the VCI and F_c or N_t of 1/2AB. It should be noted that in addition to the 1/2AB used in this study, 1/2AB with three other different F_c were also prepared by the LAA tests with different N_t to examine the effectiveness of the LAA test for reproducing the lightly or heavily aged ballast with different F_c or VCI . Regarding the calculation parameters of VCI , the specific gravities (G_{sb} , G_{sf}) of fresh ballast and fouling material are the same. Besides, the e_f and M_f of fouling material with different LAA test times can be calculated by assuming the dry

mass of fresh ballast with different LAA test times is same, and the dry mass of fouling material is same to that of fine fraction content (F_c). Based on this definition, Figure 3-8 shows the relationship between Void Contaminant Index (VCI) and fine fraction content (F_c) of 1/2AB. As can be seen in this figure, the VCI increases approximately linearly with the increase of F_c .

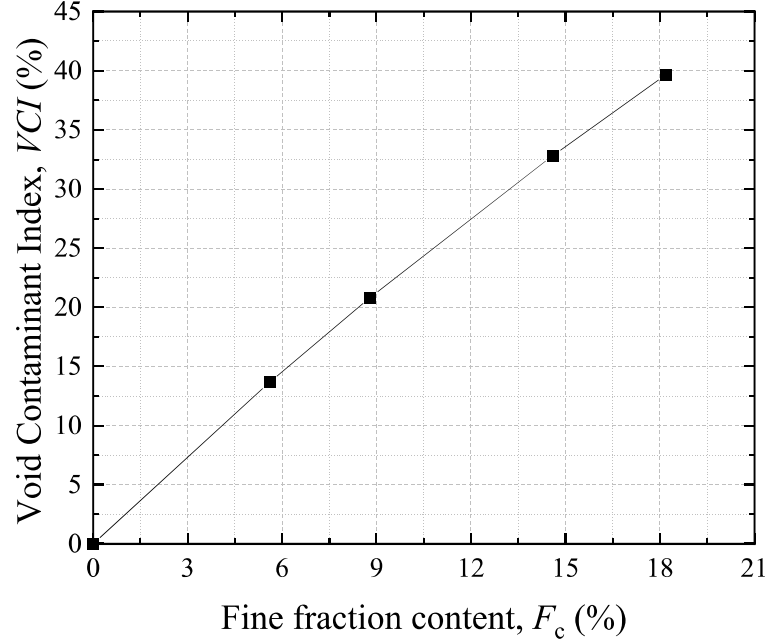


Figure 3-8 Relationship between VCI with F_c .

3.1.3.2 Image analysis of particle shape

The particle shape of the ballast was calculated by the image analysis method. For the particle shape, Figure 3-9 shows the comparisons between the average values of angularity index and radius index of 1/2AB (12 particles) with number of turns (N_t) and those of 1/1AB (12 particles) with $F_c=4\%$. It should be noted that the initial average diameter of these particles of 1/2AB before the LAA test is close to the mean grain size of 1/2CB ($D_{50}=21.4$ mm, see in Table 3-1), and the average diameter of these particles of 1/1AB ($F_c=4\%$) is close to the mean grain size of 1/1CB ($D_{50}=40.7$ mm, see in Table 3-1). The angularity index commonly decreases as number of turns (N_t) increases, while the radius index decreases slightly in the initial stage and remains almost constant after N_t of 660 turns. It indicates that the angular and rough ballast particles tend to be more rounded and

smoother due to the abrasion during the LAA test. Moreover, the angularity index and radius index of N_{t330} are close to those of 1/1AB ($F_c=4\%$). The image analysis results indicate that by changing the number of turns (N_t), the changes in particle shape from fresh ballast particles to aged ballast particles with different degrees of aging can be effectively simulated using the LAA test.

Table 3-1 Physical properties of test samples.

Sample	ρ_s (g/cm ³)	ρ_{dmax} (g/cm ³)	ρ_{dm} (g/cm ³)	D_{cm} (%)	w_{opt} (%)	D_{50} (mm)	U_c (mm/mm)	LL (%)	PL (%)	PI (%)	F_c (%)	k_s (m/s)
1/2CB	2.73	1.652	-	-	3.3	21.4	1.42	NP	-	-	0	6.51×10^{-2}
1/2AB	2.73	2.162	-	-	7.5	12.2	384.9	21.4	17.6	3.8	14.6	3.39×10^{-4}
1/1CB	2.70	1.682	1.581	94.0	3.0	40.7	1.33	NP	-	-	0	-
1/1AB	2.70	2.185	2.198	101.0	7.8	20.3	-	22.0	16.3	5.7	15.0	-

ρ_s : Soil specific density; ρ_{dmax} and w_{opt} are maximum dry density and optimum moisture content, obtained from compaction E-b method according to the standard of Japanese Geotechnical Society (JGS 0711-2009); ρ_{dm} : In-situ measured dry density; D_{cm} : In-situ measured degree of compaction; D_{50} : Mean grain size; U_c : Uniformity coefficient; LL , PL , and PI , are liquid limit, plastic limit, and plasticity index, respectively, obtained from test method for liquid and plastic limit of soils based on the standard of Japanese Geotechnical Society (JGS 0141-2009); F_c : Fine fraction content, means the percentage by weight of ballast sample passing the 0.075 mm sieve; k_s : Saturated coefficient of permeability at degree of compaction (D_c) of 94% for 1/2CB, and 101% for 1/2AB, fitted by the results of saturated permeability tests with different degree of compaction (D_c) shown in Figure 4-3.

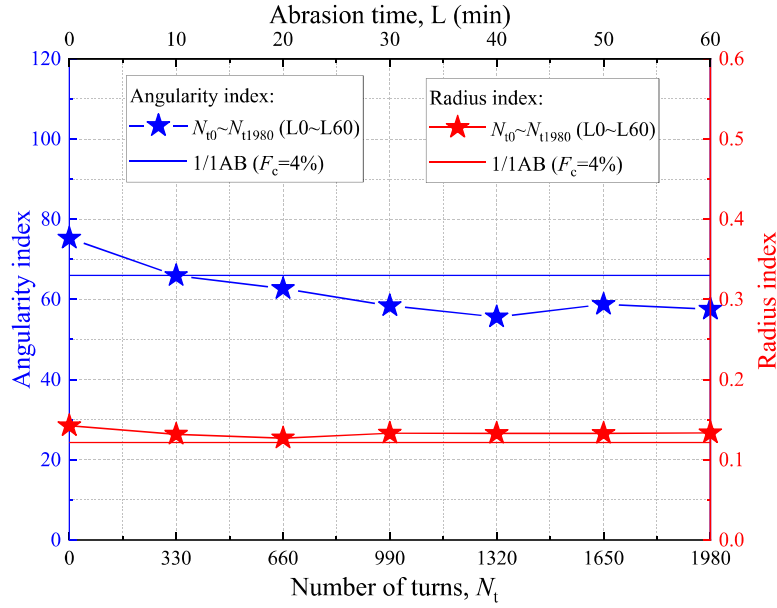


Figure 3-9 Comparisons between the angularity index and radius index of 1/2AB with number of turns (abrasion time) and those of 1/1AB.

The above results of sieve analysis and image analysis for particle size and particle shape prove that by changing the number of turns (N_t) the aged ballasts with different degrees of aging extracted from lightly or heavily aged ballasted tracks can be effectively simulated by the LAA test, though the effect of the LAA test on the grain size distribution under the same number of turns (N_t) depends on the physical properties (i.e., toughness and soundness) of stone samples. Accordingly, 1/2AB used in this study was prepared by the LAA test with N_{t1650} . Physical properties of test samples were listed in Table 3-1. As shown in Table 3-1, the liquid and plastic limit (LL , PL) of 1/2AB are close to those of 1/1AB. It indicates that soil properties of the fines content for 1/2AB are similar to those for 1/1AB, and both of them contain some plastic fines.

3.2 Water Retention Test

The preparation method for a sample in the water retention test was as follows: an air-dried sample (1/2CB, $w=1.69\%$; 1/2AB, $w=2.01\%$) was put into the mold in two layers step by step. Each layer was compacted by the vibrator to achieve the targeted degree of compaction (D_c) as shown in Table 3-1. Test conditions of water retention tests were listed

in Table 3-2. It is worth mentioning that the degree of compaction (D_c) values of water retention tests shown in Table 3-2 are slightly less than those targeted values shown in Table 3-1, regardless of test samples. The reason for this is considered due to the difference in the specimen height (H) as it applied the same compaction energy to the water retention test as the triaxial compression test where the degrees of compaction (D_c) are close to the targeted values.

Table 3-2 Water retention test conditions and results.

Sample	σ'_c (kPa)	ρ_{dc} (g/cm ³)	D_c (%)	S_{rs} (%)	S_{rr} (%)	R^2
1/2CB	20.0	1.456	89.0	100	6.6	0.998
1/2AB	20.0	2.018	93.0	100	46.1	0.965
1/1CB (Ishikawa et al., 2017)	20.0	1.535	93.0	100	6.5	0.999

For the process of water retention test, the test was commenced at an almost saturation condition with the matric suction (s) of a test sample being approximately zero. After the saturation process, the sample was one-dimensionally consolidated under a specified overburden pressure (σ_v) of 20 kPa in a drained condition until attaining an equilibrium condition. It is noted that the overburden pressure (σ_v) of 20 kPa was selected for this test to correspond to the effective confining pressure (σ'_c) of 20 kPa in triaxial compression tests. After the consolidation, the suction method was firstly adopted under the matric suctions (s) ranged from 0.1 to 4.0 kPa. At the initial condition, the water level in the beaker placed on the electronic force balance was adjusted to the same level as the top of the test specimen (Figure 2-1). Subsequently, the prescribed matric suction (s) was applied by continuously lowering the position of electronic force balance to reduce the pore water pressure (u_w) of the sample and keep the pore air pressure (u_a) unchanged. The change in water volume with the applied matric suction (s) was reading by an electronic force balance and used to calculate the degree of saturation of the sample. Upon attaining an equilibrium condition at matric suction (s) of 4 kPa, the pressure membrane method was adopted under the matric suctions (s) ranged from 4 to 100 kPa. In this case, the specimen was consolidated under a specific net normal stress (σ_{net}) of 20 kPa with overburden pressure (σ_v) of 220 kPa, the pore air pressure (u_a) of 200 kPa, and the pore water pressure (u_w) of 196 kPa. Here, σ_{net} is defined as $\sigma_{net} = \sigma_v - u_a$. It is noted that the definition of net normal stress (σ_{net}) in water retention tests is different from that in the triaxial compression test, which

will be explained in the later section. Subsequently, a predetermined matric suction (s) was applied by reducing the pore water pressure (u_w) while keeping the values of overburden pressure (σ_v) and pore air pressure (u_a) constant. when the equilibrium condition was reached, the water content related to the applied matric suction (s) was obtained by measuring the change in water volume between two successively applied matric suctions (s). This process was repeated to increase the matric suction (s) by gradually reducing the pore water pressure (u_w) until the matric suction (s) was equal to 100 kPa.

3.3 Permeability Test

3.3.1 Saturated permeability test

For the saturated permeability test, the preparation method of a sample in this test was as follows: an air-dried sample (1/2CB, $w=1.69\%$; 1/2AB, $w=2.01\%$) was put into the mold in five layers for a specimen height (H) of 300 mm or in two layers for a specimen height (H) of 125 mm step by step. Subsequently, each layer was compacted by a vibrator to achieve the targeted degree of compaction (D_c) similar to the actual track conditions shown in Table 3-1. However, the degree of compaction (D_c) of test specimens for saturated permeability tests ranges from 88% to 97% (Table 3-3) to check the influence of the degree of compaction (D_c) on the saturated permeability coefficient (k_s). Test conditions of the saturated permeability tests were shown in Table 3-3. The process of the saturated permeability test was as follows:

- (1) Keep the water level on the water supply and drainage constant using the permeability cylinder and the overflow tank.
- (2) Measure the water volume changes within a certain time at the outlet of the overflow tank.
- (3) The saturated permeability coefficient (k_s) can be calculated using the following equation.

$$k_s = \frac{H}{h} \times \frac{Q}{At} \quad (3-13)$$

where, k_s is saturated permeability coefficient; H is height of the specimen (cm); h stands for the difference in water level (cm³); Q is the water outflow (cm³/s); A is cross-sectional area of the test specimen (cm²); t is the measurement time (s).

Table 3-3 Saturated permeability test conditions and results.

Sample	H (mm)	ρ_{dc} (g/cm ³)	D_c (%)	k_s (m/s)
1/2CB	125	1.451	88	6.85×10^{-2}
1/2CB	125	1.544	93	5.94×10^{-2}
1/2CB	125	1.553	94	5.50×10^{-2}
1/2CB	125	1.589	96	5.33×10^{-2}
1/2CB	300	1.456	88	8.83×10^{-2}
1/2CB	300	1.528	92	7.41×10^{-2}
1/2CB	300	1.596	97	5.49×10^{-2}
1/2AB	125	1.903	88	7.50×10^{-4}
1/2AB	125	2.010	93	5.80×10^{-4}
1/2AB	125	2.076	96	4.51×10^{-4}
1/2AB	300	2.011	93	7.56×10^{-4}

3.3.2 Unsaturated permeability test

For the unsaturated permeability test, the size of a cylindrical specimen in this apparatus was 125 mm in height (H) and 150 mm in diameter (D). The preparation method of a sample in the test was as follows: an air-dried sample (1/2CB, $w=1.69\%$; 1/2AB, $w=2.01\%$) was put into the mold in in two layers for a specimen height (H) of 125 mm step by step. Subsequently, each layer was compacted by a vibrator to achieve the targeted degree of compaction (D_c) similar to the actual track conditions shown in Table 3-1. The process of the saturated permeability test was as follows:

- (1) Under a specific rainfall intensity (RI : 5~200 mm/h), water can be supplied to the upper surface of the sample through a rainfall simulator. When the weight of the sample becomes constant, the moisture content of the sample was measured. Thus, the unsaturated coefficient of permeability (k_{us}) of the soil under this water content was obtained.
- (2) Change the rainfall intensity (RI) and perform the unsaturated permeability test for the different water contents.

(3) When the rainfall intensity (RI) is ranged from 300~2500 mm/h, the unsaturated permeability test was performed using a spray head to apply the higher water content in the soil. When the weight of sample was unchanged, the unsaturated coefficient of permeability (k_{us}) of the soil with the higher water content can be obtained.

In this study, the unsaturated coefficient of permeability (k_{us}) of the soil can be calculated using the following equations.

Basic formula for vertical one-dimensional penetration,

$$Q = k_{us}A \left\{ \frac{\partial(\Psi/\gamma_w)}{\partial z} + 1 \right\} \quad (3-14)$$

It is assumed that the volumetric water content (θ) of the soil is constant in the steady state. In this case,

$$Q = \partial(\Psi/\gamma_w) = 0 \quad (3-15)$$

Therefore, the constant outflow flow rate (Q) in unsaturated conditions can be calculated as follows:

$$Q = k_{us} = \frac{Q}{A} \times \frac{1}{100} \quad (3-16)$$

where, k_{us} is unsaturated coefficient of permeability(m/s); Q is the constant outflow rate (cm^3/s); A is cross-sectional area of the test specimen (cm^2).

Here, the volumetric water content (θ) for unsaturated soils is defined as follows:

$$\theta = \frac{W-W_0}{\rho_w V} \quad (3-17)$$

where, θ is the volume water content; W is the total weight of unsaturated sample (g); W_0 is dry weight of specimen (g); ρ_w is the water density (g/cm^3); V is the specimen volume (cm^3).

Table 3-4 Unsaturated permeability test conditions

Sample	H (mm)	ρ_{dc} (g/cm^3)	D_c (%)
1/2CB	125	1.596	97
1/2AB	125	2.011	93

3.4 Triaxial Compression Test

Monotonic & cyclic loading triaxial compression tests were performed by a medium-size triaxial test apparatus in this study. For triaxial tests, the size of a cylindrical specimen in this apparatus is 300 mm in height (H) and 150 mm in diameter (D). The preparation method for a specimen in the test was as follows: an air-dried sample (1/2CB, $w=1.69\%$; 1/2AB, $w=2.01\%$) was put into the mold in five layers step by step. Each layer was compacted by the vibrator to achieve the targeted degrees of compaction (D_c) of 94% for 1/2CB, and 101% for 1/2AB (Table 3-1). The uniformity of a specimen was ensured because the variations in the initial dry density of the same sample were less than 1%. Test conditions of the monotonic triaxial compression tests were shown in Table 3-5, and test conditions of the cyclic loading triaxial compression tests were shown in Table 3-6.

3.4.1 Monotonic loading triaxial compression test

Monotonic loading triaxial compression (ML) tests were performed under three different water contents (air-dried, unsaturated ($s=5$ kPa), and saturated) according to standards of Japanese Geotechnical Society (JGS 0523-2009; JGS 0524-2009; JGS 0527-2009). It is noted that the matric suction (s) of 5 kPa was selected under the unsaturated condition because results of water retention test show that the water content of the two test samples hardly decreases when the matric suction (s) was greater than 5 kPa. Moreover, field measurements show that the in-situ confining pressure provided by the sleeper and ballast ranges from 10 to 60 kPa (Indraratna et al., 2013b; Suiker et al., 2005). Considering that the structure of the typical ballasted track in Japan is similar to that mentioned in these studies, the effective confining pressures (σ'_c) of 20 kPa and 40 kPa were selected in this study. Here, σ'_c is defined as $\sigma'_c = \sigma_c - u_w$.

Table 3-5 Monotonic loading triaxial compression test conditions and results.

Test name	Sample	Water content	$\dot{\epsilon}_a$ (%/min)	σ'_c , σ_{net} (kPa)	ρ_{dc} (g/cm ³)	D_c (%)	S_r (%)	q_{max} (kPa)	$\left(\frac{\sigma'_1}{\sigma'_3}\right)_{max}$
CD	1/2CB	Air-dried	0.05	20.0	1.562	94.5	4.42	237.5	12.88
CD	1/2CB	Air-dried	0.05	40.0	1.551	93.9	5.03	462.5	12.56
CD	1/2CB	$s=5$ kPa	0.05	20.0	1.555	94.1	7.23	195.0	10.75
CD	1/2CB	$s=5$ kPa	0.05	40.0	1.557	94.3	8.62	376.1	10.41

CD	1/2CB	Saturated	0.05	20.0	1.552	93.9	100	185.0	10.25
CD	1/2CB	Saturated	0.05	40.0	1.555	94.1	100	300.6	8.52
CU	1/2CB	Air-dried	0.05	20.0	1.561	94.5	4.46	-	11.93
CU	1/2CB	Air-dried	0.05	40.0	1.558	94.3	4.87	-	11.63
CU	1/2CB	$s=5\text{kPa}$	0.05	20.0	1.554	94.1	10.13	-	9.94
CU	1/2CB	$s=5\text{kPa}$	0.05	40.0	1.562	94.6	8.54	-	9.20
CU	1/2CB	Saturated	0.05	20.0	1.553	94.1	100	-	8.97
CU	1/2CB	Saturated	0.05	40.0	1.548	93.7	100	-	8.72
CD	1/2AB	Saturated	0.01	20.0	2.189	101.2	100	223.1	12.16
CD	1/2AB	Saturated	0.05	20.0	2.181	100.9	100	219.8	11.99
CD	1/2AB	Saturated	0.05	40.0	2.178	100.7	100	310.3	8.76
CU	1/2AB	Air-dried	0.05	20.0	2.213	102.4	21.79	-	15.50
CU	1/2AB	Air-dried	0.05	40.0	2.206	102.0	21.33	-	14.40
CU	1/2AB	$s=5\text{kPa}$	0.05	20.0	2.204	101.9	67.38	-	11.08
CU	1/2AB	Saturated	0.01	20.0	2.199	101.7	100	-	8.95
CU	1/2AB	Saturated	0.05	20.0	2.203	101.9	100	-	9.37
CU	1/2AB	Saturated	0.05	40.0	2.210	102.4	100	-	8.05

For the ML test, the consolidation process was conducted as follows. For the air-dried sample, the sample after preparation was isotropically consolidated under a designated effective confining pressure (σ'_c) of 20 or 40 kPa. In terms of the saturated sample, the de-aired water was firstly supplied from the bottom end of the sample, and then pore water pressure (u_w) value of 200 kPa was applied to ensure the pore water pressure coefficient B-value was above 0.96. Following the saturation, the sample was isotropically consolidated under a specified effective confining pressure (σ'_c) of 20 kPa or 40 kPa for 24 hours with confining pressure (σ_c) of 220 kPa or 240 kPa, and the pore water pressure (u_w) of 200 kPa. For the unsaturated sample, the de-aired water was immersed from the bottom of the sample until the initial degree of saturation reached about 90%. Afterwards, the sample was isotropically consolidated under a prescribed net normal stress (σ_{net}) of 20 kPa or 40 kPa for 24 hours with confining pressure (σ_c) of 220 kPa or 240 kPa, pore air pressure (u_a) of 200 kPa, and pore water pressure (u_w) of 200 kPa. Here, σ_{net} is defined as $\sigma_{\text{net}} = \sigma_c - u_a$. It is worth mentioning that the net normal stress (σ_{net}) in the unsaturated condition was similar to the effective confining pressure (σ'_c) in the saturated condition. In the air-dried condition, the effective confining pressure (σ'_c) was equal to the confining pressure (σ_c). Subsequently, an unsaturated sample with the matric suction (s) of 5 kPa was prepared by

reducing pore water pressure (u_w) value while keeping both confining pressure (σ_c) and pore air pressure (u_a) unchanged.

After that, when the equilibrium condition was reached in the last process, the samples with different water content were sheared by applying axial deviator stress (q) at the constant axial strain rate ($\dot{\epsilon}_a$) of 0.05%/min or 0.01%/min under fully drained conditions (CD test) or fully undrained conditions (CU test), and the shearing stage stopped when the axial strain reached 15% or when more than 3% axial strain has been reached after the maximum value of axial load reading regardless of water content according to the standard of the Japanese Geotechnical Society (JGS 0524-2009). Here, the q is defined as $q = \sigma'_1 - \sigma'_3$. It is noted that the axial strain rate ($\dot{\epsilon}_a$) of 0.05%/min was selected according to the triaxial compression test method for unsaturated soil based on the standard of the Japanese Geotechnical Society (JGS 0527-2009), while the axial strain rate ($\dot{\epsilon}_a$) of 0.01%/min was calculated based on the consolidation time of the aged ballast according to the standard of the Japanese Geotechnical Society (JGS 0524-2009). Moreover, for the unsaturated specimen in the CD test both pore air pressure (u_a) and pore water pressure (u_w) are allowed to drain, whereas in the CU test both pore air pressure (u_a) and pore water pressure (u_w) are undrained.

3.4.2 Cyclic loading triaxial compression test

The cyclic loading triaxial compression (CL) test was performed by a medium-size triaxial apparatus (Figure 2-5). The pressure membrane method was adopted in CL tests for unsaturated specimens to apply the matric suction (s). The preparation method of a cylindrical specimen ($H=300$ mm, $D=150$ mm) in this test was as follows: an air-dried sample (1/2CB, $w=1.69\%$; 1/2AB, $w=2.01\%$) was put into the mold in five layers step by step. Each layer was compacted by the vibrator to achieve the targeted D_c values similar to the actual track conditions. The uniformity of a specimen was ensured because the variations in the initial dry density of the same sample were less than 1%. The test conditions of the CL tests in this study were shown in Table 3-6. As shown in Table 3-6, CL tests were performed under three different water contents (air-dried, unsaturated ($s=5$ kPa), and saturated) according to the standards of Japanese Geotechnical Society (JGS 0523-2009; JGS 0524-2009; JGS 0527-2009). It is noted that the s of 5 kPa was selected under the unsaturated condition because results of water retention tests showed that the

water content of the two test samples hardly decreases when the s was greater than 5 kPa. Besides, the effective confining pressure (σ'_c) of 20 kPa was selected in CL test, which was referred to that in monotonic loading triaxial compression (ML) test as shown in Table 3-5.

Table 3-6 Cyclic loading triaxial compression test conditions.

Sample	Drainage condition	Water content	$\sigma'_c, \sigma_{\text{net}}$ (kPa)	ρ_{dc} (g/cm ³)	D_c (%)	S_r (%)
1/2CB	CD test	Air-dried	20.0	1.571	95.1	4.22
1/2CB	CD test	Unsaturated ($s=5$ kPa)	20.0	1.539	93.2	10.62
1/2CB	CD test	Saturated	20.0	1.541	93.3	100
1/2AB	CD test	Air-dried	20.0	2.220	101.6	21.7
1/2AB	CD test	Unsaturated ($s=5$ kPa)	20.0	2.198	100.6	67.2
1/2AB	CD test	Saturated	20.0	2.205	100.9	100
1/2AB	CU test	Air-dried	20.0	2.196	100.5	21.3
1/2AB	CU test	Unsaturated ($s=5$ kPa)	20.0	2.195	100.5	68.5
1/2AB	CU test	Saturated	20.0	2.199	100.6	100

4 HYDRAULIC PROPERTIES OF UNSATURATED BALLAST

4.1 Soil-Water Characteristic Curve

4.1.1 Effect of specimen height

To discuss the influence of specimen height (H) on the water retentivity as well as confirm the validity of the newly developed water retention apparatus for coarse granular sample, the comparing results of SWCCs for the ballast based on this newly developed water retention apparatus with the results based on medium-size triaxial apparatus, e.g., [Matsutani et al. \(2017\)](#) are shown in Figure 4-1. It can be seen that the SWCC curves obtained by the two apparatuses are similar regardless of the samples, although the sample height of this newly developed apparatus is only 1/3 of that of the medium-size triaxial test apparatus for the water retention test. It can be concluded that the specimen height (H) has little effect on the water retention. From the above observations, it can be confirmed that the newly developed water retention apparatus for coarse-grained sample is effective.

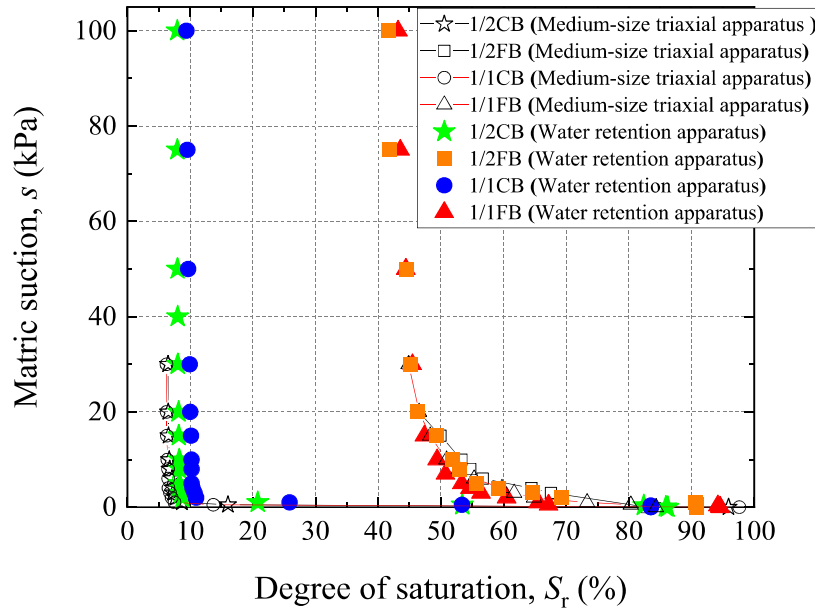


Figure 4-1 Comparing results of SWCCs between water retention test apparatus and medium-size triaxial apparatus (after [Aoyagi et al., 2019](#)).

4.1.2 Effect of fine fraction content

Figure 4-2 illustrates the soil water characteristic curves (SWCCs) of tested samples obtained from the water retention test and fitted results by the Van Genuchten model (Van Genuchten, 1980). As the results, the SWCC of 1/1CB is close to that of 1/2CB with a parallel gradation. More importantly, the SWCC of 1/2AB is located on the right side of 1/2CB, and the residual degree of saturation (S_{rr}), which can be calculated by assumption that the maximum degree of saturations (S_{rs}) of the test sample is 100%, of 1/2AB ($S_{rr}=46.1\%$) is much higher than that of 1/2CB ($S_{rr}=6.6\%$). It is considered that the aged ballast has a better grade and higher fine fraction content than the fresh ballast. These findings present that the presence of fine fraction content in aged ballast remarkably improves the water retentivity compared with fresh ballast.

Table 4-1 Regression parameters of Van Genuchten model.

Van Genuchten model		Samples		
Equation	Parameters	1/2CB	1/2AB	1/1AB
$S_e = \left[\frac{1}{1+(\alpha s)^n} \right]^m$; ($m=1-1/n$)	S_{rs} (%)	100	100	100
	S_{rr} (%)	6.6	46.1	6.5
	α	1.9367	0.0786	1.9367
	n	4.1661	1.3406	4.1661
	R^2	0.998	0.965	0.999

α , n , and m are fitted parameters of the Van Genuchten model. S_e is the normalized water content between the saturated state and the residual degree of saturation (Brooks and Corey, 1964).

$$S_e = \frac{S_r - S_{rr}}{S_{rs} - S_{rr}} \quad (4-1)$$

where, S_{rs} is the degree of saturation under saturated conditions, and S_{rr} is the residual degree of saturation. It is noted that the effective degree of saturation (S_e) of the soil in the air-dried state in this study is considered as zero.

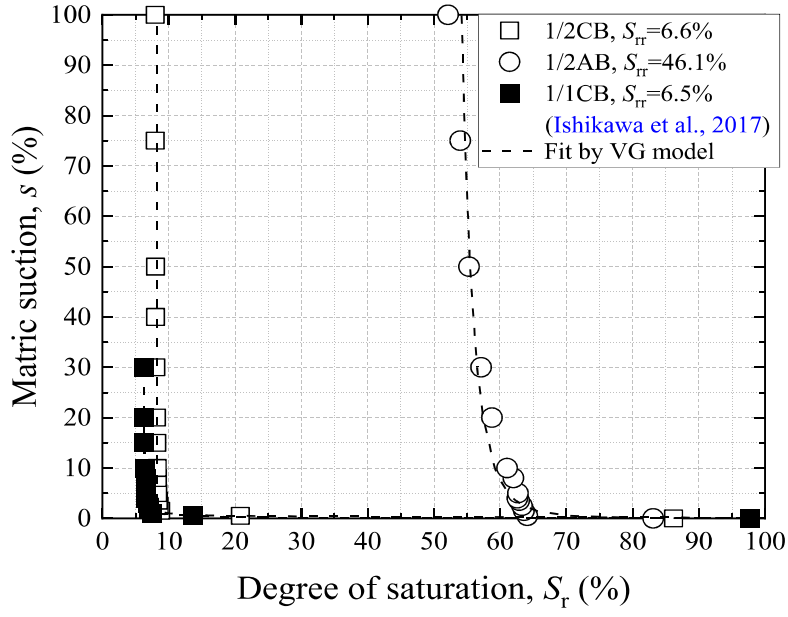


Figure 4-2 Soil water characteristic curves of test samples.

4.2 Saturated Coefficient of Permeability

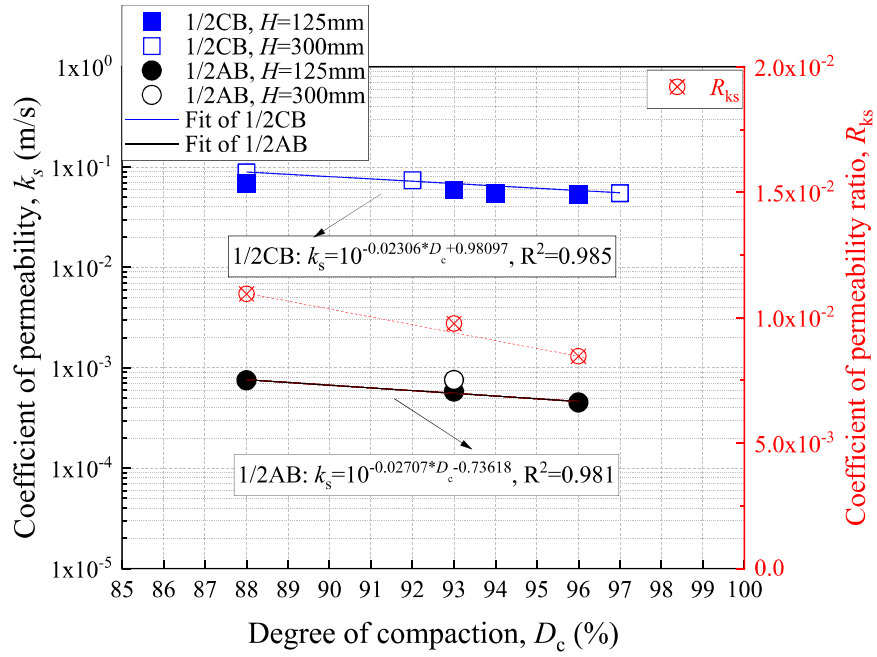


Figure 4-3 Variations of saturated coefficient of permeability with degree of compaction.

For saturated permeability test, the saturated coefficients of permeability (k_s) of 1/2CB and 1/2AB were obtained from the saturated permeability tests by considering the effects of specimen height (H), degree of saturation (D_c), and fine fraction content (F_c). Figure 4-3 shows the saturated coefficients of permeability (k_s) of two test samples with different degrees of compaction (D_c) and specimen heights (H).

4.2.1 Effect of specimen height

In Figure 4-3, when comparing the test results with different specimen heights (H), the saturated coefficients of permeability (k_s) of both samples with the specimen height (H) of 125 mm are close to those of 300 mm, thus indicating that the specimen height (H) has little effect on the saturated coefficient of permeability (k_s), regardless of test samples. Therefore, the reliability of test results with the specimen height (H) value of 125 mm is verified by a comparison of that of 300 mm recommended by the standard (JGS 0311-2009).

4.2.2 Effect of degree of compaction

The saturated coefficients of permeability (k_s) of two test samples with different degrees of compaction (D_c) are shown in Figure 4-3. As the results, the saturated coefficients of permeability (k_s) for all specimens decrease slightly with the increase of degree of compaction (D_c), regardless of samples and specimen heights (H). Moreover, the coefficients of permeability ratio (R_{ks}) for both samples decrease linearly with the increase of degree of compaction (D_c). The above results show that the saturated coefficient of permeability (k_s) of the ballast is reduced with the increasing degree of compaction (D_c), irrespective of test samples.

4.2.3 Effect of fine fraction content

To evaluate the effect of fine fraction content (F_c) on the saturated coefficient of permeability (k_s), Figure 4-3 also compares the saturated coefficients of permeability (k_s) of 1/2CB ($F_c=0\%$) and 1/2AB ($F_c=15\%$) under the same test condition. As the results, when the degree of compaction (D_c) is constant, the saturated coefficient of permeability (k_s), of 1/2CB is much greater than that of 1/2AB. For example, the coefficient of permeability ratio (R_{ks}), that is a ratio of saturated coefficient of permeability (k_s), for 1/2AB to saturated coefficient of permeability (k_s), for 1/2CB under the same degree of compaction (D_c)

condition, is about 8.1×10^{-3} at the degree of compaction (D_c) of 94%. It indicates that the saturated coefficient of permeability (k_s) of the aged ballast is remarkably reduced as compared with fresh ballast due to the presence of the fine fraction content.

The above results show that both degree of compaction (D_c) and fine fraction content (F_c) have an influence on the saturated coefficient of permeability (k_s), while it is hardly affected by the specimen height (H). When other test conditions are constant, the saturated coefficient of permeability (k_s) is remarkably reduced with the increase of the fine fraction content, though it is also slightly affected by the degree of compaction (D_c).

4.3 Unsaturated Coefficient of Permeability

Test results of the unsaturated coefficient of permeability (k_{us}) were fitted by the Van Genuchten model as follows:

$$k_{us} = k_s S_e^{1/2} \left(1 - (1 - S_e^{1/m})^m \right)^2 \quad (4-2)$$

where, k_s is the saturated coefficient of permeability, which can be obtained from the saturated permeability test as described in Section 4.2. k_s value of 1/2CB is 6.51×10^{-2} m/s, and k_s value of 1/2AB is 3.39×10^{-4} m/s.

Figure 4-4 shows the difference of unsaturated coefficient of permeability (k_{us}) between 1/2CB and 1/2AB. Test results were also fitted by the Van Genuchten model (Van Genuchten, 1980) as shown in Eq. (4-1). It is noted that in this equation, the value of input parameters m which can be obtained by fitting water retention test results. It demonstrates that the unsaturated coefficient of permeability (k_{us}) is rapidly decrease with the decrease of the volumetric water content, regardless of test samples. More importantly, the measured values under different water content can be well fitted by the Van Genuchten model (Van Genuchten, 1980) for both samples. Besides, when considering the effect of aging, the unsaturated coefficient of permeability (k_{us}) of aged ballast is much lower than that of clean ballast, though the relative permeability coefficient (k_{us}/k_s) of fresh ballast is more significantly affected by the water content.

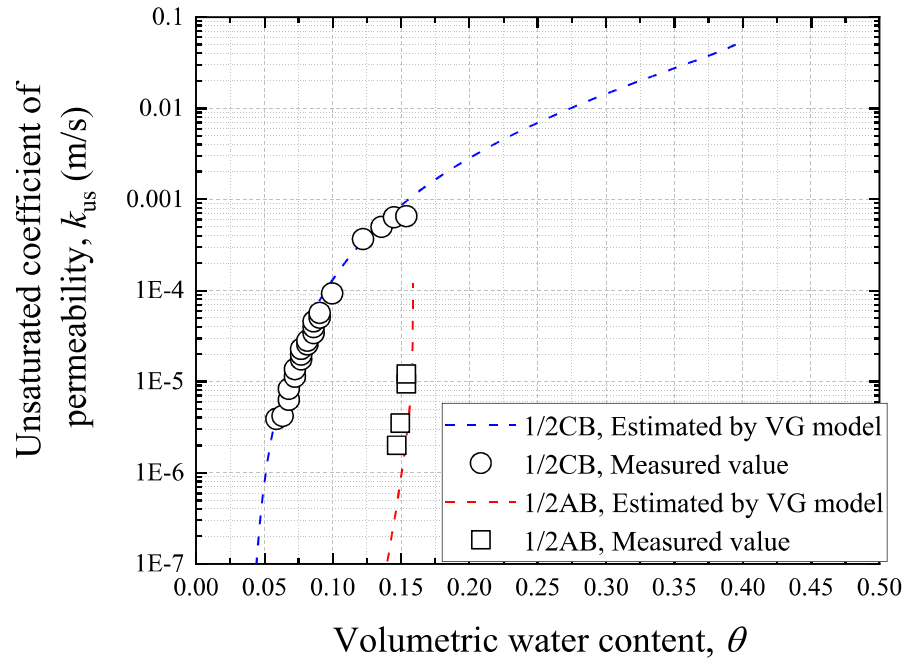


Figure 4-4 Variation of unsaturated coefficient of permeability with volumetric water content.

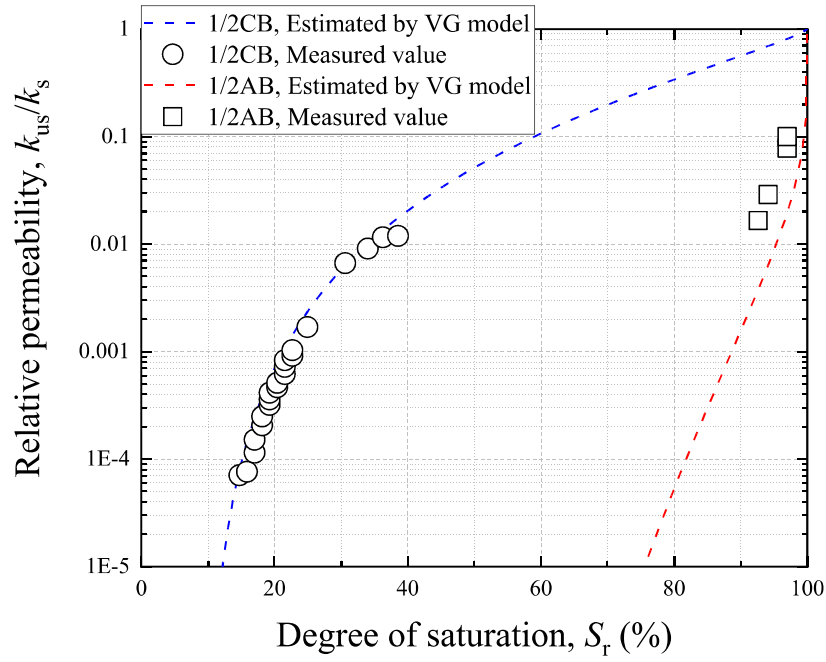


Figure 4-5 Variation of relative permeability with degree of saturation.

4.4 Summary

Hydraulic properties of railroad ballast are seriously affected by aging. The results of the saturated/unsaturated permeability tests and the water retention test prove that the aging significantly affects the permeability and water retentivity of ballast. The mainly findings of the chapter are as follows:

- (1) Results of water retention test with different specimen heights indicate that the water retentivity of test samples was affected by the specimen height. However, the presence of fine fraction content in aged ballast remarkably increases the water retentivity compared with fresh ballast.
- (2) Results of saturated permeability test show that both degree of compaction and fine fraction content have an influence on the saturated coefficient of permeability, while it is hardly affected by the specimen height. When other test conditions are constant, the saturated coefficient of permeability is remarkably reduced with the increase of fine fraction content, though it is slightly decrease as the degree of compaction increases.
- (3) Results of unsaturated permeability test prove that the unsaturated coefficient of permeability of aged ballast is much lower than that of clean ballast, though the relative permeability coefficient of fresh ballast is more significantly affected by the water content.

5 INFLUENCE OF AGING ON SHEAR STRENGTH OF UNSATURATED BALLAST

5.1 Shear Behavior

5.1.1 Shear behavior of fresh ballast

Figure 5-1 presents the variations of deviator stress (q), and volumetric strain (ε_v) with axial strain (ε_a) for 1/2CB in the CD and CU tests. In the CU test (Figure 5-1(a)), with the increase of axial strain (ε_a), the deviator stress (q) rapidly rises to the peak strength ($q_{\max}$) at axial strain (ε_a) of from 4% to 6%, before gradually decreasing to the ultimate strength at axial strain (ε_a) of about 12%. Accordingly, the compacted 1/2CB under the drained condition exhibits a strain-softening behavior after the peak strength, regardless of the confining pressure and water content. However, as the water content increases, the peak strength ($q_{\max}$) under different confining pressures is close to ultimate strength. For the dilatancy behavior, the volume of the specimen is first compressed, followed by dilation with the increase in axial strain (ε_a) during the shear process. when the effective confining pressure (σ'_c) is unchanged, the specimen under higher degree of saturation shows a downward trend in the dilation. Guo and Su (2007) pointed out that the shear resistance of the granular soil is caused by interparticle friction and dilation. Thus, the volumetric strain (ε_v) of the air-dried sample is greater than those of the other samples, indicating that the air-dried sample requires greater deviator stress to break the interlock between the particles.

In the CU test (Figure 5-1(b)), before the axial strain (ε_a) reaches about 3%, the q value of the CU test is smaller than that of the CD test, and after that, the deviator stress (q) in the CU test rises to the peak strength ($q_{\max}$) value at $\varepsilon_a=7\%$ or over and its peak strength ($q_{\max}$) value is greater than that in the CD test, irrespective of the confining pressure and water content. This trend can be explained as the change in effective confining pressure (σ'_c) caused by the generation of excess pore water pressure in the CU test, which is further discussed in Figure 5-2. Moreover, when examining the effect of water content on the q - ε_a relationship, the peak strength ($q_{\max}$) in the CD test increases in order of saturated, unsaturated, and air-dried specimens as shown in Figure 5-1(a). While the peak strength

($q_{\max}$) for the saturated specimen in the CU test is larger than those for air-dried and unsaturated specimens, regardless of confining pressure as shown in Figure 5-1(b). The reason for this trend is that the effective confining pressure (σ'_c) values of specimens with different water contents are not constant in CU tests (Figure 5-2(b)).

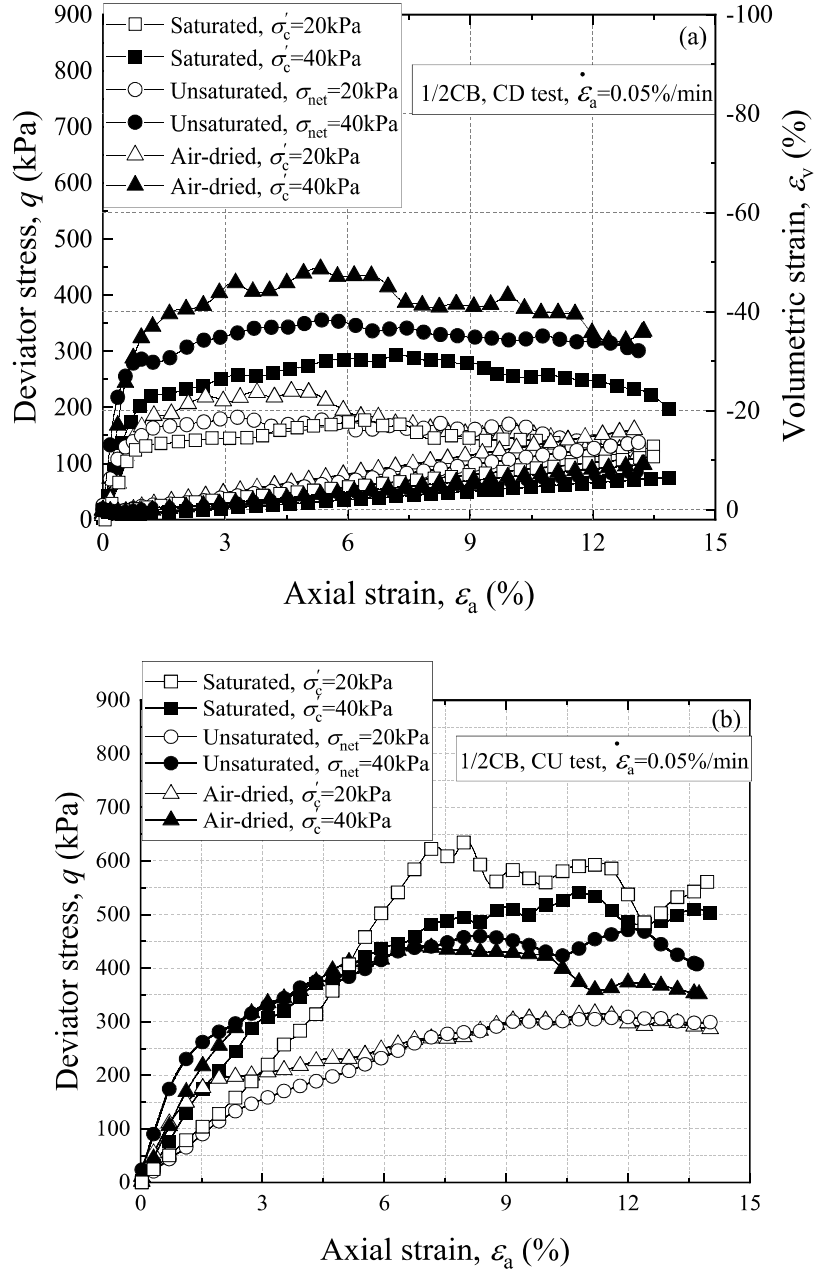


Figure 5-1 Relationship between deviator stress/volumetric strain and axial strains for 1/2CB with different water contents: (a) CD test; (b) CU test.

To explain the different trends of the peak strength ($q_{\max}$) with water content in the CU and CD tests, Figure 5-2 shows the changes in the pore water pressure (u_w), pore air pressure (u_a), matric suction (s), and effective confining pressure (σ'_c) in CU tests. It is noted that the initial value of pore air pressure (u_a) in the air-dried condition was increased from 0 to 200 kPa to compare the pore water pressure (u_w) and pore air pressure (u_a) in saturated and unsaturated conditions. As can be seen in Figure 5-2(a), the positive excess pore water and air pressures are generated in the initial stage, and gradually decreased to the negative values, which is consistent with the volume changes of specimens in CD tests shown in Figure 5-1(a). Besides, the variation trends of pore air pressure (u_a) and pore water pressure (u_w) are similar and continuous throughout the shearing process, which is consistent with previous findings of CU tests (Oka et al., 2010). This proves the reliability of the measurement system for pore air pressure (u_a) used in this study. Moreover, the matric suction (s) shows a whole decreasing trend until finally stabilizing at about 2 kPa. The final values of matric suction (s) are smaller than the initial values regardless of confining pressures, this may be caused by the pore air pressure (u_a) slowly dissipating into the water during the shearing process. It is worth noting that the matric suction (s) fluctuates in the range of 3% to 6% of axial strain (ε_a) under the net normal stress (σ'_{net}) value of 20 kPa, which may be due to changes in pore air pressure at this stage close to the measurement accuracy of the transducer.

Furthermore, in Figure 5-2(b), the effective confining pressure (σ'_c) of all specimens shows a whole trend of increasing with the increase of axial strain (ε_a) during the shearing process, though a decreasing trend in the initial stage. To be specific, when the axial strain (ε_a) of the specimen is greater than 1%, the effective confining pressure (σ'_c) continues to increase. This is the reason why the peak strength ($q_{\max}$) in the CU test under the same initial test condition is significantly higher than that in the CD test. Besides, when comparing the maximum values of effective confining pressure (σ'_c) under different water content conditions, the maximum value of effective confining pressure (σ'_c) increases by the sequences in air-dried, unsaturated, and saturated specimens. The change in effective confining pressure (σ'_c) in the CU test is related to the dilatancy trend in the CD test (Figure 5-1(a)). As illustrated in Figure 5-1(a), the positive dilatancy trends become stronger by the sequences in saturated, unsaturated, and air-dried specimens. It means that the dilatancy

trends of test specimens with different water contents are opposite to changes in effective confining pressure (σ'_c).

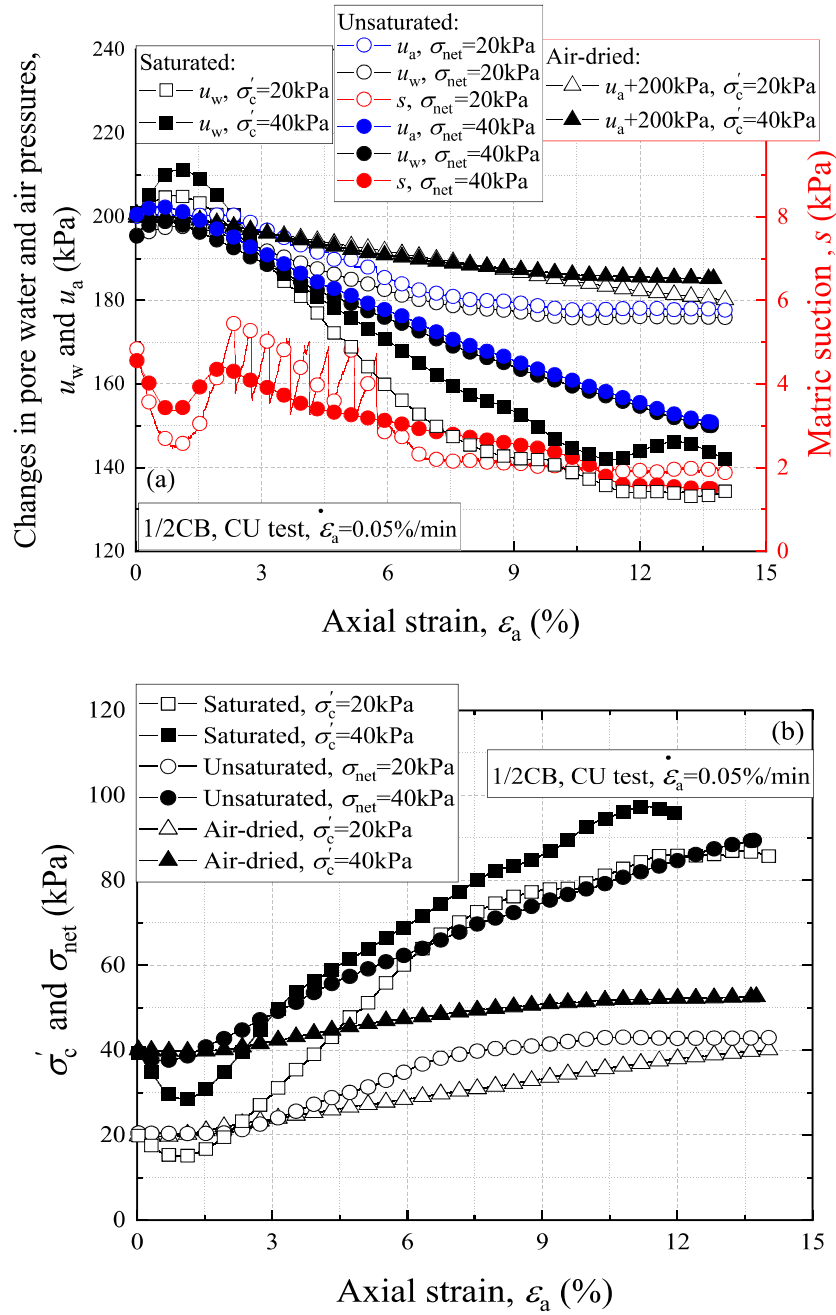


Figure 5-2 Changes in pore water pressure, pore air pressure, and effective confining pressure for 1/2CB:(a) pore water and air pressures; (b) effective confining pressure.

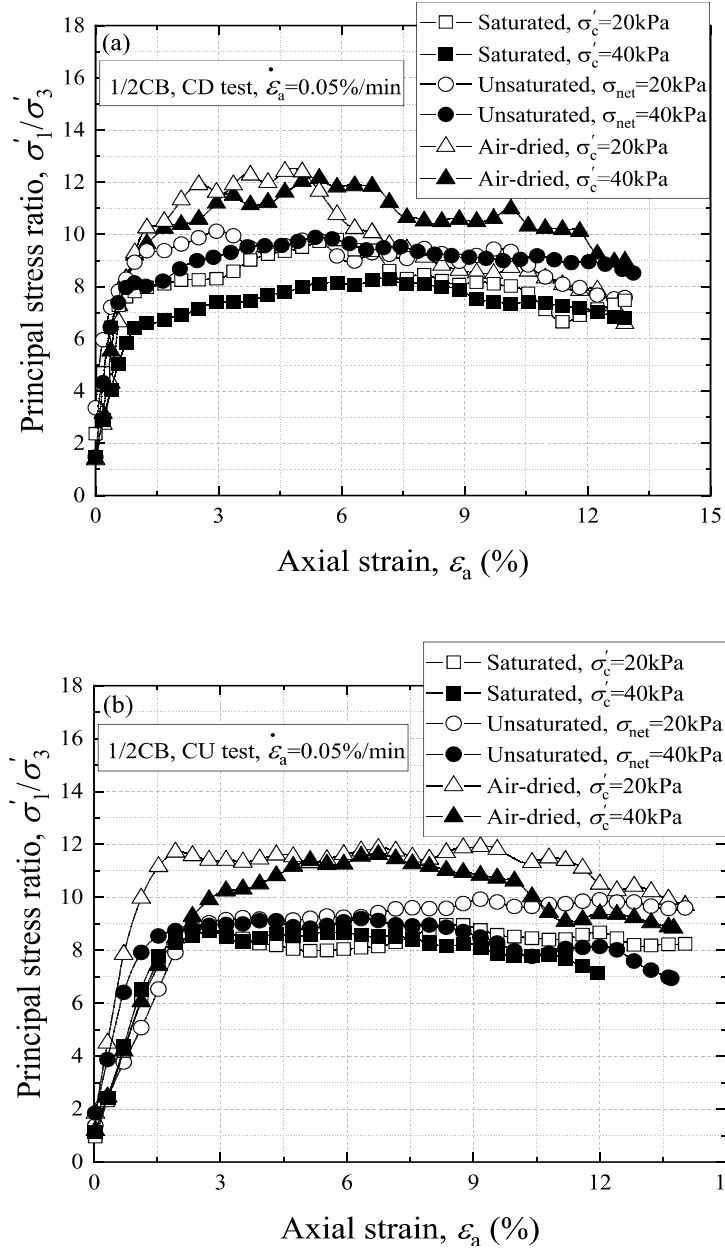


Figure 5-3 Relationship between principal stress ratio and axial strain for 1/2CB: (a) CD test; (b) CU test.

Considering that the effective confining pressure (σ'_c) is not constant in the CU test, Figure 5-3 compares the relationship between principal stress ratio (σ'_1/σ'_3) and axial strain (ϵ_a) of 1/2CB with different water contents in the CD and CU tests. Comparing results of the CD and CU tests, when the initial effective confining pressure (σ'_c) is 20 kPa, the maximum principal stress ratios ($(\sigma'_1/\sigma'_3)_{\max}$) of air-dried, unsaturated, and saturated

specimens in CU tests are 11.93, 9.94, and 8.97, while those in CD tests are 12.88, 10.75, and 10.25, respectively. When the initial effective confining pressure (σ'_c) is 40 kPa, the $(\sigma'_1/\sigma'_3)_{\max}$ values of air-dried, unsaturated, and saturated specimens in CU tests are 11.63, 9.20, and 8.72, while those in CD tests are 12.56, 10.41, and 8.52, respectively. These results demonstrate that when the water content is unchanged, the $(\sigma'_1/\sigma'_3)_{\max}$ of fresh ballast are almost the same in the CD and CU tests, regardless of confining pressures. More importantly, the higher degree of saturation of the soil reduces the $(\sigma'_1/\sigma'_3)_{\max}$, regardless of confining pressure and drainage conditions.

5.1.2 Shear behavior of aged ballast

The relationship between the principal stress ratio (σ'_1/σ'_3) and axial strain (ϵ_a) of 1/2AB under different test conditions is represented in Figure 5-4. It is noted that due to the limitation of the load range of the test apparatus, the CD tests for 1/2AB under air-dried and unsaturated conditions are not tested in this study. As illustrated in Figure 5-4(b), with the increase of axial strain (ϵ_a), the principal stress ratio (σ'_1/σ'_3) of 1/2AB sharply increases to the maximum value at the axial strain (ϵ_a) of about 0.5% and before gradually decreasing to the ultimate strength until the end of shearing process. It is noted that only the air-dried specimen shows brittle failure where a significant strain-softening phenomenon appears after the axial strain (ϵ_a) of 0.5%. This can be explained by Guo and Su (2007)'s indications that the shear strength of granular samples was provided by interparticle friction, particle rearrangement, and dilation as follows. The particle interlock restricts the sliding and rotation between particles, while the dilation tend reduces this interlock. Compared with other conditions, the air-dried sample has a larger dilation trend and therefore requires greater stress to break the interlock between particles. Besides, when comparing the relationship between the principal stress ratio (σ'_1/σ'_3) and axial strain (ϵ_a) of 1/2AB and 1/2CB, the trend between the principal stress ratio (σ'_1/σ'_3) and the axial strain (ϵ_a) of 1/2AB is significantly different from that of 1/2CB (Figure 5-3). To be specific, the $(\sigma'_1/\sigma'_3)_{\max}$ of 1/2AB appears at about axial strain (ϵ_a) of 0.5% in the air-dried condition, and then a significant strain-softening trend appears until the end of the shearing process. However, the $(\sigma'_1/\sigma'_3)_{\max}$ of 1/2CB appears at the axial strain (ϵ_a) of 6% or over, and there is no significant strain-softening phenomenon until the end of shear. This strain-softening phenomenon of aged ballast with a high initial dry density is similar to that of the previous

study on dense sand (Taylor et al., 2011). Moreover, when comparing the results of CD and CU tests under the saturated condition as shown in Figure 5-4(a) and Figure 5-4(b), it is unexpected that the principal stress ratio (σ'_1/σ'_3) of 1/2AB in CD tests are different from those in CU tests, regardless of confining pressures.

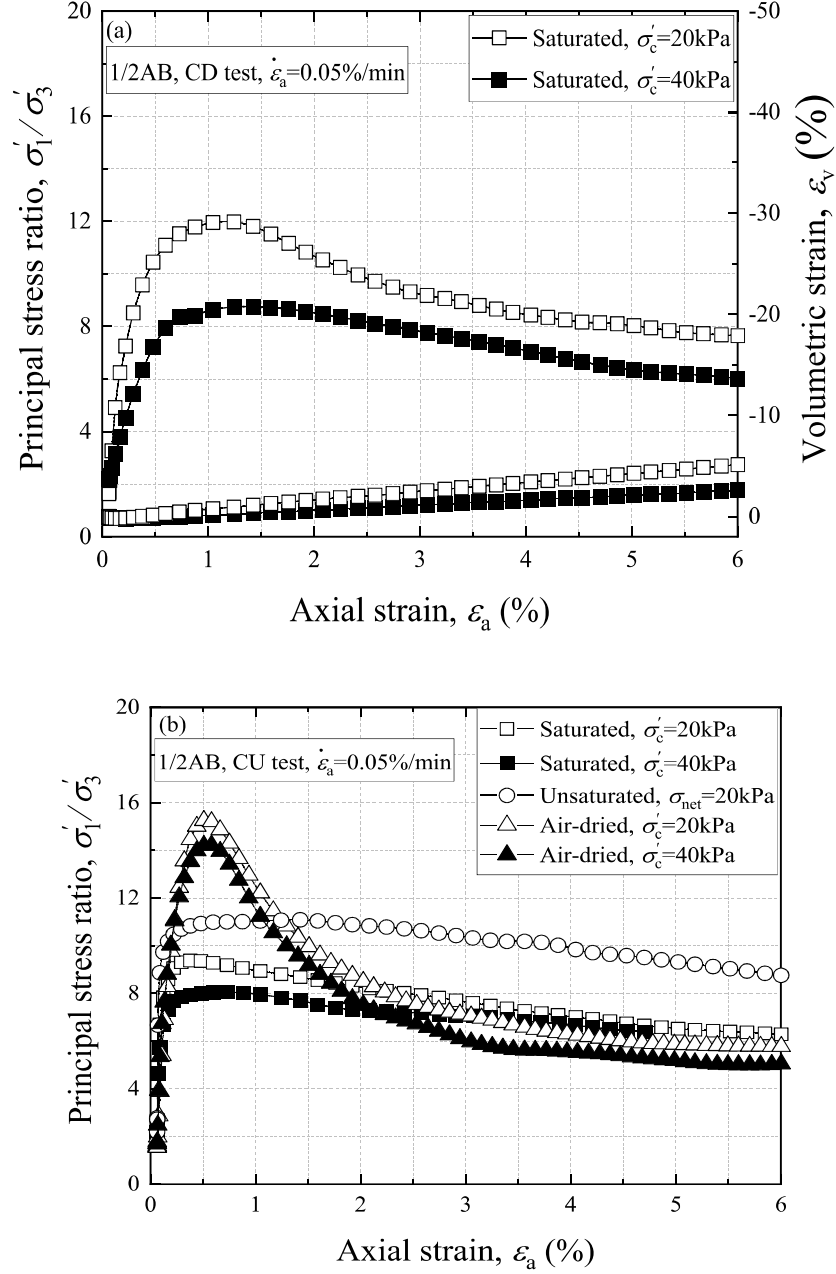


Figure 5-4 Relationship between principal stress ratio and axial strain for 1/2AB: (a) CD test; (b) CU test.

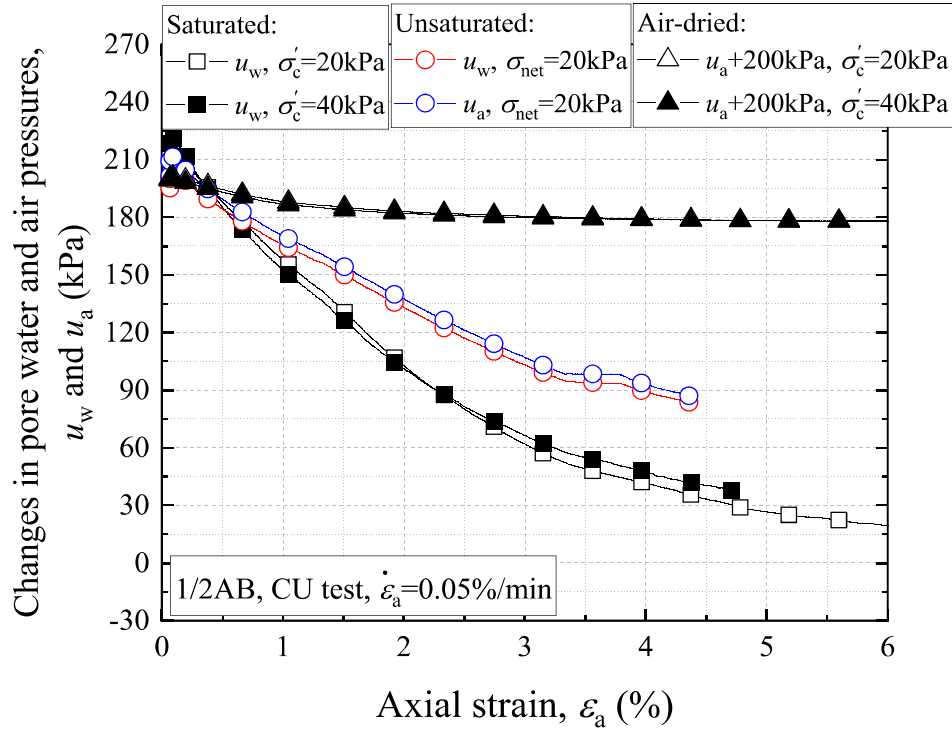


Figure 5-5 Changes in pore water pressure and pore air pressure for 1/2AB in CU tests.

Figure 5-5 presents the changes in pore water pressure (u_w) and pore air pressure (u_a) for 1/2AB in CU tests. The pore water pressure (u_w) in saturated conditions, the pore air pressure (u_a) in air-dried conditions, and the pore water pressure (u_w) and pore air pressure (u_a) under unsaturated conditions increase at the beginning and then gradually decrease, irrespective of water content and confining pressure. Moreover, the downward trends of pore water pressure (u_w) or pore air pressure (u_a) are more pronounced in the case of the specimen with the higher water content. These trends are similar to test results for 1/2CB in CU tests (Figure 5-2(a)), which implies that the changes in effective confining pressure (σ'_c) of both samples are opposite to dilatancy trends of specimens with different water contents in CD tests. These indicate that the reliability of test results in CU tests was confirmed as compared with those in CD tests.

5.2 Shear Strength Parameters

5.2.1 Shear strength parameters of fresh ballast

To obtain the effective shear strength parameters (c' , ϕ') of fresh ballast under different water contents, [Figure 5-6](#) presents the effective shear strength parameters (c' , ϕ') of 1/2CB with different water contents in CD and CU tests calculated by the Mohr circles at failure. When assuming the effective cohesion (c') of the air-dried or saturated 1/2CB to be zero, the effective internal friction angle (ϕ') calculated by the effective confining pressure (σ'_c) of 20 kPa is close to that calculated by the effective confining pressure (σ'_c) of 40 kPa, which proves that the air-dried or saturated 1/2CB is considered to be a non-cohesive sample, and its shear strength under different effective confining pressure (σ'_c) conditions can be expressed through a linear Mohr-Coulomb failure envelope. Therefore, in this study, the shear strength of the fresh ballast under air-dried and saturated conditions is calculated through a linear shear strength equation with an effective cohesion (c') equal to zero. For the unsaturated specimen, [Fredlund et al. \(1978\)](#) proposed the shear strength expressions for unsaturated soils as shown in Eqs. (5-1) and (5-2). Eq. (5-2) defines that the total cohesion (c) of unsaturated soils is composed of two parts: one is the effective cohesion (c') for the saturated soil, and the other is related to the matric suction.

$$\tau_f = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \tan \phi^b \quad (5-1)$$

$$c = c' + (u_a - u_w) \tan \phi^b \quad (5-2)$$

where, τ_f stands for shear strength for unsaturated soils, ϕ' represents the effective internal friction angle for unsaturated and saturated soils, and ϕ^b is the internal friction angle with respect to the matric suction. Based on above theory, the effective internal friction angle (ϕ') values of test samples under unsaturated condition and under saturated condition are assumed to be the same, and the total cohesion (c) under the unsaturated condition can be calculated by Eqs. (5-1) and (5-2). Besides, previous study on coarse granular samples also proved that the effective internal friction angle (ϕ') values for saturated and unsaturated soils are almost constant, and the increase in shear strength for unsaturated soils can be calculated by an apparent cohesion ($(u_a - u_w) \tan \phi^b$, see in Eq.(5-2)) caused by matric suction ([Zhang et al., 2014](#)). Thus, the effective internal friction angle (ϕ') values of saturated and air-dried 1/2CB in CU tests are about 53.5° and 56.3°, respectively, which are slightly

smaller than those in CD tests (55.3° and 58.9°). For the unsaturated specimen, the total cohesion (c) values of unsaturated specimens obtained through CD and CU tests are 1.56 kPa and 3.7 kPa, respectively. The slight differences of test results between CD and CU tests demonstrate that the effective shear strength parameters of fresh ballast with different water contents can be obtained through both CD and CU tests.

Furthermore, in the critical state theory of soil, the maximum effective stress ratio occurs in the critical state of soil (Wood, 1990).

$$M = \eta_{\max} = q_{\max}/p' \quad (5-3)$$

where, M is the critical state effective stress ratio; $\eta_{\max}$ is the maximum effective stress ratio; p' and $q_{\max}$ are the mean effective stress and deviator stress at critical state, respectively. Based on this concept, the effective internal friction angle (ϕ') and effective cohesion (c') can be also calculated as follows (Wood, 1990).

$$\frac{q}{p' + c' \cot \phi'} = \frac{6 \sin \phi'}{3 - \sin \phi'} \quad (5-4)$$

If the $c'=0$, the ϕ' can be calculated as follows,

$$M = \frac{6 \sin \phi'}{3 - \sin \phi'} \quad (5-5)$$

$$\sin \phi' = \frac{3M}{6+M} \quad (5-6)$$

Table 5-1 Effective shear strength parameters of 1/2CB calculated by M values.

Samples	Test conditions	Water content	$\sigma'_c, \sigma_{\text{net}}$ (kPa)	M	ϕ' (deg.)	c', c (kPa)
1/2CB	CD	Air-dried	20.0	2.40	59.0	0
1/2CB	CD	Unsaturated ($s=5$ kPa)	20.0	2.27	54.7	1.1
1/2CB	CD	Saturated	20.0	2.24	54.7	0
1/2CB	CD	Saturated	40.0	2.14	52.1	0
1/2CB	CU	Air-dried	20.0	2.30	56.4	0
1/2CB	CU	Unsaturated ($s=5$ kPa)	20.0	2.22	53.0	2.8
1/2CB	CU	Saturated	20.0	2.18	53.0	0
1/2CB	CU	Saturated	40.0	2.14	52.1	0

c' stands for effective cohesions in saturated and air-dried conditions; c stands for total cohesion in unsaturated condition.

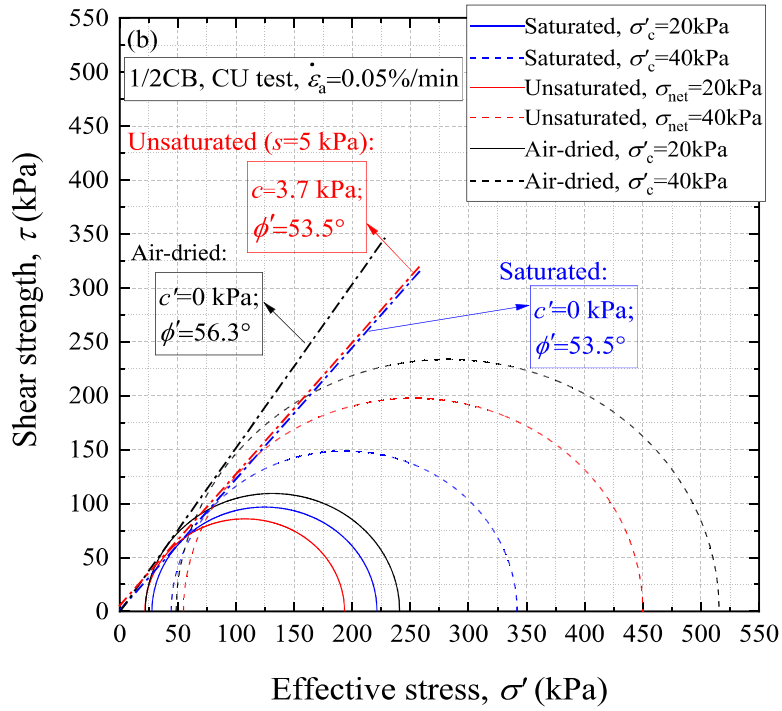
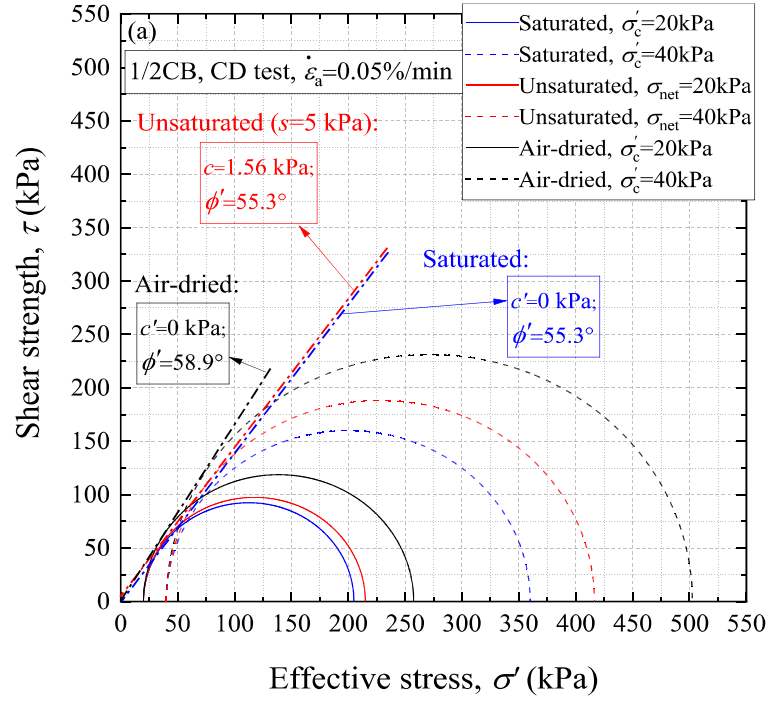


Figure 5-6 Effective shear strength parameters of 1/2CB with different water contents: (a) CD test; (b) CU test.

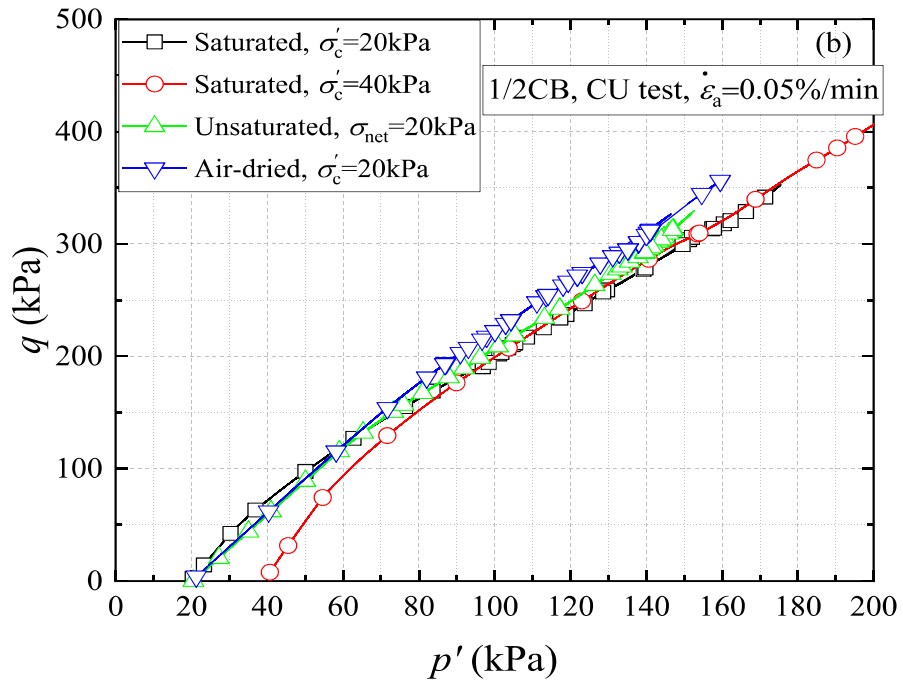
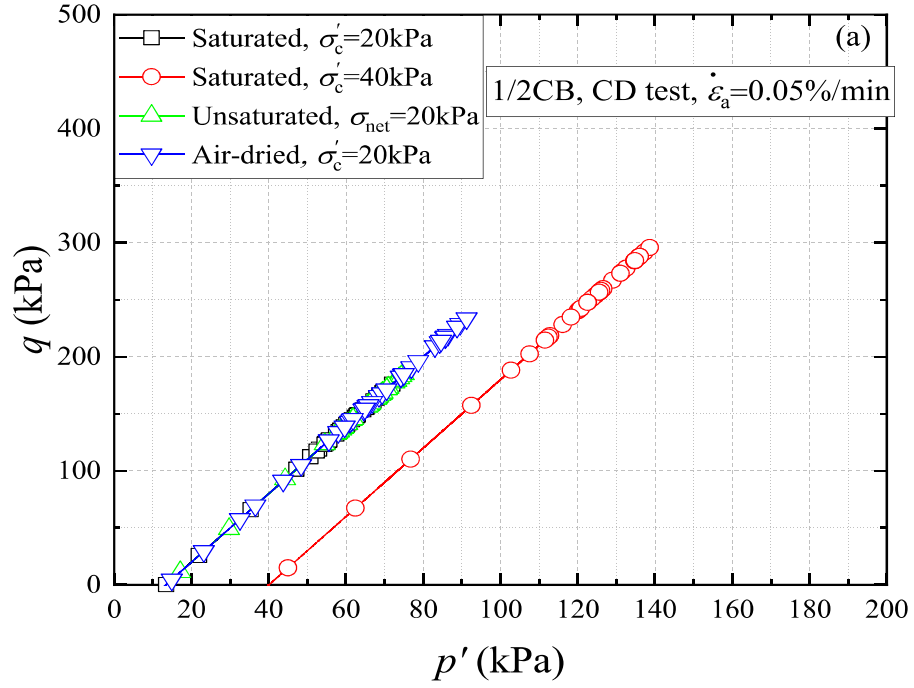
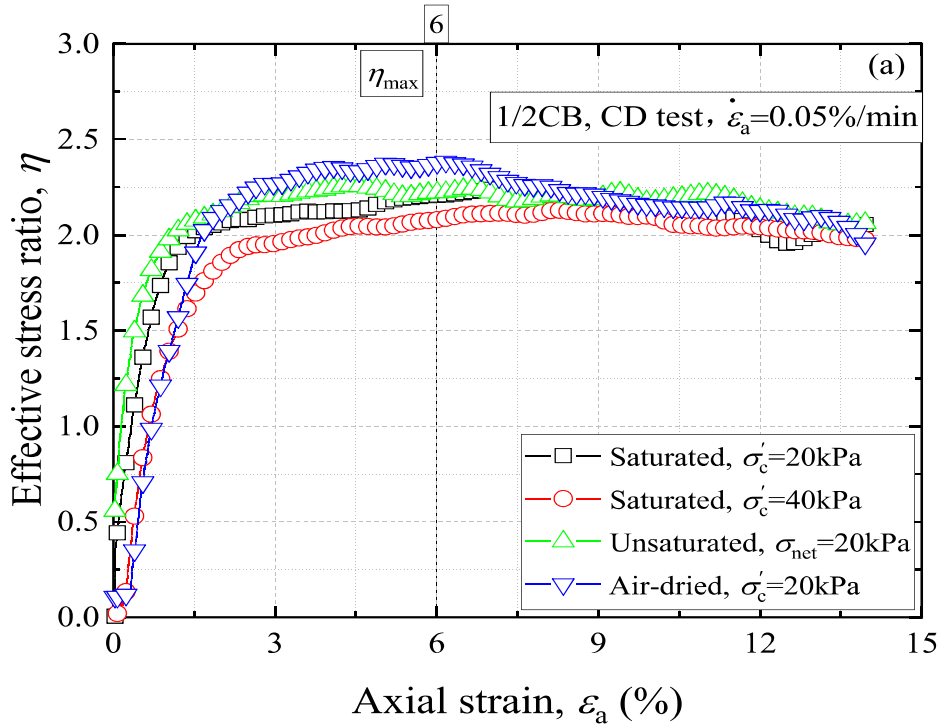


Figure 5-7 p' - q of 1/2CB in the CD and CU tests: (a) CD test; (b) CU test.

Figure 5-7 shows the p' - q plane of 1/2CB in CD and CU tests. As the results, the ratio of q and p' increases from the initial state, and test samples seem to reach critical states because the q - p' relationship seems to be linear with the increase of p' . For a more specific description, Figure 5-8 presents the q/p' - ε_a relations in these tests. As the result, the $\eta_{\max}$ values of 1/2CB occur at the axial strain of about 6% and 2% in CD and CU tests, respectively. When comparing different water contents, the $\eta_{\max}$ values decrease with the water content, irrespective of drainage conditions. In q - p' plane, the effective cohesion (c') values of air-dried and saturated 1/2CB are assumed to be zero, and thus the effective internal friction angle (ϕ') values of 1/2CB can be calculated by Eq. (5-6). The c under the unsaturated condition can be calculated by Eqs. (5-1) and (5-2). In this case, the values of c' or c , and the effective internal friction angle (ϕ') for 1/2CB under different test conditions based on M values are shown in Table 5-1. These results show that effective shear strength parameters of fresh ballast are affected by water content, and not by confining pressure or drainage conditions. Thus, the effective shear strength parameters of fresh ballast with different water contents in this study can be calculated by the average values obtained from CD tests and CU tests with different calculation methods (Figure 5-6 and Table 5-1), as listed in Table 5-2.



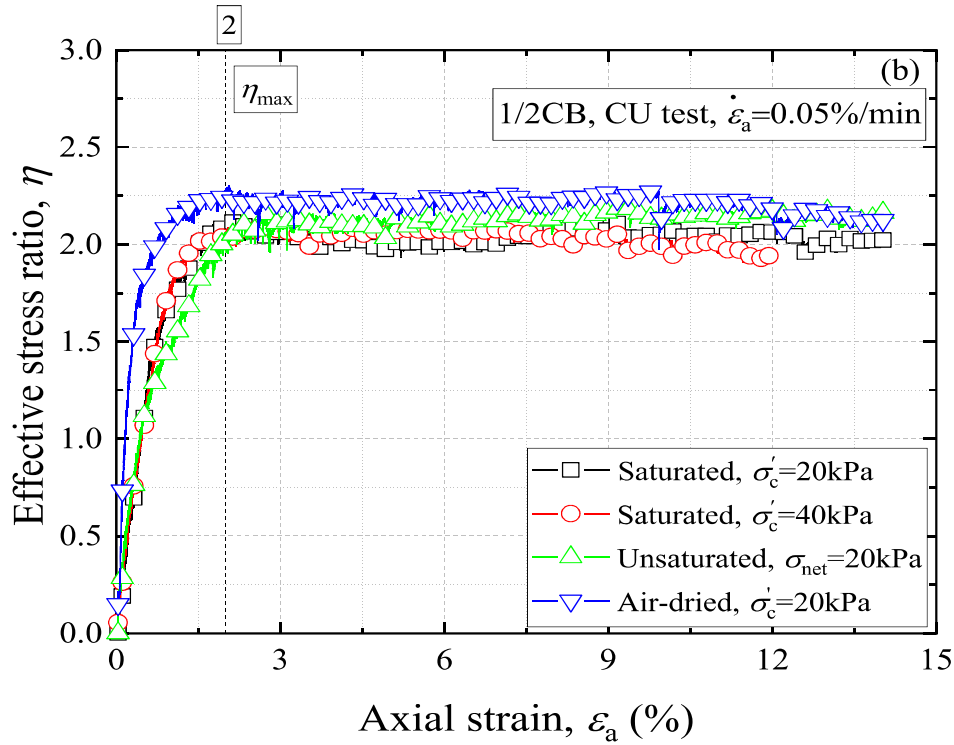


Figure 5-8 $q/p'-\epsilon_a$ relations of 1/2CB in the CD and CU tests: (a) CD test; (b) CU test.

Table 5-2 Average effective shear strength parameters of 1/2CB.

Samples	Water content	S_r (%)	CD-M-C		CU-M-C		CD- $q-p'$		CU- $q-p'$		Average value	
			ϕ'	c', c	ϕ'	c', c	ϕ'	c', c	ϕ'	c', c	ϕ'	c', c
			(deg.)	(kPa)	(deg.)	(kPa)	(deg.)	(kPa)	(deg.)	(kPa)	(deg.)	(kPa)
1/2CB	Air-dried	4.7	58.9	0	56.3	0	59.0	0	56.4	0	57.7	0
	$s=5$ kPa	8.6	55.3	1.56	53.5	3.7	54.7	1.1	53.0	2.8	54.1	2.3
	Saturated	100	55.3	0	53.5	0	54.7	0	53.0	0	54.1	0

5.2.2 Shear strength parameters of aged ballast

For the effective shear strength parameters (c' , ϕ') of aged ballast, the effective shear strength parameters of 1/2AB in CD and CU tests based on Mohr circles at failure are shown in Figure 5-9(b). In terms of shear strength parameters for ballast samples, previous studies (Huang et al., 2009; Wnek et al., 2013) indicated that the shear strength parameters of the fouled (clay, mineral, and coal) ballast with different moisture contents and normal stresses can be determined by a linear Mohr-Coulomb failure envelope. Therefore, in this study, the aged ballast with different water contents under different effective confining

pressure (σ'_c) conditions is also calculated by a linear Mohr-Coulomb failure envelope. Unlike the results of fresh ballast under different drainage conditions shown in Figure 5-5, there are large differences in the effective shear strength parameters of aged ballast between CD and CU tests under saturated conditions regardless of confining pressure, due to a partially drained phenomenon occurred in CD test for aged ballast. It seems that the effective shear strength parameters of aged ballast obtained from CU tests are more accurate than those obtained through CD tests because the pore water pressure (u_w) for 1/2AB cannot be well controlled at a constant value in the CD test. Thus, it is more reliable to use the results of CU tests to calculate effective shear strength parameters of aged ballast.

Table 5-3 Effective shear strength parameters of 1/2AB calculated by M values.

Samples	Test conditions	Water content	σ'_c, σ_{net} (kPa)	η_{max}	M	ϕ' (deg.)	c', c (kPa)
1/2AB	CD	Saturated	20.0	2.18	1.74	42.3	33.5
1/2AB	CD	Saturated	40.0	1.99		42.3	33.5
1/2AB	CU	Air-dried	20.0	2.48	2.42	59.6	5.0
1/2AB	CU	Air-dried	40.0	2.45		59.6	5.0
1/2AB	CU	$s=5$ kPa	20.0	2.30	2.04	49.5	9.2
1/2AB	CU	Saturated	20.0	2.22	2.04	49.5	4.4
1/2AB	CU	Saturated	40.0	2.10		49.5	4.4

Table 5-4 Average effective shear strength parameters of 1/2AB.

Samples	Water content	S_r (%)	CU-M-C		CU- $q-p'$		Average value	
			ϕ' (deg.)	c', c (kPa)	ϕ' (deg.)	c', c (kPa)	ϕ' (deg.)	c', c (kPa)
1/2AB	Air-dried	21.6	59.1	5.1	59.6	5.0	59.4	5.1
	$s=5$ kPa	67.4	49.4	9.5	49.5	9.2	49.5	9.4
	Saturated	100	49.4	5.2	49.5	4.4	49.5	4.8

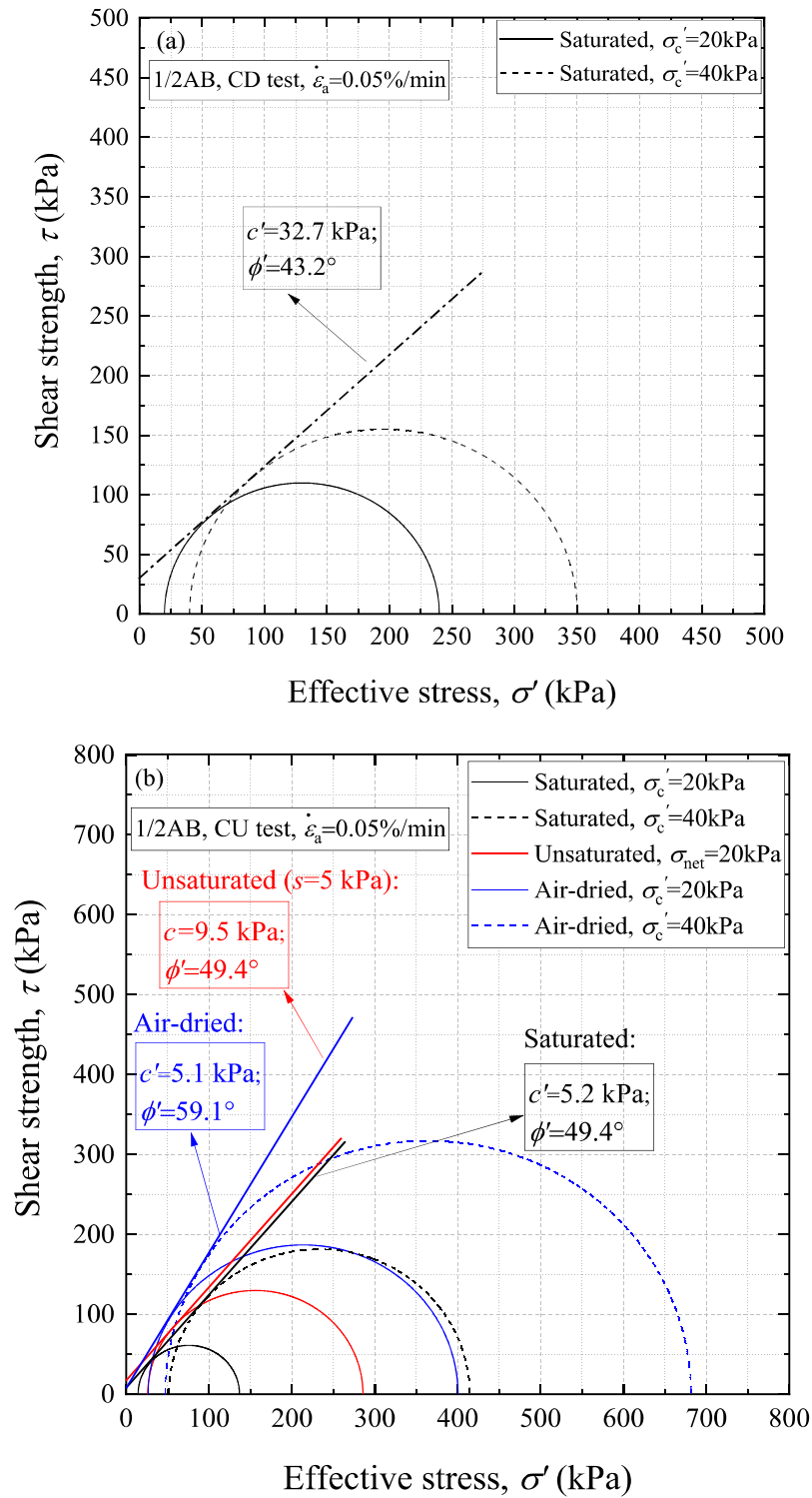


Figure 5-9 Effective shear strength parameters of 1/2AB calculated by Mohr circles at failure: (a) CD test; (b) CU test.

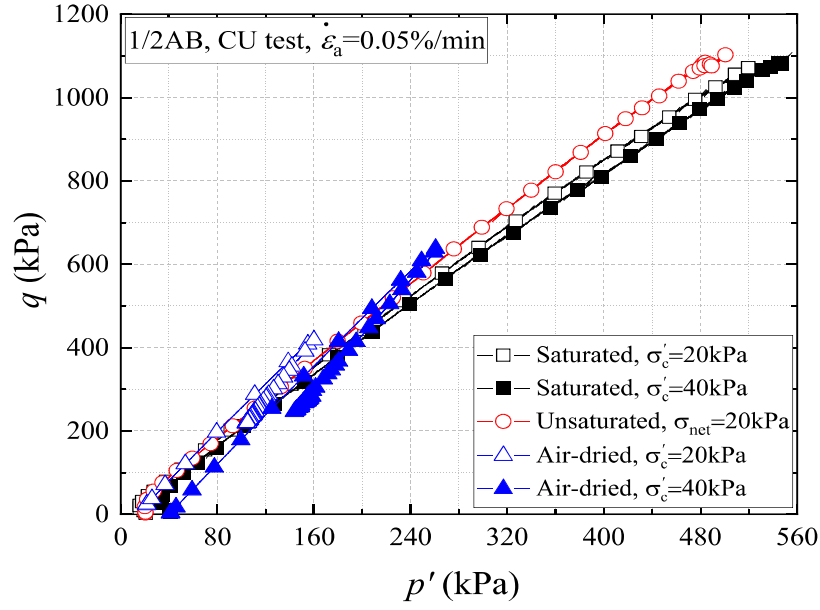


Figure 5-10 p' - q curves of 1/2AB in CU tests.

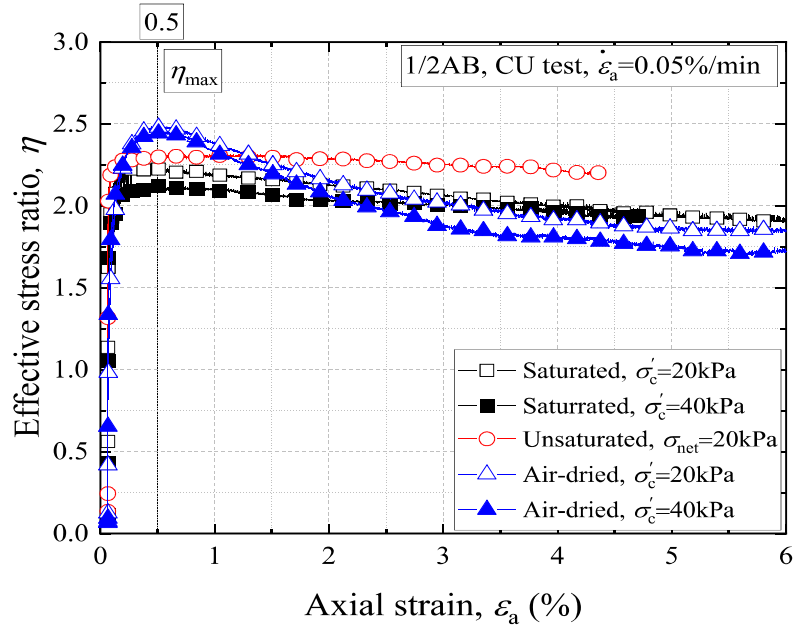


Figure 5-11 η - ε_a relations of 1/2AB in CU tests.

Similarly, Figure 5-10 shows p' - q relations of 1/2AB in the CU test regarding water content and effective confining pressure (σ'_c). When the p' value is constant, the ratio of q to p' decreases with increasing water content. Furthermore, Figure 5-11 shows the change in effective stress ratio ($\eta=q/p'$) with the axial strain (ε_a) under different water contents and

effective confining pressure (σ'_c) conditions. The results prove that the $\eta_{\max}$ values in CU tests occur at axial strain (ϵ_a) value of 0.5% or less. More importantly, the effective shear strength parameters of 1/2AB under different test conditions are calculated according to Eq. (5-4), as listed in Table 5-3. The above results demonstrate that the effective shear strength parameters of aged ballast are affected by the water content and drainage condition, and the CU test is more suitable for calculating these parameters. Thus, the effective shear strength parameters of aged ballast with different water contents in this study can be calculated by the average values of Figure 5-9(b) and Table 5-3, as listed in Table 5-4. Besides, when comparing M values of 1/2CB (Table 5-1) and 1/2AB (Table 5-3), M values of 1/2CB under saturated and unsaturated conditions are higher than those of 1/2AB under the same conditions, while the former is smaller than the latter under air-dried condition. The trends of M with different water contents are consistent with those of effective internal friction angle (ϕ') values under the same conditions, irrespective of the test sample.

5.3 Influence of Aging on Shear strength

5.3.1 Effect of drainage condition

For the saturated specimen in the CD test, the pore water was drained from the bottom end of the sample. Considering that a large difference in the shear strength for the saturated 1/2AB between the CU test and CD test, the pore water pressure (u_w) at the top of the specimen was also measured in the CD test. As shown in Figure 5-12, for the 1/2CB, the pore water pressure (u_w) at the top of specimen maintains a constant value of 200 kPa during the CD shearing process under the axial strain ratio ($\dot{\epsilon}_a$) value of 0.05%/min due to the high permeability of fresh ballast. On the contrary, the positive and negative pore water pressures appear in the sample of 1/2AB under the same axial strain ratio ($\dot{\epsilon}_a$) condition, irrespective of confining pressures. The phenomenon indicates that the aged ballast used in this study is under a partially drained condition rather than a fully drained condition during the shearing process. Considering the effect of the axial strain ratio ($\dot{\epsilon}_a$) on the soil drainage, a lower axial strain ratio ($\dot{\epsilon}_a$) value of 0.01%/min for 1/2AB was also checked in the CD test by referring to JGS 0524-2009. The excess pore water pressure fluctuates greatly even at the axial strain ratio ($\dot{\epsilon}_a$) value of 0.01%/min, and the value of excess pore pressure at this axial strain ratio ($\dot{\epsilon}_a$) is almost the same as that at the axial strain ratio ($\dot{\epsilon}_a$)

value of 0.05%. Moreover, the values of excess pore water pressure are greatly affected by the effective confining pressure (σ'_c), and a greater excess pore water pressure is seen in the CD test under the higher effective confining pressure (σ'_c). The above results show that for low-permeability coarse-grained samples (i.e., aged ballast), it is difficult for the traditional CD test to simulate the fully drained condition and correctly evaluate its effective shear strength.

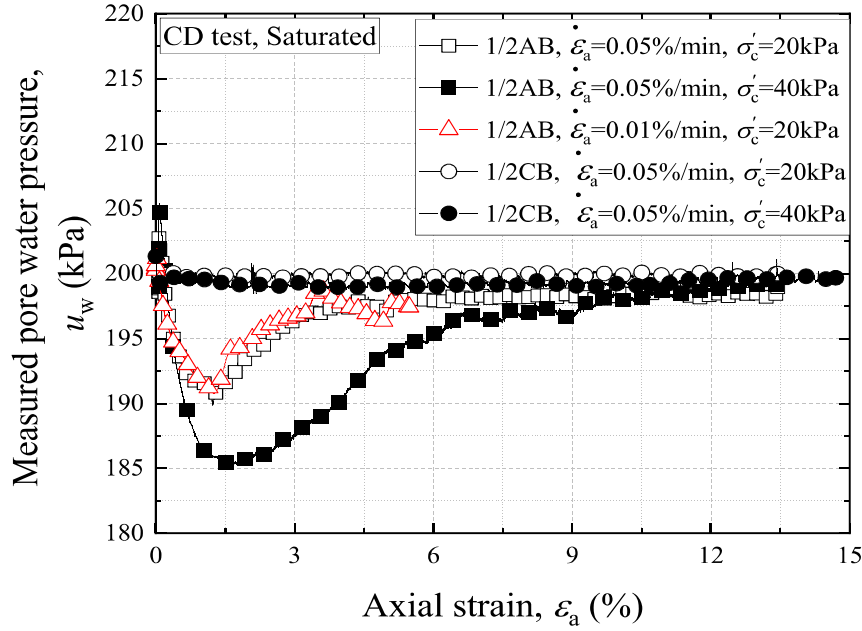


Figure 5-12 Measured pore water pressure of test specimens in CD tests.

Figure 5-13 presents a comparison of principal stress ratio (σ'_1/σ'_3) for 1/2AB under different axial strain ratio ($\dot{\epsilon}_a$) and drainage conditions. In the CD test, with the increase of axial strain ratio ($\dot{\epsilon}_a$), the principal stress ratio (σ'_1/σ'_3) does not increase correspondingly, which is different from previous findings of Yamamuro et al. (1993) and Zhang et al. (2014) that the shear strength under the drained condition increases slightly with a larger axial strain ratio ($\dot{\epsilon}_a$). This may be due to the presence of fine particles reduces the permeability of the aged ballast as mentioned before, resulting in not keeping the drained condition under a high axial strain rate. On the other hand, in the CU test, the $(\sigma'_1/\sigma'_3)_{\max}$ at the axial strain ratio ($\dot{\epsilon}_a$) value of 0.05%/min is higher than that at the axial strain ratio ($\dot{\epsilon}_a$) value of 0.01%/min. These results are consistent with the observations of Mun et al. (2016), who also pointed out that the undrained shear strengths in both saturated and unsaturated conditions increase with increasing the axial strain ratio ($\dot{\epsilon}_a$) from a series of CU tests.

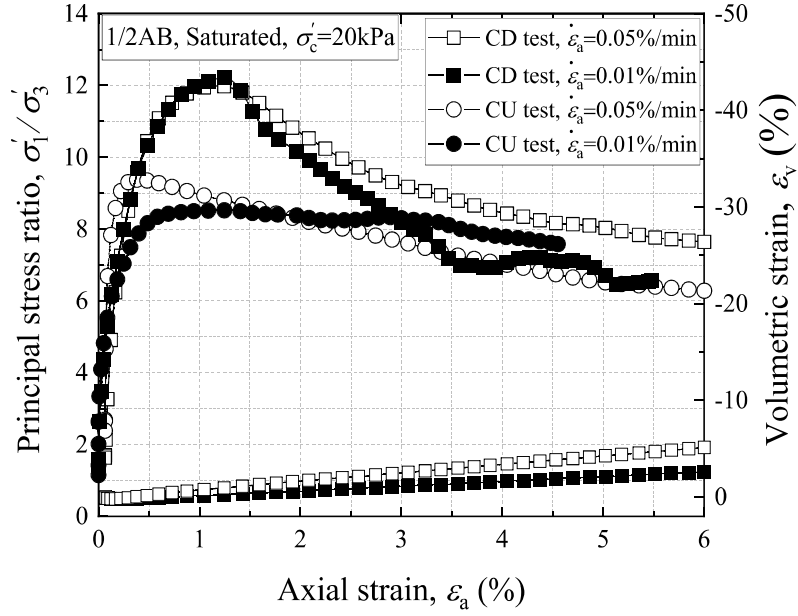


Figure 5-13 Effects of the strain rate and drainage condition on stress-strain relations.

5.3.2 Effect of water content

Figure 5-14 depicts the relationship between the average shear strength parameters and the effective degree of saturation (S_e) for both samples (Table 5-2 and Table 5-4). As shown in Figure 5-14(a), when other test conditions are constant, the effective internal friction angle (ϕ') value of 1/2CB is greater than that of 1/2AB under the high effective degree of saturation (S_e) condition, although the former is smaller than the latter under the low effective degree of saturation (S_e) condition. Moreover, with the increase in effective degree of saturation (S_e), the effective internal friction angle (ϕ') of both samples decreases significantly from air-dried to unsaturated conditions, while they remain unchanged from unsaturated to saturated conditions based on the assumptions of unsaturated soil mechanics proposed by Fredlund et al. (1978). It is noted that for air-dried and saturated soils, the total cohesion (c) is equal to the effective cohesion (c') because the apparent cohesion is zero. Thus, Figure 5-14(b) shows the relationship between the total cohesion (c) of soils and the effective degree of saturation (S_e). When fixed the same effective degree of saturation (S_e) value, the total cohesion (c) of 1/2AB is far greater than that of 1/2CB. These indicate that the presence of fine fraction content reduces the effective internal friction angle (ϕ'), while it increases the total cohesion (c) of the ballast, especially in unsaturated and saturated conditions. Besides, with the increase of effective degree of saturation (S_e), the total

cohesions (c) for both test samples increase firstly and then decrease. It indicates that the total cohesion (c) for the unsaturated soil increases with the increasing matric suction within a certain range value. When the water content is less than the residual value, the total cohesion (c) seems to be unaffected by the matric suction. This can be explained by previous studies (Cunningham et al., 2003; Vanapalli et al., 1996) that once the residual suction condition is exceeded, the shear strength of most soils does not seem to increase significantly.

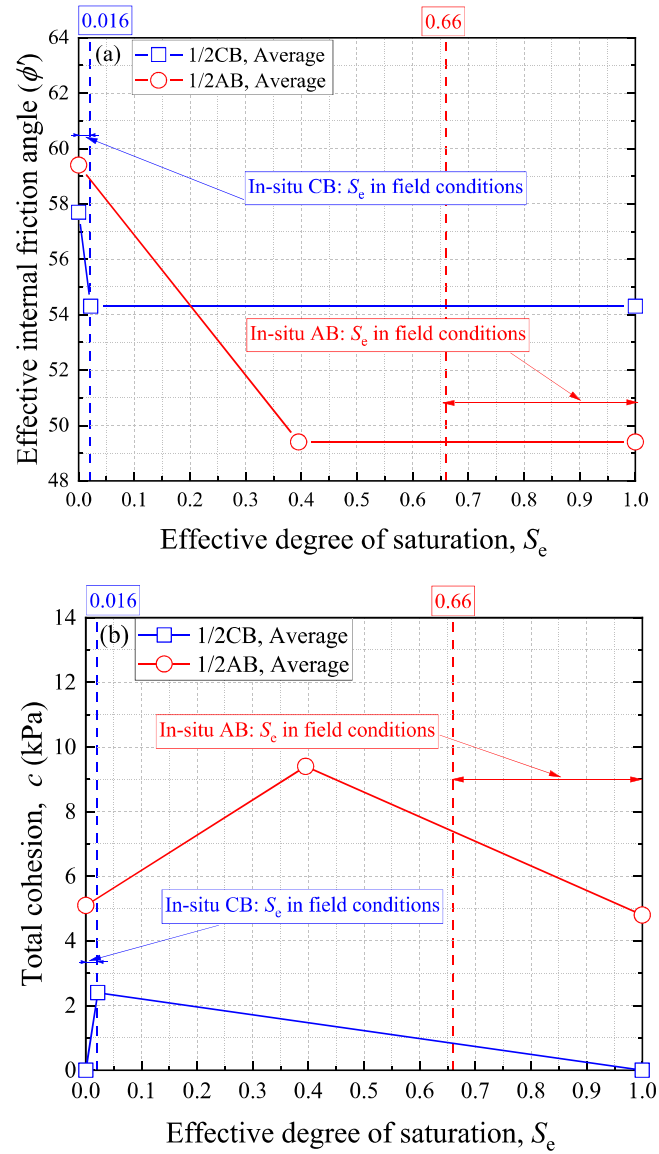


Figure 5-14 Shear strength parameters-water content relations of all test samples: (a) ϕ' ; (b) c .

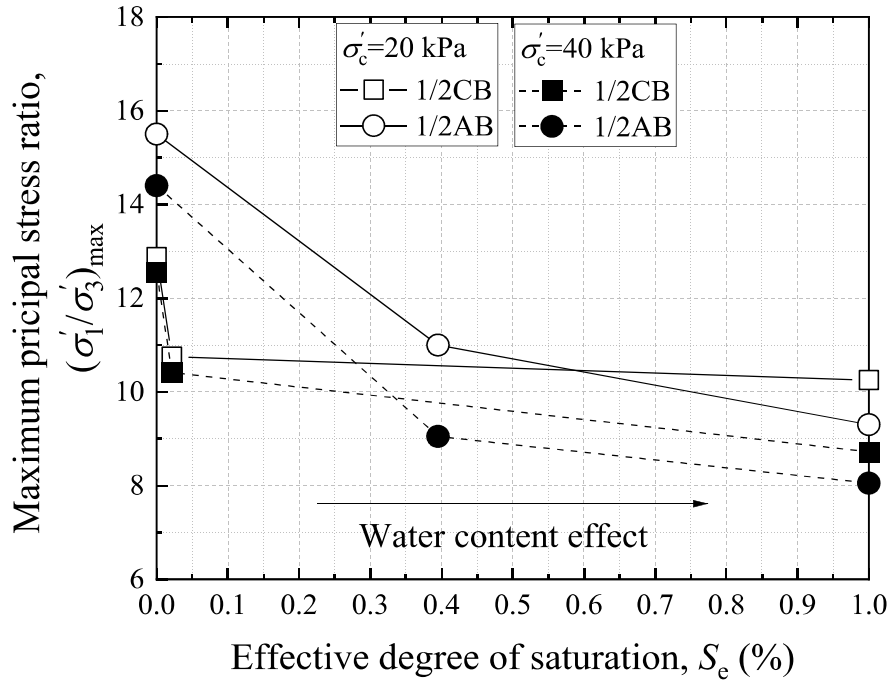


Figure 5-15 Relationship between $(\sigma'_1/\sigma'_3)_{\max}$ and S_e

To further elaborate the influence of water content on the shear strength of ballast, Figure 5-16 presents relationship between the $(\sigma'_1/\sigma'_3)_{\max}$ and effective degree of saturation (S_e). When comparing the effect of water content, the $(\sigma'_1/\sigma'_3)_{\max}$ decreases at the higher effective degree of saturation (S_e), and the declining trend of $(\sigma'_1/\sigma'_3)_{\max}$ is more noticeable in 1/2AB as compared with 1/2CB, irrespective of confining pressure.

5.3.3 Effect of aging

To investigate the influence of fine fraction content (F_c) on the shear strength of ballast, Figure 5-16 presents relationship between the $(\sigma'_1/\sigma'_3)_{\max}$ and F_c . As can be seen in this figure, the $(\sigma'_1/\sigma'_3)_{\max}$ value of 1/2CB is higher than that of 1/2AB in unsaturated and saturated conditions, though the opposite trend appears in the air-dried condition, regardless of confining pressures. Although effective internal friction angles (ϕ') of aged ballast under saturated and unsaturated conditions are less than those of fresh ballast, a larger effective cohesion (c') of aged ballast improves its shear strength, especially under the low effective confining pressure and air-dried conditions. The above results point out that the both water content and fine friction content synergistically affect the shear strength

of ballast, and the increase of water content negatively strengthens the adverse effect of fine fraction content on the shear strength.

In addition, it should be emphasized that the effects of particle shape changes and fine fraction content on the shear strength of aged ballast are considered simultaneously in this study, though the effect of particle shape changes was not evaluated in previous studies on fouled ballast (Huang et al., 2009; Kim et al., 2005; and Trinh et al., 2012). Compared with those previous studies, test results in this study indicate that aged ballast has an unexpected higher shear strength than fresh ballast under the air-dried condition. The similar trend was also confirmed in Qian et al. (2015). The reasonable explanation for this phenomenon is that the smaller and rounder particles produced in the process of ballast breakdown and abrasion can provide a stabilizing effect by filling the voids among larger particles, thus greatly enhancing the aggregate matrix under the air-dried condition, though fine particles can provide a weakening effect in the aggregate matrix by preventing contact between large particles, and this weakening effect is more pronounced under the saturated condition. Conclusively, the difference in shear strength between fouled ballast and aged ballast demonstrates that when evaluating the shear strength of aged ballast, it is essential to consider the influence of particle shape, fine fraction content, and water content comprehensively.

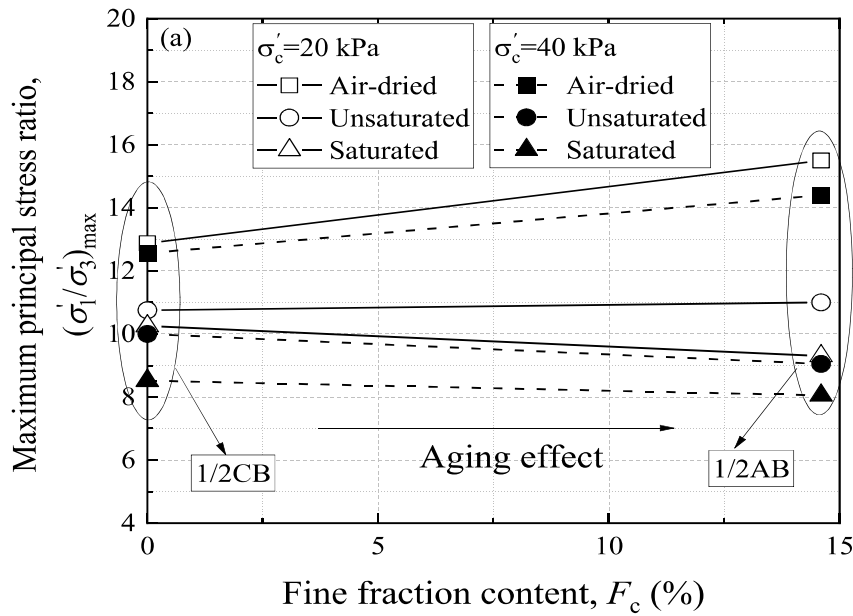


Figure 5-16 Relationship between $(\sigma'_1/\sigma'_3)_{\max}$ and F_c

According to effective shear strength parameters of fresh and aged ballasts obtained above, Figure 5-17(a) illustrates variations in shear strength (τ_f) of two samples with different water contents relate to the in-situ confining pressure (20-40 kPa). Under in-situ stress state and water content conditions, the shear strength of fresh ballast is overall larger than that of aged ballast, especially under the high confining pressure and high saturation conditions, although the aged ballast shows a higher shear strength in the air-dried condition. Field measurements for the Japanese traditional aged ballasted track (Wakayama line, the total annual load is 2.6×10^6 tf) by the Japanese Railway Technical Research Institute (RTRI) show that in natural climatic conditions, the effective degree of saturation (S_e) of the in-situ fresh ballast is generally below 0.016 (In-situ $S_r=8.0\%$) and the effective degree of saturation (S_e) of the in-situ aged ballast with fine fraction content (F_c) value greater than 10% is generally above 0.66 (In-situ $S_r=81.7\%$) because there are large differences in hydraulic properties between fresh ballast and aged ballast (Table 3-1 and Figure 4-2). It is worth noting that, according to previous study on fresh ballast and fouled ballast, the SWCCs of full-scale ballasts are similar to those of small-scale ballasts with a parallel gradation (Ishikawa et al., 2017). Therefore, the effective degree of saturation (S_e) for in-situ fresh ballast and in-situ aged ballast was calculated by the S_{rr} and S_{rs} for 1/1CB and 1/2AB (Table 3-2) based on Eq. (5-7). In this case, Figure 5-17(b) shows the changes in shear strength (τ_f) with the effective degree of saturation (S_e) at effective confining pressures (σ'_c) of 20 kPa and 40 kPa. As the result, the shear strength of fresh ballast is larger than that of aged ballast in actual field conditions because the in-situ fresh ballast is almost in an air-dried state, while the in-situ aged ballast is almost in a saturated state. Moreover, the difference in shear strength between fresh ballast and aged ballast is more noticeable under the higher effective confining pressure (σ'_c) condition. In the in-situ stress state, the effective confining pressure (σ'_c) increases when the train passes. Therefore, the mechanical properties of the aged ballast are generally worse than those of the fresh ballast in actual railway tracks. Considering this situation, when discussing the impact of ballast aging on the mechanical behavior of railway tracks, the influence of water content and effective confining pressure should be synergistically considered.

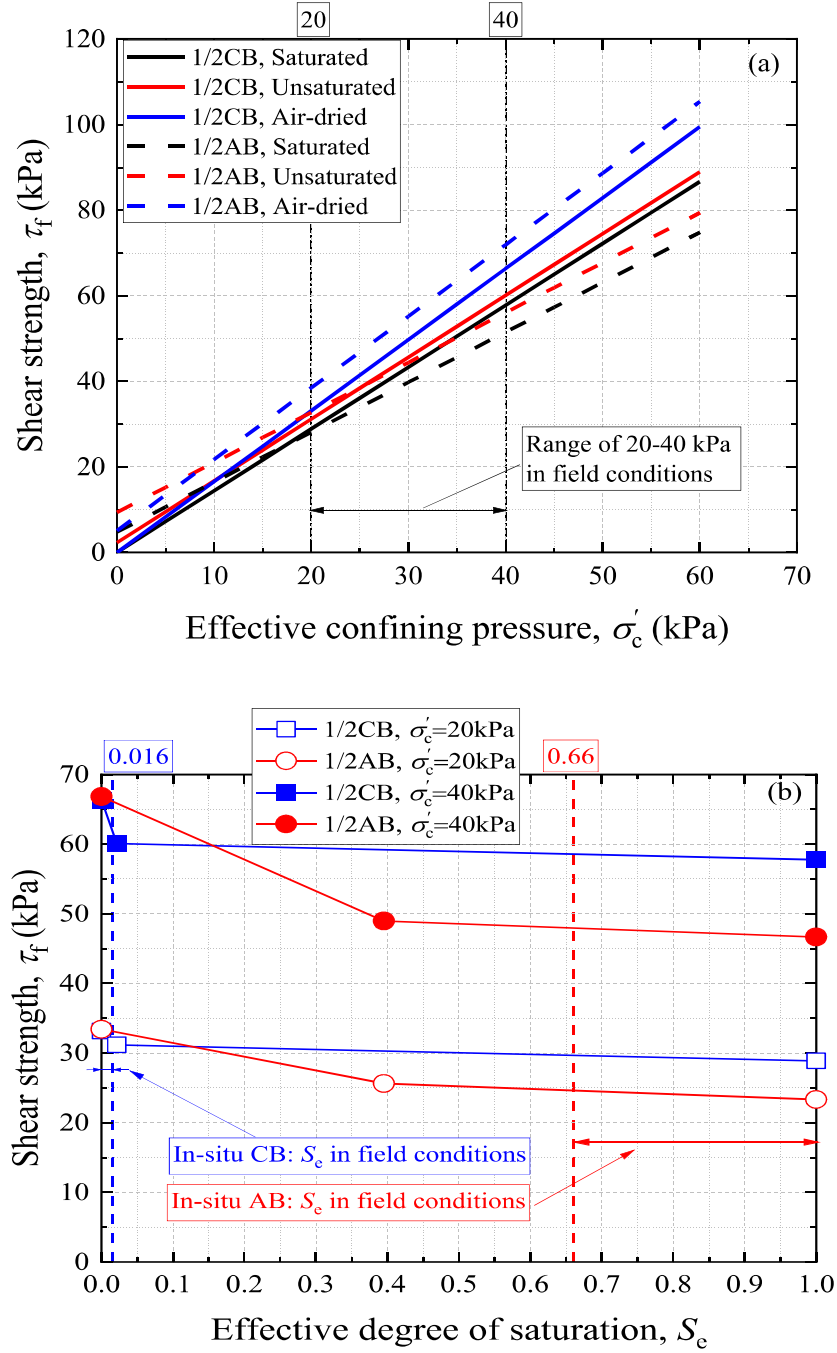


Figure 5-17 Effects of effective confining pressure (σ'_c) and effective degree of saturation (S_e) on the shear strength (τ_f) for test samples with different fine fraction contents: (a) τ_f - σ'_c ; (b) τ_f - S_e .

5.4 Summary

In this chapter, a series of monotonic loading triaxial compression tests (i.e., CD and CU tests) were conducted to evaluate the shear strength behavior of unsaturated fresh and aged ballasts. The following findings can be mainly observed:

- (1) The effective shear strength parameters of fresh ballast with different water contents can be obtained through the CU and CD tests. However, for the aged ballast, the effective shear strength parameters are obtained more accurately through the CU test, due to the low permeability of aged ballast.
- (2) The presence of fine fraction content in aged ballast reduces the effective internal friction angle (ϕ'), while it increases the total cohesion (c) as compared with the fresh ballast, especially under the high saturation condition.
- (3) When considering the effect of water content, the effective internal friction angles (ϕ') of fresh and aged ballast decrease significantly from the air-dried state to the unsaturated state, while they remain unchanged from the unsaturated state to saturated state, and the total cohesions (c) decrease from the residual value to the saturated state. Thus, the shear strength of fresh and aged ballast decreases as the water content increases, and the declining trend of the shear strength is more noticeable in the aged ballast.
- (4) The shear strength of fresh ballast is overall larger than that of aged ballast in the field condition. This is because the degree of saturation of aged ballast in the field condition is generally much higher than that of fresh ballast, due to its poor permeability, resulting in the poor mechanical performance of aged ballast. The higher effective confining pressure caused by the train passing further increases the difference in shear strength between the fresh ballast and aged ballast.

6 INFLUENCE OF AGING ON PERMANENT DEFORMATION OF UNSATURATED BALLAST UNDER CYCLIC LOADING

6.1 Permanent Deformation Behavior

Track deteriorations with a continuous increase in permanent deformation are always observed at ballasted tracks mainly due to ballast aging. However, the influence of aging (particle size and shape) on the cyclic plastic deformation was not evaluated in previous studies on fouled ballast. In this chapter, the influence of aging on the permanent axial strain of unsaturated ballast was comprehensively evaluate based on the results of CL tests. The CL test conditions and results for test samples were listed in Table 6-1.

Table 6-1 CL test conditions and results.

Sample	Drainage condition	Water content	σ'_c, σ_{net} (kPa)	ρ_{dc} (g/cm ³)	D_c (%)	S_r (%)	$\varepsilon_{p, max}$ (%)
1/2CB	CD test	Air-dried	20.0	1.571	95.1	4.22	1.08
1/2CB	CD test	$s=40$ kPa	20.0	1.614	97.7	6.64	1.36
1/2CB	CD test	$s=20$ kPa	20.0	1.568	94.9	6.86	2.50
1/2CB	CD test	$s=5$ kPa	20.0	1.539	93.2	10.62	2.97
1/2CB	CD test	Saturated	20.0	1.541	93.3	100	3.53
1/2AB	CD test	Air-dried	20.0	2.220	101.6	21.7	0.65
1/2AB	CD test	$s=5$ kPa	20.0	2.198	100.6	67.2	6.40
1/2AB	CD test	Saturated	20.0	2.205	100.9	100	12.09
1/2AB	CU test	Air-dried	20.0	2.196	100.5	21.3	0.97
1/2AB	CU test	$s=5$ kPa	20.0	2.195	100.5	68.5	8.24
1/2AB	CU test	Saturated	20.0	2.199	100.6	100	Failed

6.1.1 Stress-strain relationship of aged ballast

For the plastic deformation of the granular material, the “shakedown theory” ([Arnold et al., 2002](#); [Werkmeister et al., 2004](#); [Xiao et al., 2017](#)) was often used to describe the plastic deformation and breakage rules of the granular material under cyclic loading. In General, the shakedown behaviors of the granular material under cyclic loading include three stages ([Xiao et al., 2017](#)): plastic shakedown (steady deformation behavior), plastic

creep (failure at a large number of loading cycles), and incremental collapse (failure at a small number of loading cycles), as shown in Figure 6-1.

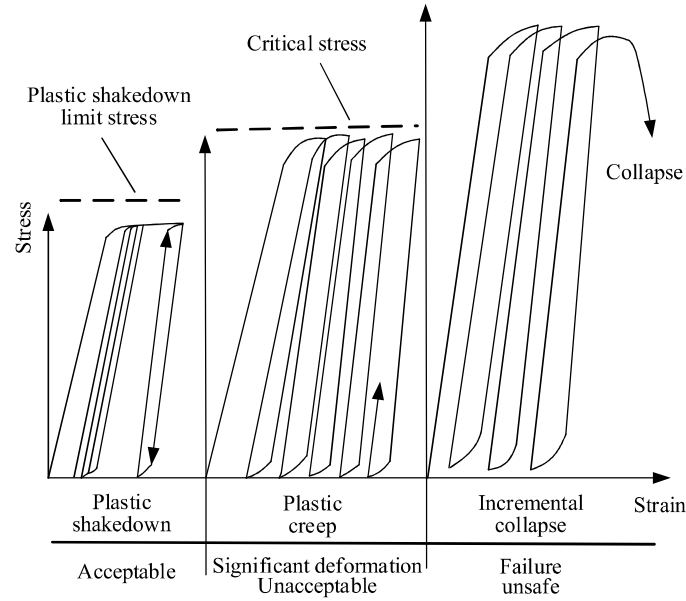


Figure 6-1 Typical shakedown behaviors of granular materials under cyclic loading (after Xiao et al., 2017).

Figure 6-2 and Figure 6-3 present the relationship between axial strain (ε_a) and axial deviator stress (σ_d) of 1/2CB and 1/2AB with various water contents at number of loading cycles (N_c) from 1000 to 1005 in CD and CU tests. This is because previous study (Xiao et al., 2017) pointed out that the fresh ballast is generally in a plastic shakedown or plastic creep state after hundreds of loading cycles. When other test conditions are constant, the elastic and plastic strains of both samples increase with the increment of water content. Besides, both samples are in a plastic shakedown state in the air-dried condition. However, in CD tests, the plastic strain rate of 1/2AB are different from those of 1/2CB in unsaturated and saturated conditions. To be specific, the specimens of 1/2CB in unsaturated and saturated conditions are in a plastic shakedown state, while the specimens of 1/2AB in those conditions are in a plastic creep state. On the other hand, under the saturated condition, the specimen of 1/2AB in the CU test is in an incremental collapse state, and the specimen is failed at a small number of loading cycles. The above results indicate that the water content, drainage condition, and ballast aging have significant influences on the plastic strain rate of ballast. As the water content increases, the aged ballast with fine fraction

content and rounded particle shape can reduce interlock between particles, thereby changing the trend of plastic stain rate of ballast from plastic shakedown to plastic creep state or incremental collapse state. Besides, the reduction of σ'_c in CU tests further increases the development of plastic strain, which will be discussed in a later section.

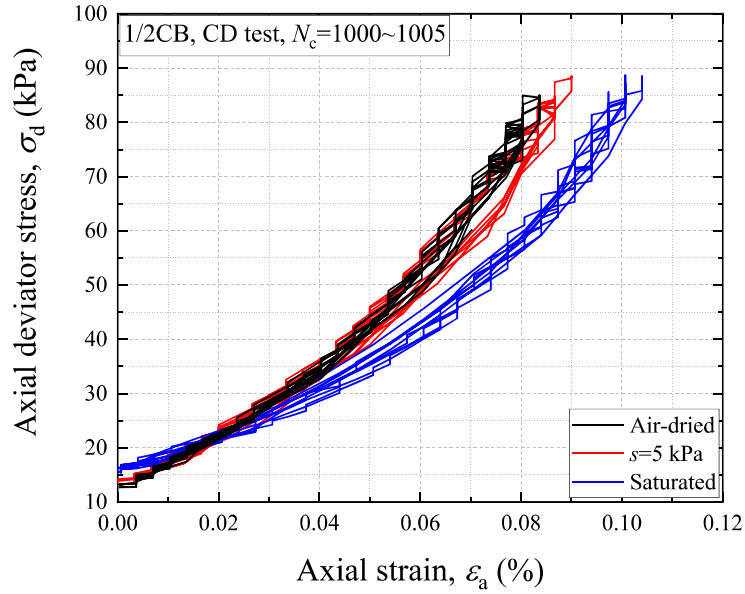
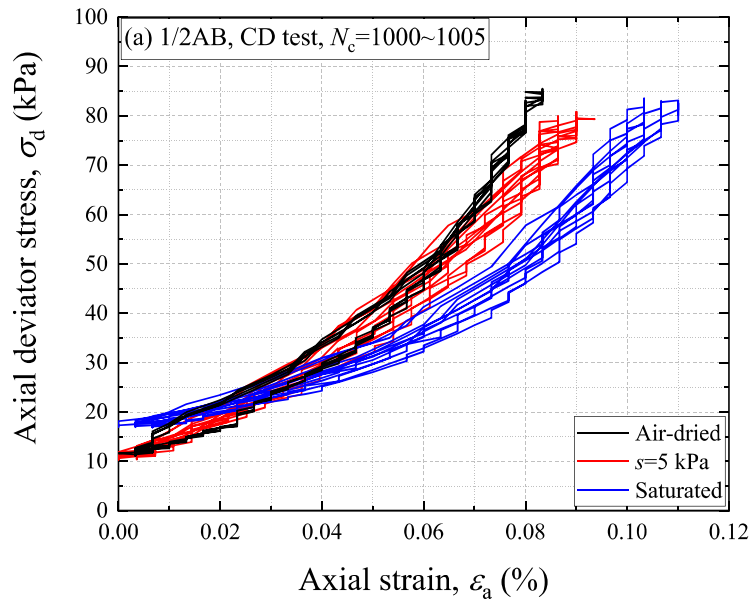


Figure 6-2 Stress-strain relationship for 1/2CB with different water contents in CD tests.



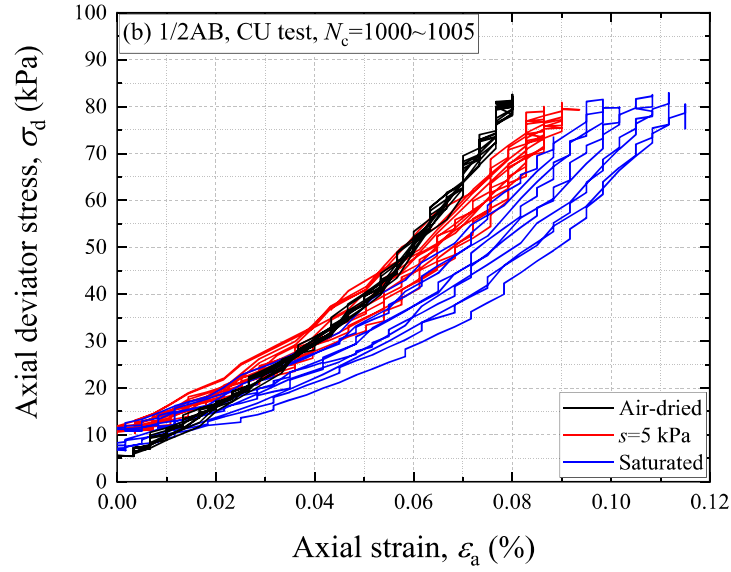


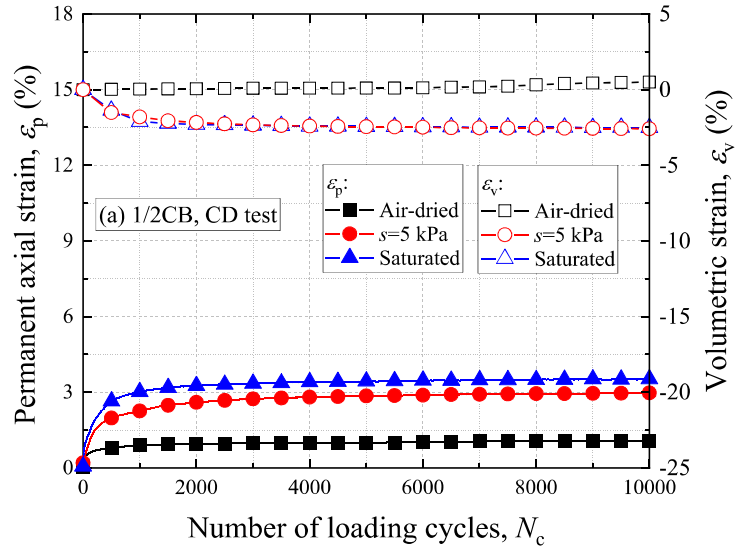
Figure 6-3 Stress-strain relationships for 1/2AB with different water contents in CD and CU tests:

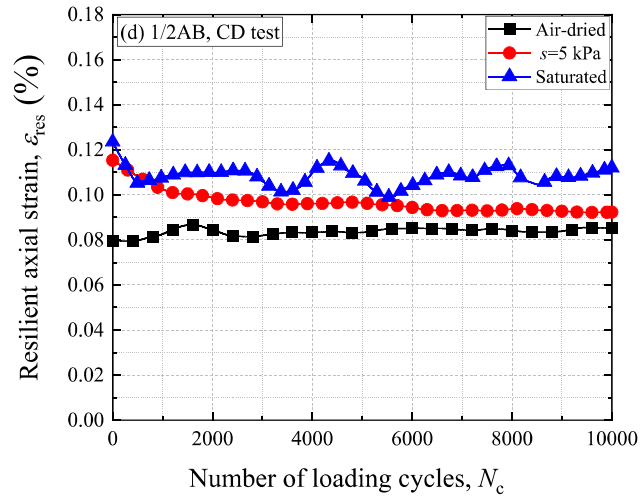
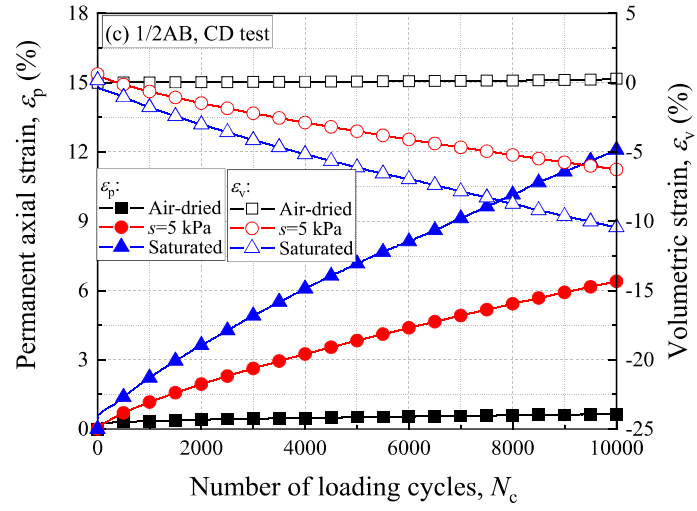
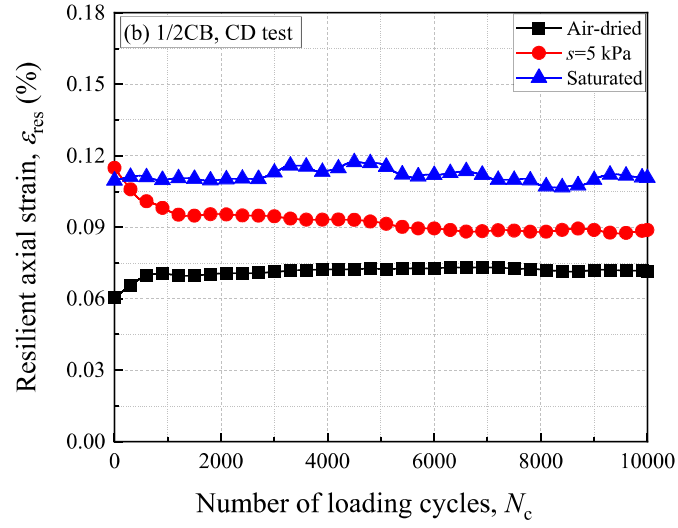
(a) CD test; (b) CU test.

6.1.2 Permanent deformation behavior of aged ballast

Figure 6-4 shows the relationships between the permanent axial strain (ε_p), volumetric strain (ε_v), resilient axial strain (ε_{res}) and number of loading cycles (N_c) up to 10,000 cycles for test specimens of 1/2CB and 1/2AB with various water contents and drainage conditions. Here, ε_p represents permanent axial strain at unloading, and ε_{res} represents the resilient axial strain between the loading and unloading. When comparing the ε_p under the same test conditions except the water content, the ε_p of all specimens increase with the increment of water content, irrespective of test samples. Besides, for the specimen of 1/2CB in CD tests (Figure 6-4 (a)), the ε_p of all specimens show a rapid growth trend in the initial cyclic loading process, and then the increasing trend of ε_p becomes stable as the N_c increases, irrespective of water contents. In terms of 1/2AB in CD tests, the ε_p of 1/2AB in the air-dried condition shows a stable trend with the increment of N_c , while the ε_p in unsaturated and saturated conditions reveal an approximately linear growth trend with the increment of N_c , as shown in Figure 6-4 (c). For the dilatancy behavior in CD tests, the volume of unsaturated and saturated specimens is dilated, while volume of air-dried specimen is compressed as the ε_p increases with the increment of N_c , irrespective of test samples. The reason for this phenomenon is that the volume of the fresh and aged ballasts

is first compressed at axial strain (ε_a) about 1%, followed by dilation with the increase in ε_a during the shear process. When other conditions are unchanged, the specimen under higher degree of saturation shows an upward trend in the dilation due to the larger ε_p . Besides, the ε_{res} of both samples in CD tests increase by the sequences in air-dried, unsaturated, and saturated specimens, though the differences in ε_{res} under different water contents are not remarkable as those in the ε_p . The above results indicate that the permanent/residual axial and volumetric strains of fresh and aged ballast in CD tests are affected by the water content. On the other hand, as shown in Figure 6-4 (e), the ε_p of 1/2AB under the air-dried condition presents a stable trend with the increment of N_c in CU tests, while ε_p under the unsaturated and saturated conditions show an approximately linear increasing trend with the increment of N_c . It is noted that under the saturated condition, 1/2AB is failed at the N_c of 5, 000 in the CU test. Besides, as shown in Figure 6-4 (f), ε_{res} in CU tests increases by in the order of air-dried, unsaturated, and saturated specimens. These results indicate that the cyclic plastic deformation of aged ballast is significantly affected by the water content in both CD and CU tests, and the effects of water content on the cyclic plastic deformation are more significant in CU tests.





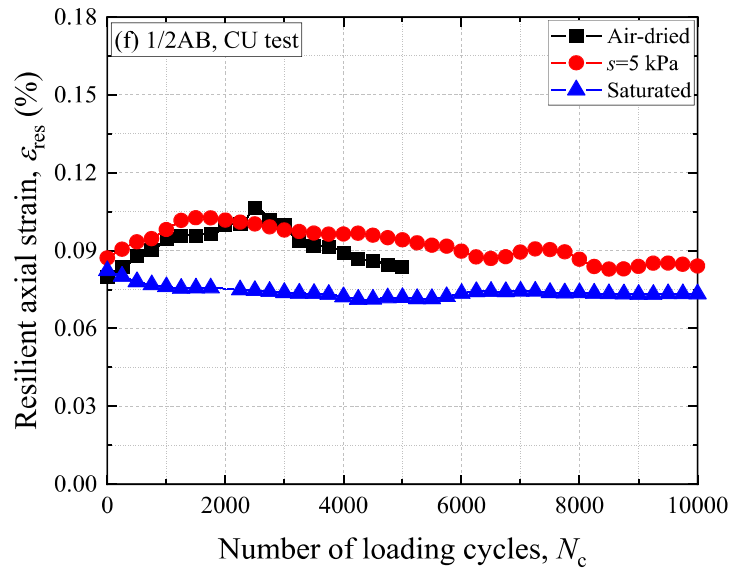
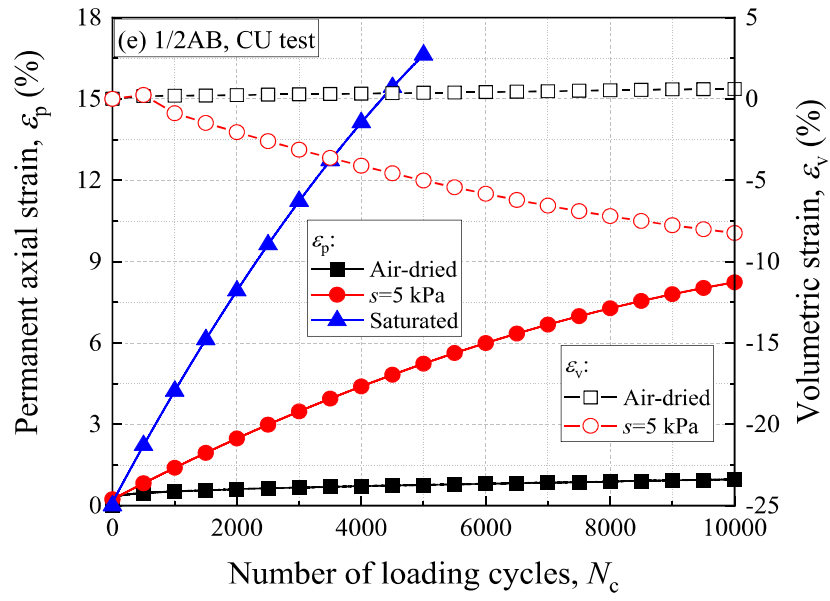
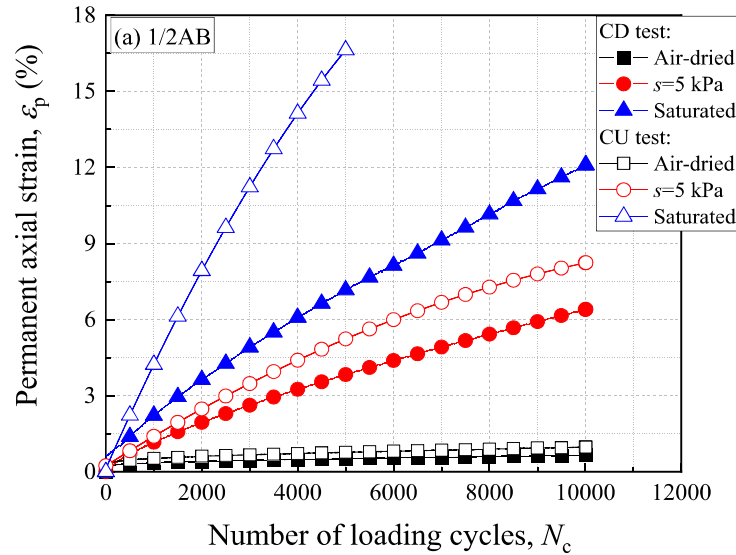


Figure 6-4 ε_p , ε_v , and ε_{res} of test samples with different water contents: (a) ε_p and ε_v of 1/2CB in CD tests; (b) ε_{res} of 1/2CB in CD tests; (c) ε_p and ε_v of 1/2AB in CD tests; (d) ε_{res} of 1/2AB in CD tests; (e) ε_p and ε_v of 1/2AB in CU tests; (f) ε_{res} of 1/2AB in CU tests. Influence of Aging on Permanent Deformation

6.2 Influence of Aging on Permanent Axial Deformation

6.2.1 Effect of drainage condition

Considering that the drainage condition may affect the permanent axial deformation of aged ballast due to its low permeability, thus both CD and CU tests were conducted for the test specimen of 1/2AB. In this case, Figure 6-5 (a) shows the comparisons of ε_p for 1/2AB between CD tests and CU tests. The ε_p of 1/2AB in CU tests are larger than those in CD tests, irrespective of water contents. Besides, in the air-dried condition, ε_p of all specimen are small, irrespective of drainage conditions. However, the differences in ε_p in unsaturated and saturated conditions are remarkable between the two types of test conditions, especially in the saturated condition. These findings indicate that in the high degree of saturation condition, the drainage condition has a significant influence on the permanent axial deformation of aged ballast. For a clearer explanation, Figure 6-5 (b) presents the relationship between the $\varepsilon_{p, \max}$ and S_e of 1/2AB. Here, $\varepsilon_{p, \max}$ means the maximum permanent axial strain at N_c of 10, 000. As can be seen in this figure, the $\varepsilon_{p, \max}$ in the CU test are larger than those in the CD test, regardless of S_e . More importantly, the difference in $\varepsilon_{p, \max}$ is greater as the water content increases. This indicates that the effect of drainage condition on the permanent axial strain becomes more significant at the higher water content.



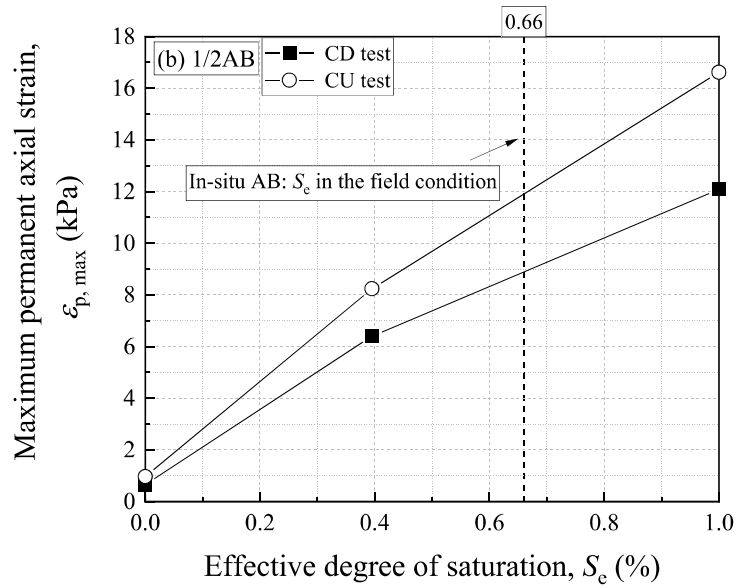


Figure 6-5 Comparison of $\varepsilon_p - N_c$ and $\varepsilon_{p, \max} - S_e$ of 1/2AB between CD test and CU test: (a) $\varepsilon_p - N_c$; (b) $\varepsilon_{p, \max} - S_e$.

Figure 6-6 shows the changes in u_w and ε_p for 1/2AB in CU tests. It is worth mentioning that the increasing trend of excess pore water pressure in the saturated condition is larger than that in the unsaturated condition, which is a probable reason why the saturated specimen in the saturated condition is failed at the N_c of 5, 000. It can be inferred that the excess pore water pressure generated in the CU test is one of the factors that affect the permanent axial deformation of the aged ballast, addition the water content itself can reduce the shear strength of aged ballast, and induce a greater permanent axial deformation. Besides, the excess pore water pressure shows a whole increasing trend with the N_c , though it increases in the initial several loading cycles before showing a downward trend until reaching a minimum value, after which it gradually increases until the end of cyclic loading process, irrespective of water content. Similar to the increasing trend of excess pore water pressure, the relationship between the ε_p and N_c also shows a stably increasing trend, due to the decrease of effective confining pressure (σ'_c) in the CU test as shown in Figure 6-6.

Figure 6-7, the σ'_c of 1/2AB shows a whole decreasing trend with the increment of N_c , though an increasing trend is observed in the initial stage. To be specific, when the N_c is greater than 500, the σ'_c decreases as the increment of the N_c and the final values of σ'_c in CU tests are smaller than those in CD tests, regardless of water contents, which is the reason

why the ε_p in CU tests under the same initial test condition are higher than those in CD tests. Besides, when comparing the final values of σ'_c under different water content conditions, the final value of σ'_c of the saturated specimen is lower than those of unsaturated and air-dried specimens. The change in σ'_c in CU tests is related to the dilatancy trend of the test specimen, since the positive dilatancy trends become stronger by the sequences in air-dried, unsaturated, and saturated specimens. The above results demonstrate that the permanent axial strain of aged ballast is affected by the drainage condition. Considering that the high S_r and low permeability of the aged ballast, the in-situ aged ballast is almost under a partially or fully undrained condition with a high degree of saturation. Therefore, the actual permanent axial strain range of the in-situ aged ballast is generally from the permanent axial strain for an unsaturated specimen in the CD test to the permanent axial strain for a saturated specimen in the CU test. In this case, the CD test under the unsaturated condition is more suitable for evaluating the permanent axial strain of slightly aged ballast, and the CD test under the unsaturated or saturated condition is more suitable for evaluating the permanent axial strain of heavily aged ballast.

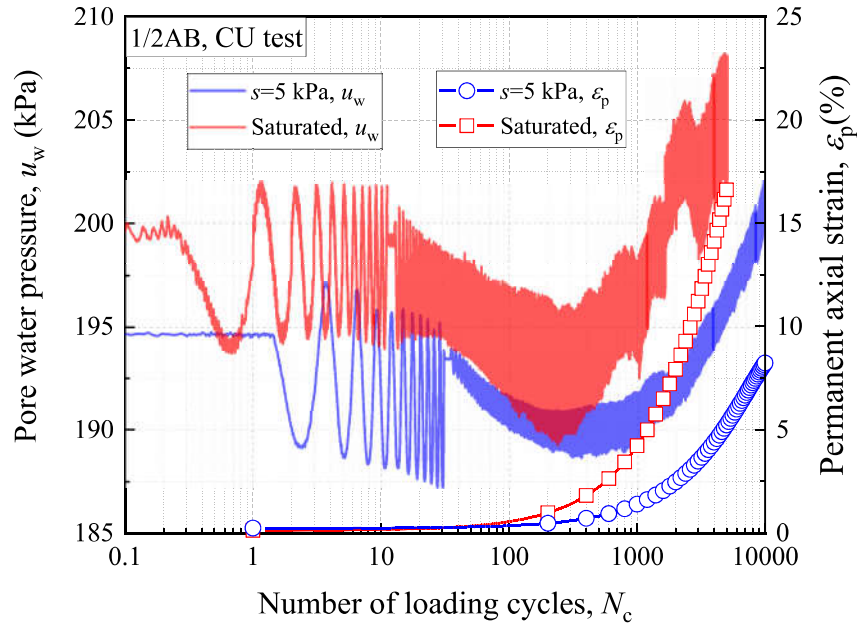


Figure 6-6 Changes in pore water pressure/permanent axial strain of 1/2AB in CU tests.

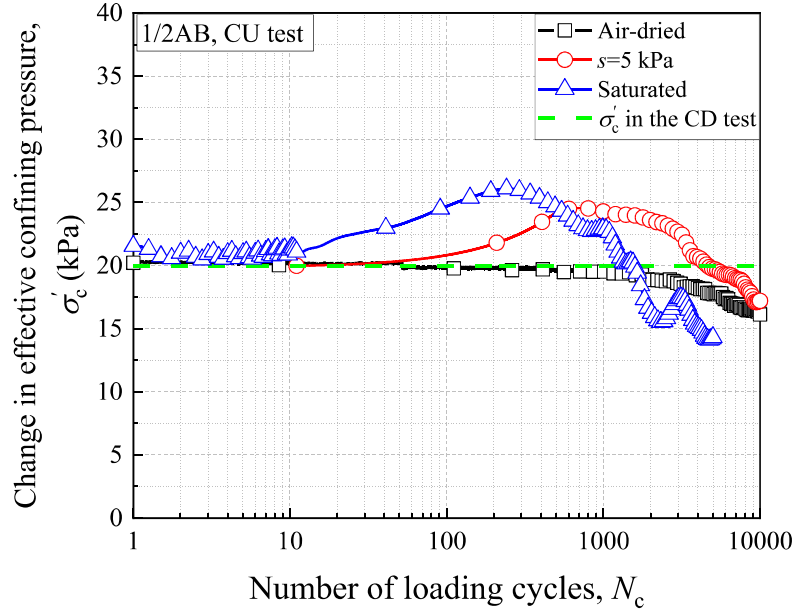


Figure 6-7 Changes in effective confining pressure of 1/2AB in CU tests.

6.2.2 Effect of water content

To evaluate the effect of water content on the ε_p , Figure 6-8(a) shows the effect of water content on ε_p for both samples in CD tests. When the other initial test conditions are constant except for the water content, the ε_p increases with the increase of water content, irrespective of test samples. Furthermore, the wetting causes aged ballast to increase ε_p dramatically as compared with fresh ballast. As shown in Figure 6-8(b), the $\varepsilon_{p, \max}$ appears a remarkably increasing trend under the higher S_c , regardless of test samples and drainage conditions. These results reveal that the water content has serious influence on permanent axial deformation of both samples, especially for the aged ballast.

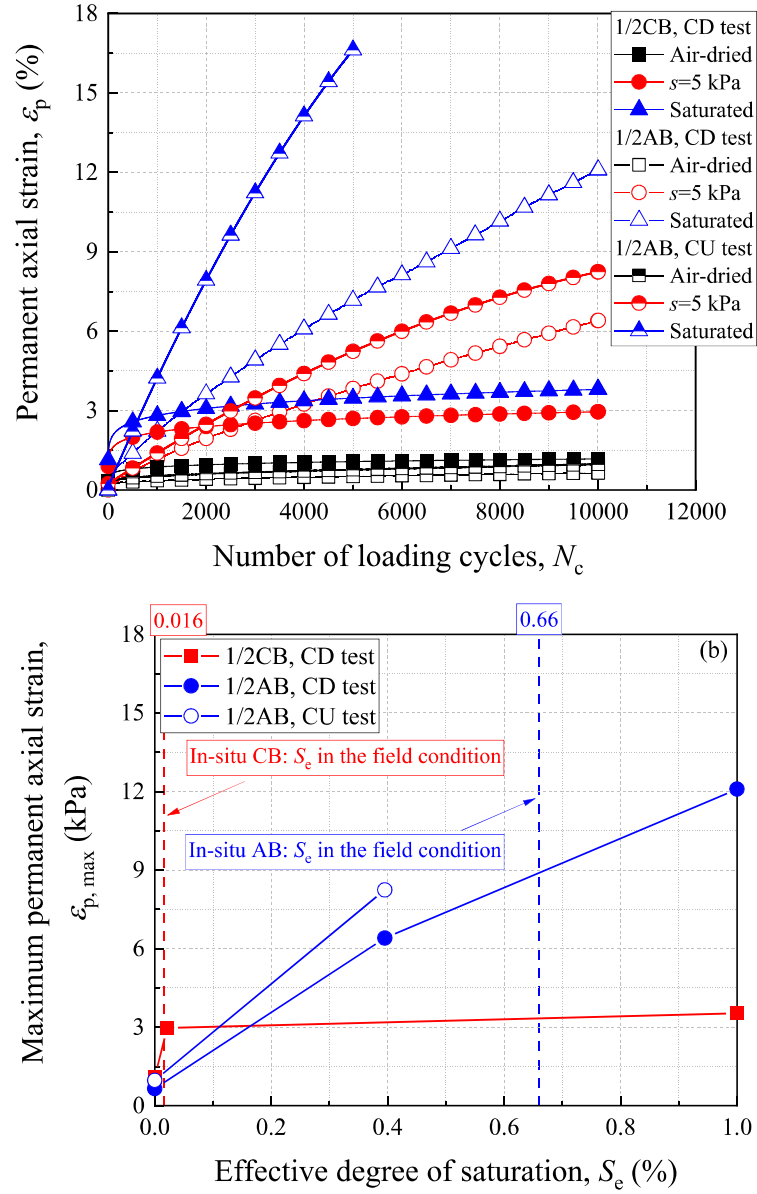
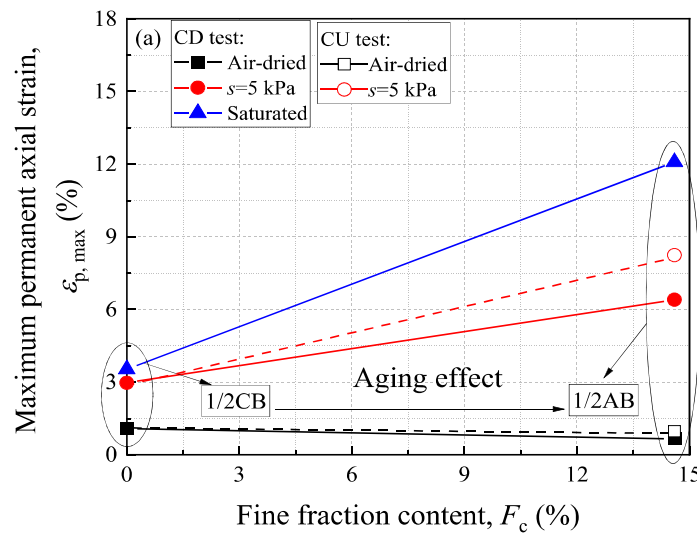


Figure 6-8 Effect of water content on ε_p and $\varepsilon_{p, \max}$: (a) $\varepsilon_p - N_c$; (B) $\varepsilon_{p, \max} - S_e$.

6.2.3 Effect of aging

To investigate the effect of aging on the ε_p of ballast, Figure 6-9(a) presents the relationship between the $\varepsilon_{p, \max}$ and F_c . As can be seen in this figure, $\varepsilon_{p, \max}$ of 1/2AB are higher than those of 1/2CB in unsaturated and saturated conditions, though the opposite trend appears in the air-dried condition. The changing trends of the $\varepsilon_{p, \max}$ with various water contents are similar to those of maximum principal stress ratio $((\sigma'_1/\sigma'_3)_{\max})$ under

the same conditions shown in Figure 6-9(b). These results point out that the both water content and aging synergistically affect the permanent axial strain of the ballast, and the increase in water content negatively strengthens the adverse effect of aging on the permanent axial strain. In addition, it should be emphasized that the effects of particle shape changes and fine fraction content on the permanent axial strain of aged ballast are considered simultaneously in this study, though the effect of particle shape changes was not evaluated in previous studies on fouled ballast (Indraratna et al., 2011; Ishikawa et al., 2019). Compared with those previous studies, test results in this study indicate that aged ballast has an unexpected smaller permanent axial strain than those of fresh and fouled ballasts under the air-dried condition, while it has a larger permanent axial strain than those of fouled ballast with clay or kaolin soils. The reasonable explanation for this phenomenon is that the smaller and rounder particles produced in the process of ballast breakdown and abrasion can provide a weakening effect in the aggregate matrix by preventing contact between large particles thus greatly reducing the aggregate matrix under the unsaturated and saturated conditions, though the fine particles can also provide a stabilizing effect by filling the voids among larger particles under the air-dried condition. Conclusively, the difference in permanent axial strain between fouled ballast and aged ballast demonstrates that when evaluating the permanent axial strain of aged ballast, it is essential to consider the influence of particle shape, fine fraction content, and water content comprehensively.



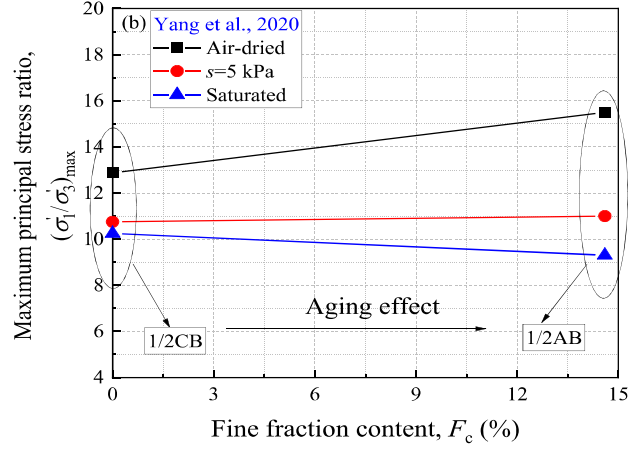


Figure 6-9 Effect of aging on $\varepsilon_{p, \max}$; and $(\sigma_1'/\sigma_3')_{\max}$: (a) $\varepsilon_{p, \max}$; (b) $(\sigma_1'/\sigma_3')_{\max}$

From the above analysis, the results of ML tests and CL tests illustrate that the τ_f and $\varepsilon_{p, \max}$ are affected by the water content and ballast aging. Thus, the relationships between the $\varepsilon_{p, \max}$ and τ_f with various water contents and drainage conditions are shown in Figure 6-10. As expected, the samples with lower τ_f appear the higher $\varepsilon_{p, \max}$, irrespectively of test samples and drainage conditions. This indicates that the effects of water content and ballast aging on the shear strength and permanent axial strain of ballast have similar phenomena, that is the increase in water content and ballast aging will cause a decrease in shear strength and an increase in permanent axial deformation of ballast, and the downward/upward trend of shear strength/permanent axial deformation is more remarkable for aged ballast in the high-water content condition.

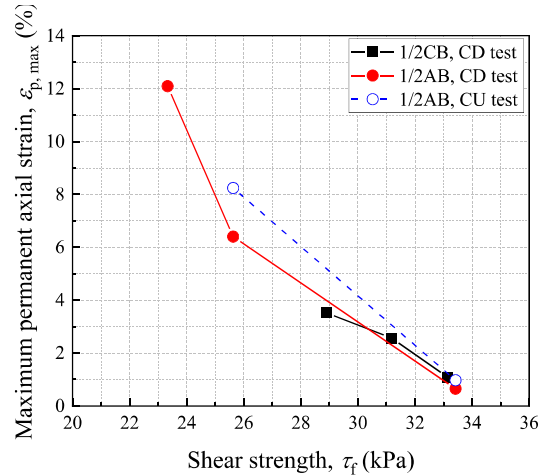


Figure 6-10 Relationship between $\varepsilon_{p, \max}$ and τ_f .

6.3 Summary

The influence of aging on cyclic plastic deformation of unsaturated ballast was evaluated in this chapter. The following findings can be mainly obtained:

- (1) The permanent axial deformation of ballast increases with the increase in water content, irrespective of test samples and drainage conditions, and wetting has a more significant impact on the permanent axial deformation of aged ballast as compared with fresh ballast. It can be inferred that the water content and aging effect synergistically affect the permanent axial deformation of ballast, and in the field conditions, the in-situ aged ballast with a high degree of saturation would appear greater permanent axial deformation than that of fresh ballast with a low degree of saturation.
- (2) For the aged ballast, the permanent axial deformation in CU tests is larger than that in CD tests, regardless of water contents. Besides, the effect of drainage condition on the cyclic plastic deformation of aged ballast becomes more significant as the degree of saturation increases due to the generation of excess pore water pressure.
- (3) The shear strength and permanent axial deformation are affected by the water content and aging. When other test conditions are constant, the unsaturated ballast with the lower shear strength in the monotonic loading condition has the higher permanent axial deformation in the cyclic loading condition, irrespective of test samples and drainage conditions. The decreasing/increasing trend of shear strength/permanent axial deformation is more remarkable for aged ballast, in case of the water content increases.

7 METHODS FOR ESTIMATION PERMANENT DEFORMATION OF UNSATURATED BALLAST

7.1 Prediction of Permanent Axial Deformation by UIUC model

7.1.1 Concept of UIUC model

The UIUC model was established to estimate permanent axial deformation of pavement materials under repeated traffic loads considering the effects of applied stress levels and shear strength properties (Qamhia et al., 2016). It belongs to a semi-empirical model, and its input parameters can be obtained from ML tests. Therefore, in this section, the applicability of the UIUC model is evaluated by comparing the predicted results by this model with measured results from CL tests. In this model (Figure 7-1), the applied deviator stress levels and effect of shear stress ratio should be considered in the prediction of permanent axial strain (ε_p) as listed in Eq. (7-1).

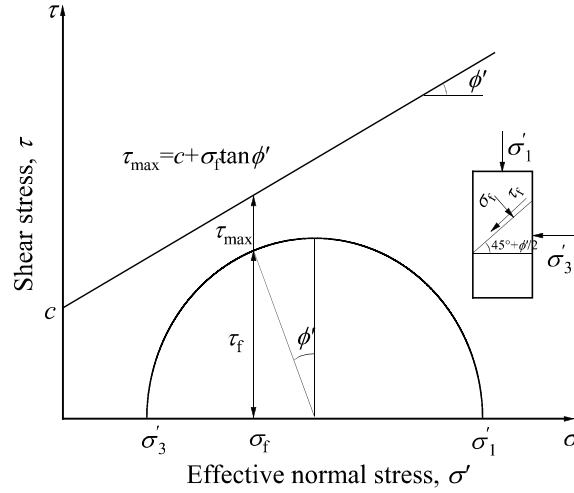


Figure 7-1 Concept of the UIUC model

$$\varepsilon_p = AN_c^B \sigma_d^C \left(\frac{\tau_f}{\tau_{max}} \right)^D \quad (7-1)$$

where, ε_p is the permanent axial strain corresponding to N_c applications (%); σ_d is the axial deviator stress (kPa); and A, B, C, D are regression parameters. Besides, τ_f and σ'_f are the effective shear stress and effective normal stress acting on failure plane calculated by

Eq. (7-2) and Eq. (7-3), respectively; $\tau_{\max}$ is the effective shear strength calculated by Eq. (7-4) determined by ML tests.

$$\tau_f = \sqrt{\left(\frac{\sigma_d}{2}\right)^2 - \left(\sigma_f - \left(\sigma'_3 + \frac{\sigma_d}{2}\right)\right)^2} \quad (7-2)$$

$$\sigma_f = \frac{2\sigma'_3 + 2\tan^2\phi'\sigma'_3 + \sigma_d + \tan^2\phi'\sigma_d - \sqrt{\tan^2\phi'\sigma_d^2 + \tan^4\phi'\sigma_d^2}}{2(1 + \tan^2\phi')} \quad (7-3)$$

$$\tau_{\max} = c + \sigma_f \tan\phi' \quad (7-4)$$

where, σ'_3 is minimum effective normal stress, and in triaxial tests, the σ_{net} ($\sigma_{\text{net}} = \sigma_c - u_a$) in the unsaturated condition, the σ_c in the air-dried condition, and σ'_c ($\sigma'_c = \sigma_c - u_w$) in the saturated condition were similar to the σ'_3 in the UIUC model; c is the total cohesion (kPa); ϕ' is the effective friction angle (deg.). It is noted that total cohesions (c) of both samples under air-dried and saturated conditions are equal to effective cohesion (c'). For an unsaturated specimen, the total cohesion (c) is composed of two parts: one is the effective cohesion (c') for the saturated soil, and the other is related to the matric suction.

In the UIUC model, the shear stress ratio (SSR), was used to identify the stress value in CL tests, which can be calculated as follows:

$$SSR = \frac{\tau_f}{\tau_{\max}} \quad (7-5)$$

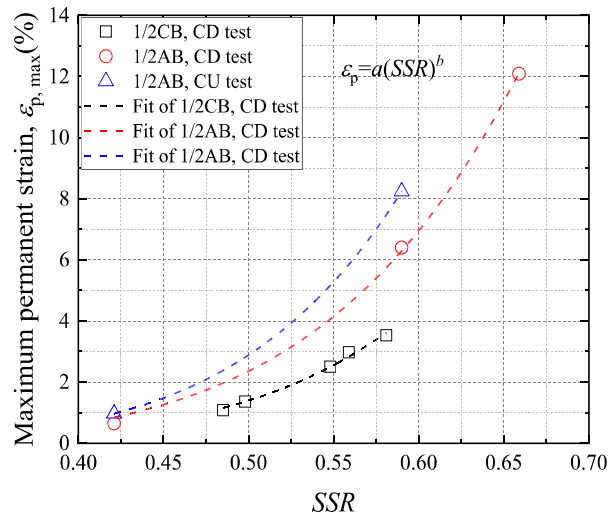


Figure 7-2 Relationship between $\varepsilon_{p, \max}$ and SSR .

Figure 7-2 reveals the relationship between the $\varepsilon_{p, \max}$ and the SSR under various water contents. The $\varepsilon_{p, \max}$ appears to vary exponentially with respect to the SSR , which can be inferred that the $\varepsilon_{p, \max}$ is affected by the SSR , irrespective of water content. Besides, the fitting curves between $\varepsilon_{p, \max}$ and SSR become different depending on test samples and drainage conditions, which means the regression parameters of UIUC model may need to be changed when considering these factors. These indicate that it is feasible to predict the permanent axial strain of fresh and aged ballasts with various water contents by the UIUC model. In this case, the input and the corresponding regression parameters of UIUC model for 1/2CB and 1/2AB are shown in Table 7-1 and Table 7-2, respectively. As shown in Table 7-2, 1/2AB has the higher A and D values than those of 1/2CB. It means that the aged ballast shows a higher initial permanent axial deformation, and that the effect of SSR on permanent axial strain becomes more pronounced for the aged ballast as compared with fresh ballast.

Table 7-1 Input parameters of UIUC model.

Sample	1/2CB	1/2CB	1/2CB	1/2CB	1/2CB	1/2AB	1/2AB	1/2AB
	Air-dried	$s=40$ kPa	$s=20$ kPa	$s=5$ kPa	Saturated	Air-dried	$s=5$ kPa	Saturated
σ_d (kPa)	80	80	80	80	80	80	80	80
σ_f (kPa)	25.77	25.93	27.11	27.11	27.11	25.57	29.58	29.58
τ_f (kPa)	20.69	20.97	22.77	22.77	22.77	20.36	25.98	25.98
$\tau_{\max}$ (kPa)	42.64	42.15	41.51	40.73	39.16	48.34	44.04	39.44
SSR	0.485	0.498	0.548	0.559	0.581	0.421	0.590	0.659

Table 7-2 Regression parameters of UIUC model.

Samples	Test conditions	Regression parameters				
		A	B	C	D	R^2
1/2CB	CD test	0.1938	0.1306	1.2130	6.5239	0.9801
1/2AB	CD test	0.3162	0.6843	0.1066	7.5684	0.9960
1/2AB	CU test (S)	0.3162	0.6843	0.1066	7.5684	-
1/2AB	CU test (D)	0.6020	0.6266	0.1022	6.8378	0.9397

CU test (S) and CU test (D) mean that the same and different regression parameters were adopted between CD tests and CU tests.

7.1.2 Applicability of UIUC model

Figure 7-3 compares the measured and estimated permanent axial strains of 1/2CB with different water contents. The estimated values by the UIUC model using the same regression parameters (Table 7-2) regardless of the difference in water content, are consistent with measured values even though the water contents are different. This indicates that the effect of water content on permanent axial deformation of fresh ballast can be effectively estimated by UIUC model. Moreover, in order to evaluate the applicability of this model to the prediction for the permanent axial deformation of aged ballast, Figure 7-4 compares the measured and estimated permanent axial strains of 1/2AB with different water contents and drainage conditions. It is noted that the regression parameters of 1/2AB are different from those of 1/2CB. It can be seen that the estimated permanent axial strains of 1/2AB by UIUC model are also consistent with measured values in CD tests (Figure 7-4 (a)), irrespective of water contents, while estimated values are difference from measured values in CU tests (Figure 7-4 (b)) by using the same regressions as the CD test. It is noted that when using the same regressions between the CD test and CU test, the input parameters of σ'_3 in the UIUC model was equal to σ'_c in CU tests. In the CU test, the input parameters of σ_d , τ_f , and τ_{max} in Eq. (7-1) at each loading cycle will be changed due to the change in the σ'_3 , which can be calculated by the Eqs. (7-2)~(7-4). After that, the permanent axial strain ($\varepsilon_{p,i}$) at different $N_{c,i}$ in the CU test can be calculated with different $\sigma_{d,i}$, $\tau_{f,i}$, and $\tau_{max,i}$ by Eq. (7-6). It is noted that the reason for the fluctuation of the predicted value is that the input parameters of $\sigma_{d,i}$, $\tau_{f,i}$, and $\tau_{max,i}$ are changed as the σ'_3 changes in CU tests, while these input parameters are constants in CD tests as shown in Table 7-1. These results prove that the applicability of the UIUC model to the prediction for the permanent axial deformation of aged ballast is limited in the CU test, since it is difficult to predict the change in excess pore pressure under cyclic loading.

On the other hand, when using different regression parameters between the CD test and CU test (Table 7-2), the estimated permanent axial strains by UIUC model are consistent with measured values in air-dried and unsaturated conditions, though the estimated value is smaller than that of the measured value in the saturated condition, as shown in Figure 7-4 (c).

The above results for fresh and aged ballasts prove that the permanent axial deformation of fresh and aged ballasts with various water contents can be well estimated by UIUC model by using the same regression parameters in CD tests, while the applicability of UIUC model to the prediction for permanent axial deformation of aged ballast may be limited in CU tests, due to the difficulty in predicting the excess pore pressure under cyclic loading.

$$\varepsilon_{p,i} = \varepsilon_{p,i-1} + A(N_{c,i})^B \sigma_{d,i}^C \left(\frac{\tau_{f,i}}{\tau_{\max,i}} \right)^D - A(N_{c,i-1})^B \sigma_{d,i}^C \left(\frac{\tau_{f,i}}{\tau_{\max,i}} \right)^D \quad (7-6)$$

where, $N_{c,i}$ represents the N_c is i ; $N_{c,i-1}$ represents the N_c is $(i-1)$; $\varepsilon_{p,i}$ is the permanent axial strain at the $N_{c,i}$; $\varepsilon_{p,i-1}$ is the permanent axial strain at the $N_{c,i-1}$; $\sigma_{d,i}$, $\tau_{f,i}$, and $\tau_{\max,i}$ are the cyclic deviator stress, effective shear stress, and effective shear strength at the $N_{c,i}$.

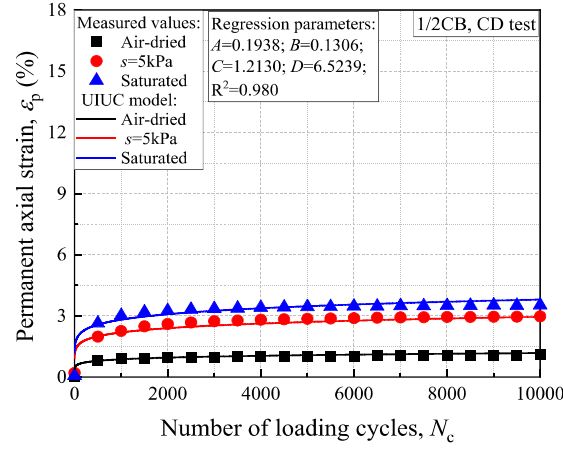
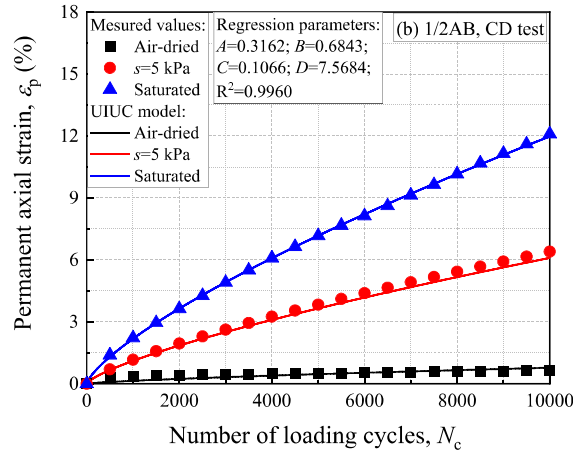


Figure 7-3 Comparison of ε_p for 1/2CB with various water contents in CD tests



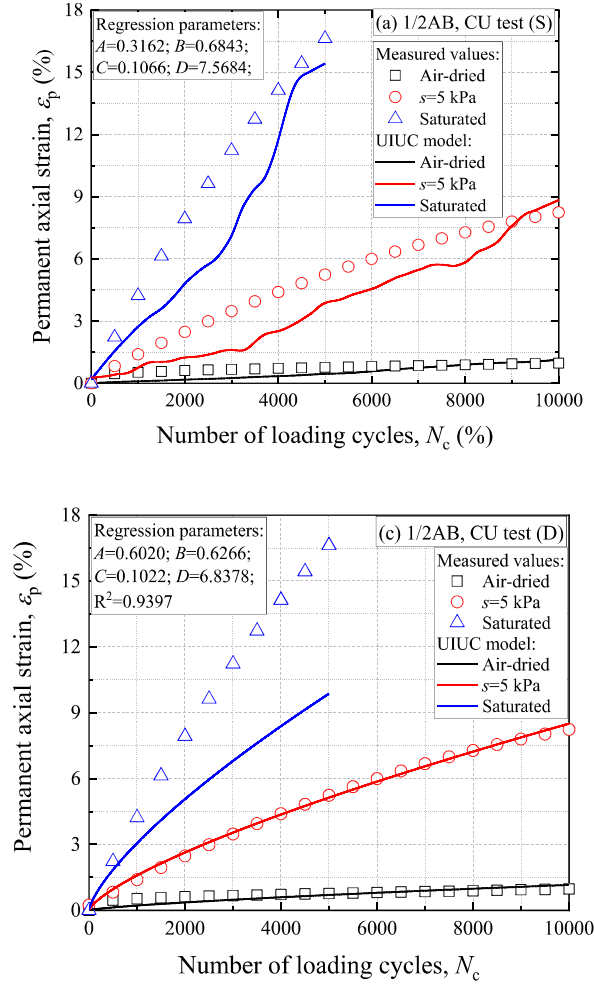


Figure 7-4 Comparison of ε_p for 1/2AB with various water contents: (a) CD test; (b) CU test (S); (c) CU test (D).

7.2 Prediction of Permanent Deformation by SSE model

7.2.1 Concept of SSE model

In elasto-plasticity theory, the strain rate $d\varepsilon_{ij}$ is additively decomposed into elastic strain rate $d\varepsilon_{ij}^e$ and plastic stretching $d\varepsilon_{ij}^p$ as follows:

$$d\varepsilon_{ij} = d\varepsilon_{ij}^e + d\varepsilon_{ij}^p \quad (7-7)$$

where $d\varepsilon_{ij}^e$ is given by

$$\varepsilon_{ij}^e = E^{-1} \circ \sigma \quad (7-8)$$

where σ is a effective stress, tensile stress is positive and compressive stress is negative. $(\circ)$ stands for the corotational rate. E is an elastic modulus, its fourth-order elastic tensor in Hooke's type is as follows:

$$E_{ijkl} = (K - 2/3 G) \delta_{ij} \delta_{kl} + G(\delta_{ik} \delta_{jl} + \delta_{il} \delta_{jk}) \quad (7-9)$$

where K and G are bulk and shear modulus, respectively. δ_{ij} is the Kronecker's delta.

In this model, the normal-yield, elastic boundary and subloading surfaces are defined as follows:

$$f(\sigma) = F(H) \quad (7-10)$$

$$f(\sigma) = R_e F(H) \quad (7-11)$$

$$f(\sigma) = R F(H) \quad (7-12)$$

where $f(\sigma)$ means loading function. H stands for the isotropic hardening variable. R_e is a ratio of the size of the elastic boundary surface to the size of the normal-yield surface (Tsutsumi et al., 2016). It is noted that the material will not have plastic deformation when the stress state lies inside the elastic boundary surface. R is a ratio of the size of the subloading surface to the size of the normal-yield surface. The normal-yield, elastic boundary and subloading surfaces are illustrated in Figure 7-5.

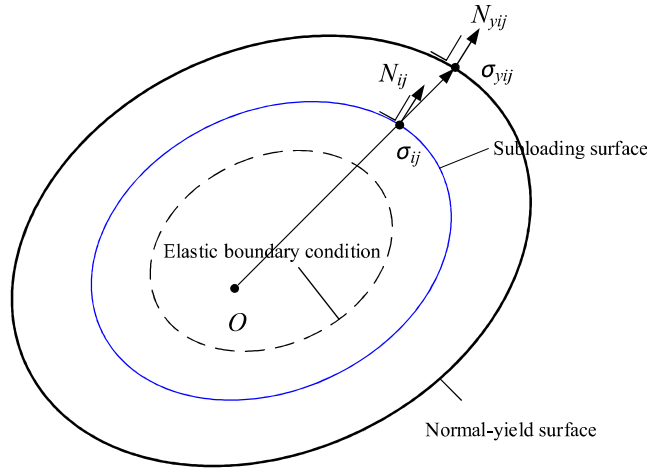


Figure 7-5 Subloading surface extension model

Time differentiation of Eq. (7-11) is given by

$$\text{tr}\left(\frac{\delta f(\sigma)}{\delta \sigma} \circ\right) = \dot{R}F(H) + R\dot{F} \quad (7-13)$$

where tr designates the trace. $(\cdot)$ stands for the material time derivative.

Considering that the stress or subloading surface would approach asymptotically to the normal-yield surface during the plastic deformation process, the evolution rate of R is assumed as

$$\dot{R} = U(R)\|\text{d}\varepsilon_{ij}^p\| \text{ for } \text{d}\varepsilon_{ij}^p \neq 0 \quad (7-14)$$

The material function of U governing the evolution rules of R . The function $U(R)$ can be given as

$$U(R) = \mu \cot\left(\frac{\pi}{2} \frac{R-R_e}{1-R_e}\right) \quad (7-15)$$

The elastic boundary surface is expanded by the accumulation of the plastic deformation. The evolution law of R_e is extended as

$$dR_e = U(R_e) \cot\left(\frac{R_e}{R_{e\max}}\right) \|\text{d}\varepsilon_{ij}^p\| \quad (7-16)$$

where $U(R_e)$ is the material constant, $R_{e\max}$ is the maximum value of the R_e .

The isotropic hardening/softening function of granular material is given by

$$F = (F_0 + p_i) \exp\left(\frac{H}{\lambda - \kappa}\right) - p_i \quad (7-17)$$

$$H = - \int \text{d}p_p^v \quad (7-18)$$

where F_0 is an initial value of the size of the normal yield surface. λ and κ are the slopes of the normal-consolidation and swelling line, respectively. p_i is the magnitude of the negative effective mean pressure to avoid infinite volume at null stress point. H is the volumetric strain.

For granular materials, the loading function $f(\sigma)$ shown in Eqs. (7-9) ~ (7-11) can be rewritten as

$$f(\sigma) = p \left(1 + \frac{\|\eta\|}{m}\right)^2 \quad (7-19)$$

$$p = -\frac{1}{3}\text{tr}\sigma, \sigma^* \equiv \sigma + pI, \eta \equiv \frac{\|\sigma^*\|}{p} \quad (7-20)$$

where p means average principal stress. σ^* designates deviatoric stress tensor. I means an identity tensor. m is a material constant to define the stress ratio $\|\sigma^*\|/p$ at the critical state line. η is the central axis of the normal-yield surface.

7.2.2 Applicability of SSE model

The reliability of the SSE model is discussed by comparing the results of CL tests for fresh and aged ballasts under different water contents and fine fraction contents with numerical results. The input parameters of SSE model for test samples were listed in Table 7-3. Here, the F_0 , μ and ϕ' can be obtained from the CL test. The λ and κ were determined can be determined from the isotropic compression test, which can be referred to previous study on air-dried ballast (Okayasu et al., 2014). The p_i , $U(R)$, $U(R_e)$, R_e are the material constants, which are constants with various water content. The $R_{e\max}$ changes with different water contents and test samples, which can be fitted by the results of CL tests.

Table 7-3 Input parameters of SSE model.

Sample	1/2CB	1/2CB	1/2CB	1/2AB	1/2AB	1/2AB
Drainage condition	CD test			CD test, CU test		
Water content	Air-dried	$s=5$ kPa	Saturated	Air-dried	$s=5$ kPa	Saturated
σ'_c	20	20	20	20	20	20
F_0	1078	887	842	1186	895	792
μ	0.3	0.3	0.3	0.3	0.3	0.3
ϕ'	58.9	55.3	55.3	59.4	49.5	49.5
λ	0.05	0.07	0.09	0.05	0.07	0.09
κ	0.0005	0.0005	0.0005	0.0005	0.0005	0.0005
p_i	45	45	45	45	45	45
$U(R)$	20	20	20	150	150	150
$U(R_e)$	4	4	4	25	25	25
R_e	0.001	0.001	0.001	0.001	0.001	0.001
$R_{e\max}$	0.150	0.067	0.060	0.047	0.037	0.022

Figure 7-6 compares the stress-strain relationship of 1/2CB under cyclic loading with various water contents between simulations and tests. As can be seen in this figure, the estimated axial strains of 1/2CB at different N_c consistent with the measured values, regardless of water content, though there is a slight difference between the estimated values and measured values. Moreover, Figure 7-7 compares the stress-strain relationship of 1/2AB under various water contents and drainage conditions between the simulations and tests. As presented in this figure, the axial strains of 1/2AB at different N_c obtained by SSE

model agree with the measured values in CD tests, irrespective of water contents. However, there is a significant difference between the estimated axial strains and measured values for the saturated specimen of 1/2AB in the CU test.

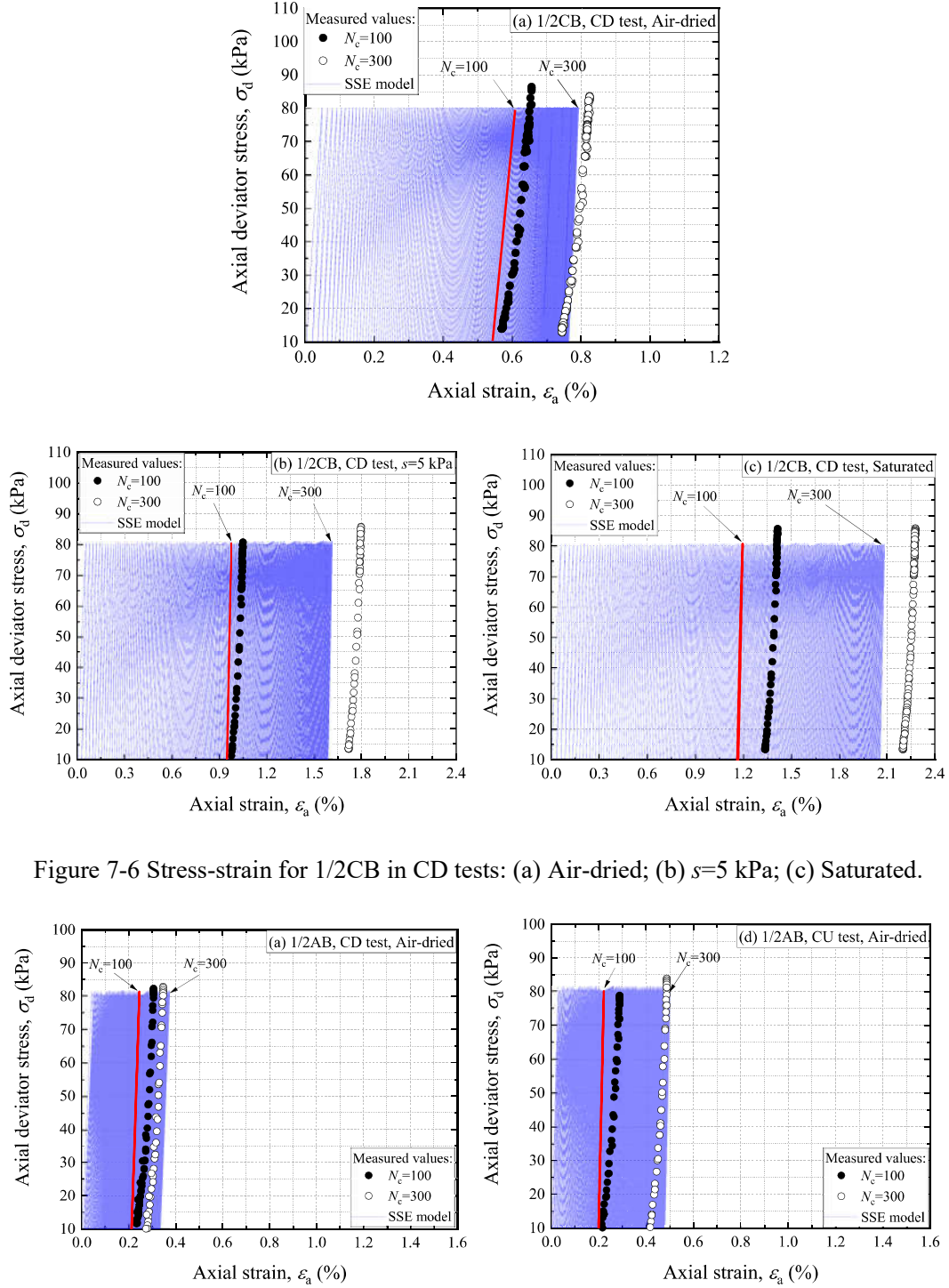


Figure 7-6 Stress-strain for 1/2CB in CD tests: (a) Air-dried; (b) $s=5$ kPa; (c) Saturated.

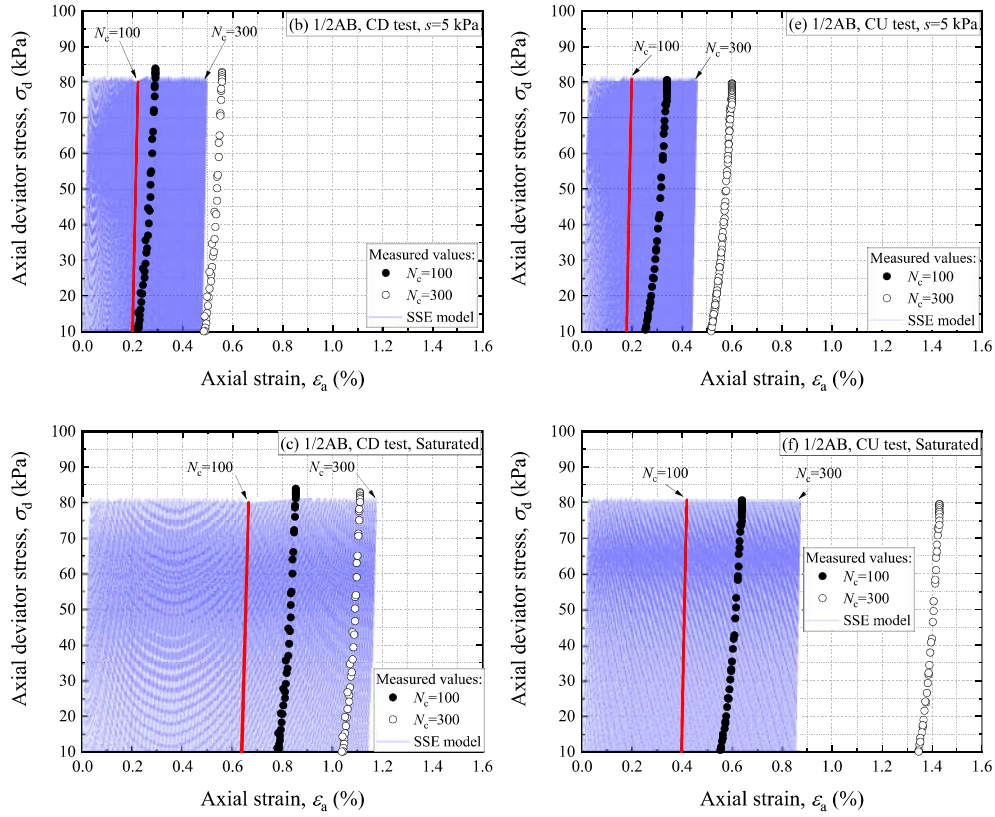


Figure 7-7 Stress-strain relationship for 1/2AB: (a) Air-dried, CD test; (b) $s=5$ kPa, CD test; (c) Saturated, CD test; (d) Air-dried, CU test; (e) $s=5$ kPa, CU test; (f) Saturated, CU test.

Figure 7-8 shows the comparisons of the permanent axial strain of 1/2CB with various water contents between simulations and tests. It can be found that the predicted values by the SSE model are consistent with measured values, regardless of water content. In addition, Figure 7-9 compares the simulated and measured permanent axial strains of 1/2AB in different drainage conditions. As can be seen in Figure 7-9(a), the permanent axial strains of 1/2AB with different water contents in CD tests can be well predicted by SSE model. However, as shown in Figure 7-9(b), there is some differences between the estimated and measured permanent axial strain of 1/2AB in CU tests, regardless of water contents. This is because only the mechanical model by the SSE model was adopted in this study, which causes the SSE model cannot accurately predict the change in effective stress changes under cyclic loading. These results prove that it is better to establish the hydro-mechanical model to improve the range and prediction accuracy of the SSE model for aged ballast in the undrained condition.

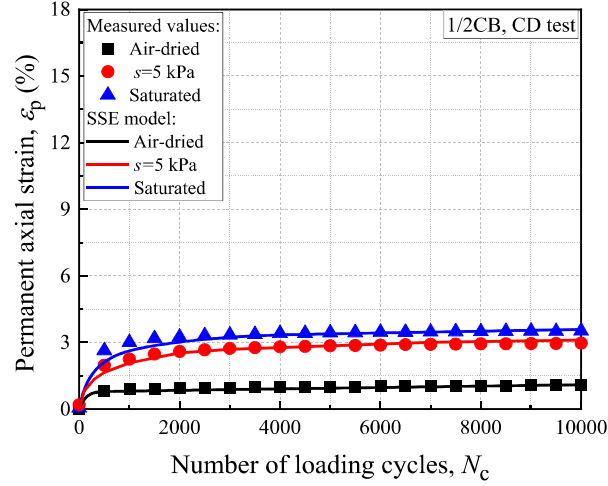


Figure 7-8 Comparison of ε_p for 1/2CB between simulations and tests.

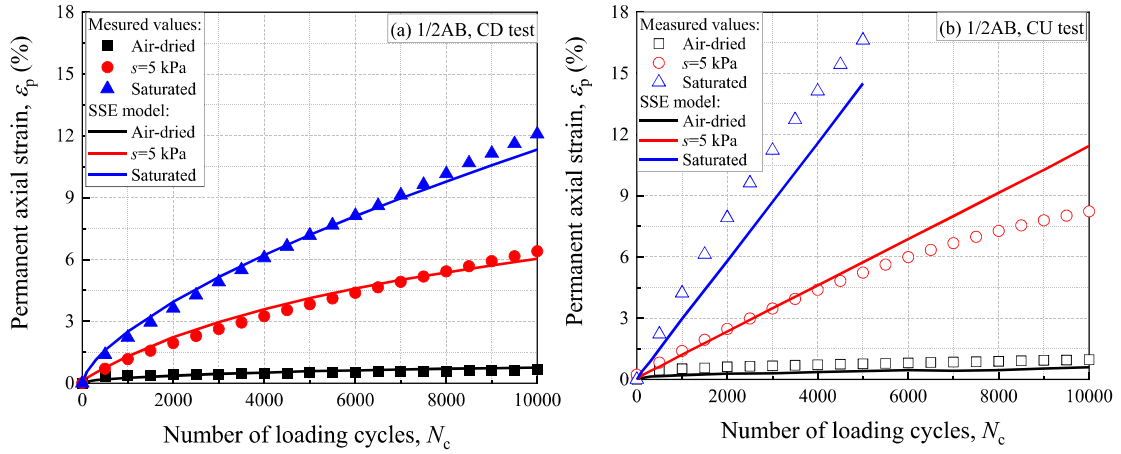


Figure 7-9 Comparisons of ε_p for 1/2AB between simulations and tests: (a) CD test (b) CU test.

To analyze the influence of water content on input parameters of SSE model for both samples, the changes in normalized input parameters (Here, the value of each input parameter of 1/2CB in the air-dried condition is taken as the standard value, and the values of normalized input parameters under various test conditions are the ratio of original values to standard values.) with the effective degree of saturation (S_e) for 1/2CB and 1/2AB are shown in Figure 7-10. It shows that the F_0 decreases with the increase in water content, regardless of test samples. As for the $U(R)$, $U(R_e)$ and R_e , they are unchanged with various water contents, regardless of test samples. Besides, the R_{emax} decreases with the increment of S_e , irrespective of test samples and drainage conditions. As for the ϕ' in the unsaturated

condition, Fredlund et al. (1978) proposed that an increase in matric suction (s) results in an increase in the apparent cohesion while maintaining the same ϕ' as the saturated soils. In this case, when other test conditions are constant, the ϕ' maintains constant, even though the S_e changes, except in the air-dried condition. Moreover, as the S_e increases, the λ increases while the κ does not change, which is consistent with the results of Alonso et al. (1990).

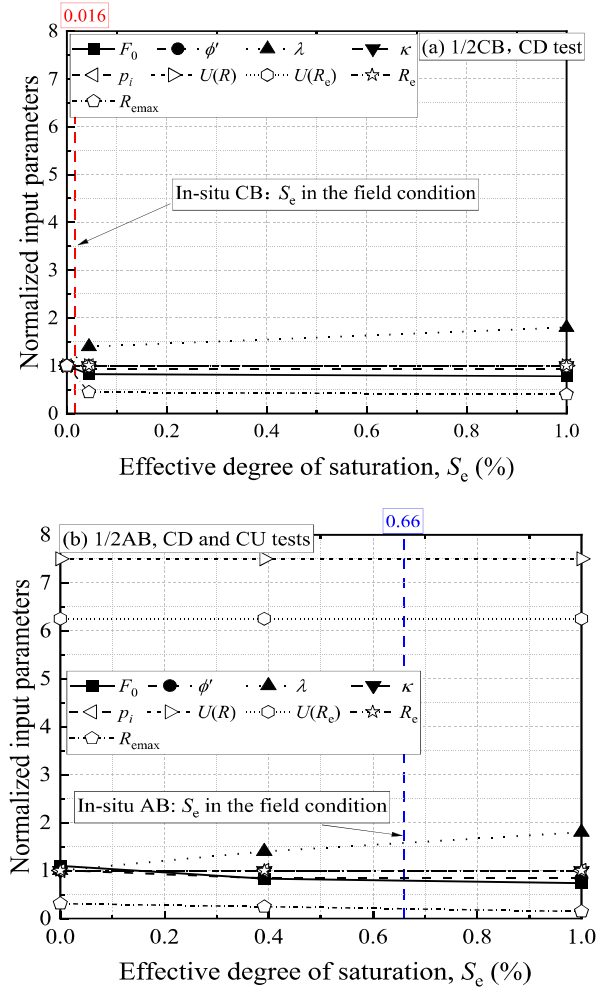


Figure 7-10 Relationship between normalized input parameters and S_e of test samples: (a): 1/2CB in CD tests; (b) 1/2AB in CD and CU tests.

7.3 Summary

In the chapter, the applicability of UIUC model and SSE model was verified to the prediction for the permanent axial deformation of unsaturated ballast by considering the effect of aging. The following findings can be mainly obtained:

- (1) The permanent axial deformation of ballast depends on the *SSR* in CD tests, irrespective of water content and test samples. Accordingly, the influence of water content on the permanent axial deformation of fresh and aged ballasts can be well estimated by the UIUC model with the same regression parameters, even though the water contents are different. However, in the unsaturated and saturated conditions, the UIUC model cannot well estimate the permanent axial deformation of aged ballast in CU tests, since it is difficult to estimate the change in the excess pore pressure.
- (2) The SSE model can well predict the permanent axial deformation of fresh and aged ballasts with different water contents in CD tests. However, in CU tests, the effect of water content on the permanent axial deformation of aged ballast cannot accurately predict by the SSE model, since a mechanical model was adopted in this study.

8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions and Recommendations

The following conclusions can be derived from this study:

(1) Hydraulic properties of railroad ballast are seriously affected by aging.

- Results of water retention test with different specimen heights indicate that the water retentivity of test samples was affected by the specimen height. However, the presence of fine fraction content in aged ballast remarkably increases the water retentivity compared with fresh ballast.
- Results of saturated permeability test show that both degree of compaction and fine fraction content have an influence on the saturated coefficient of permeability, while it is hardly affected by the specimen height. When other test conditions are constant, the saturated coefficient of permeability is remarkably reduced with the increase of fine fraction content, though it is slightly decrease as the degree of compaction increases.
- Results of unsaturated permeability test prove that the unsaturated coefficient of permeability of aged ballast is much lower than that of clean ballast, though the relative permeability coefficient of fresh ballast is more significantly affected by the water content.

(2) A series of monotonic loading triaxial compression tests (i.e., CD and CU tests) were conducted to evaluate the shear strength behavior of unsaturated fresh and aged ballasts:

- The effective shear strength parameters of fresh ballast with different water contents can be obtained from the CU and CD tests. However, for the aged ballast, the effective shear strength parameters are obtained more accurately from the CU test, due to the low permeability of aged ballast.

- The presence of fine fraction content in aged ballast reduces the effective internal friction angle, while it increases the total cohesion as compared with the fresh ballast, especially under the high saturation condition.
- When considering the effect of water content, the effective internal friction angles of fresh and aged ballast decrease significantly from the air-dried state to the unsaturated state, while they remain unchanged from the unsaturated state to saturated state, and the total cohesions decrease from the residual value to the saturated state. Thus, the shear strength of fresh and aged ballast decreases as the water content increases, and the declining trend of the shear strength is more noticeable in the aged ballast.
- The shear strength of fresh ballast is overall larger than that of aged ballast in the field condition. This is because the degree of saturation of aged ballast in the field condition is generally much higher than that of fresh ballast, due to its poor permeability, resulting in the poor mechanical performance of aged ballast. The higher effective confining pressure caused by the train passing further increases the difference in shear strength between the fresh ballast and aged ballast.

(3) The influence of aging on permanent axial deformation of unsaturated ballast was evaluated by cyclic loading triaxial compression tests:

- The permanent axial deformation of ballast increases with the increase in water content, irrespective of test samples and drainage conditions, and wetting has a more significant impact on the permanent axial deformation of aged ballast as compared with fresh ballast. It can be inferred that the water content and aging effect synergistically affect the permanent axial deformation of ballast, and in the field conditions, the in-situ aged ballast with a high degree of saturation would appear greater permanent axial deformation than that of fresh ballast with a low degree of saturation.
- For the aged ballast, the permanent axial deformation in CU tests is larger than that in CD tests, regardless of water contents. Besides, the effect of drainage condition on the cyclic plastic deformation of aged ballast becomes more significant as the degree of saturation increases due to the generation of excess pore water pressure.

- The permanent axial deformation of ballast increases with the increase in water content, irrespective of test samples and drainage conditions, and wetting has a more significant impact on the permanent axial deformation of aged ballast as compared with fresh ballast. It can be inferred that the water content and aging effect synergistically affect the permanent axial deformation of ballast, and in the field conditions, the in-situ aged ballast with a high degree of saturation would appear greater permanent axial deformation than that of fresh ballast with a low degree of saturation.

(4) Applicability of UIUC model and SSE model was verified to the prediction for the permanent axial deformation of ballast by considering effects of aging, water content, and drainage condition:

- The influence of water content on the permanent axial deformation of fresh and aged ballasts in CD tests can be well estimated by the UIUC model with the same regression parameters, even though the water contents are different. However, the UIUC model is limited to estimate the permanent axial deformation of aged ballast in CU tests, since it is difficult to predict the change in excess pore water pressure under cyclic loading.
- The SSE model can well predict the permanent axial deformation of fresh and aged ballasts with different water contents in CD tests. However, in CU tests, the effect of water content on the permanent axial deformation of aged ballast cannot accurately predict by the SSE model, since a mechanical model was adopted in this study.

The findings of this research indicate that it is necessary to consider the effects of the drainage condition and water content on the hydro-mechanical behavior of the aged ballast, due to the low permeability and high residual degree of saturation. For the fresh ballast, the air-dried specimen in triaxial tests under the fully drained condition can reproduce the actual conditions. However, for the aged ballast, the unsaturated and saturated specimens in triaxial tests under partially and fully undrained conditions can reproduce the actual conditions. Besides, both the UIUC model and SSE model can well predict the permanent axial deformation of clean and aged ballasts in CD tests. However, the applicability of the UIUC model to the prediction for the permanent axial deformation of aged ballast is limited,

since it is difficult to predict the excess pore pressure under cyclic loading. Therefore, for the actual design, the UIUC model is suitable for predicting the permanent axial deformation of fresh ballast or slightly aged ballast under the drained condition, and the SSE model shows a good potential to estimate the permanent axial deformation of fresh and aged ballasts by considering the effects of water content and drainage condition.

8.2 Future Assignments

- (1) The findings in this study were obtained under limited test conditions (only one unsaturated state and one fine fraction content). Further laboratory element tests (i.e., water retention tests, saturated/unsaturated permeability tests, and triaxial compression tests) considering more different confining pressures, fine fraction contents, and degrees of saturation need to be conducted in the future to comprehensively investigate the hydro-mechanical behavior of aged ballast.
- (2) The applicability of UIUC model to the prediction for permanent axial deformation of aged ballast in the CU test still needs to further study.
- (3) Further examination of the validity, limitation of application, calibration method for regression parameters in the UIUC model needs to be conducted in future in order for the outcomes of this research to be practically applicable.
- (4) Only the mechanical model by the SSE model was adopted in this study. It is better to establish the hydro-mechanical model to improve the range and prediction accuracy of the SSE model for aged ballast in the undrained condition.
- (5) It is meaningful to establish a three-dimensional model for the whole track structure in terms of the SSE model to make this model practice. However, the applicability of SSE model with different stress distributions and complex boundary conditions still needs to further research.

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LIST OF NOTATIONS

Description	Symbol
cross-sectional area of the specimen in permeability test	A
total cohesion	c
effective cohesion	c'
specimen diameter	D
degree of compaction	D_c
mean grain size	D_{50}
hardening function	F
initial value of the size of the normal yield surface	F_0
fine fraction content	F_c
specimen height	H
saturated coefficient of permeability	k_s
unsaturated coefficient of permeability	k_{us}
time of Los Angeles abrasion test	L
liquid limit	LL
plastic limit	PL
plastic index	PI
critical state effective stress ratio	M
number of rotations	N_t
number of loading cycles	N_c
outflow flow rate	Q
deviator stress	q
maximum deviator stress	$q_{\max}$
ratio of subloading surface to normal yield surface	R

ratio of elastic surface to normal yield surface	R_e
maximum value of R_e	$R_{e\max}$
saturated coefficient of permeability ratio	R_{ks}
matric suction	s
degree of saturation	S_r
effective degree of saturation	S_e
residual degree of saturation	S_{rr}
maximum degree of saturation	S_{rs}
pore air pressure	u_a
pore water pressure	u_w
uniformity coefficient	U_c
material constant for elastic properties	$U(R_e)$
material constant for R	$U(R)$
water content	w
total weight of unsaturated sample	W
weight of dry sample	W_0
optimum moisture content	w_{opt}
maximum effective stress ratio	$\eta_{\max}$
volumetric water content	θ
slope of the normal-consolidation line	λ
slope of the swelling line	κ
effective internal friction angle	ϕ'
internal friction angle associated with matric suction	ϕ^b
shear strength	τ_f
specific density	ρ_s
dry density after compaction	ρ_{dc}
in-situ measured dry density	ρ_{dm}

maximum dry density	ρ_{dmax}
axial strain	ε_{a}
axial strain rate	$\dot{\varepsilon}_{\text{a}}$
volumetric strain	ε_{v}
permanent strain	ε_{p}
maximum permanent strain	$\varepsilon_{\text{p,max}}$
water gravity	γ_{w}
maximum principal stress	σ'_1
minimum principal stress	σ'_3
confining pressure	σ_{c}
effective confining pressure	σ'_{c}
overburden pressure	σ_{v}
net normal stress	σ_{net}
principal stress ratio	(σ'_1/σ'_3)
maximum principal stress ratio	$(\sigma'_1/\sigma'_3)_{\text{max}}$
