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**Experimental study on the effect of suffusion on mechanical
properties of densely compacted pumice sand in Satozuka
District, Sapporo, Japan**

**札幌市里塚地区火山灰質砂の密な締固め状態における内部
侵食が力学特性に及ぼす影響に関する実験的研究**

A dissertation submitted in partial fulfilment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

in Engineering

by

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Under the guidance and supervision of

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Abstract

Satozuka district in Sapporo, Japan, stands on a layer of pumice sand of volcanic origin (or pyroclastic flow deposit) formed approximately 40,000 years ago from the eruption of Mount Shikotsu (currently caldera of Lake Shikotsu). This soil contains a high fraction of non-plastic fines (typically greater than 40%). The suburb of Satozuka located in Sapporo city's south-east part was developed by excavating the hills made of pumice sand and filling into the valleys in preparation to meet the growing demand for residential land particularly in the 1970s. An underground conduit network was installed in this area during the land development process to regulate the flow of groundwater. Evidence of suffusion was observed inside the underground drainage pipes installed, during the geotechnical investigation conducted after the Hokkaido Eastern Iburi Earthquake occurred in 2018.

Suffusion is a sort of internal erosion, which is the process of migration of fine soil particles through coarse soil particles under the action of a hydraulic gradient. This phenomenon can alter the microstructure of natural and artificial soil deposits, thus changing the geo-mechanical behavior of soils in different stress states. Instances of experimental studies on the effect of suffusion on the mechanical behavior of different types of soils are available in the literature which implies that the mechanical behavior of suffusion-induced soil is very complex and differs depending on factors such as soil type, gradation, packing, test conditions etc. Moreover, studies on the influence of suffusion on the mechanical properties of well-graded volcanic soil containing a high fraction of fines are limited. As Sapporo is the central hub in the Hokkaido region of Japan and is still growing, the future construction sites in the expanded Sapporo city will have a requirement of a high degree of compaction

(preferably over 95%). The influence of suffusion on the mechanical behavior of densely packed volcanic soil meeting the requirement of the high degree of compaction remains a key issue. Furthermore, the influence of variation in the degree of saturation (or moisture content) during compaction on suffusion has scarcely been studied for volcanic soil denoting a deficiency in understanding the role of soil microfabric in suffusion.

The aim of this study is to assess the suffusion characteristics and the impact of suffusion on pumice sand with a high proportion of fines, based on its mechanical properties, with an emphasis on shear strength and dilatancy particularly focusing on high-density specimens with a variation in moisture. Firstly, the physical properties of the soil are evaluated, followed by the assessment of its stability against internal erosion. Subsequently, suffusion characteristics of Satozuka pumice sand are evaluated with respect to the influence of the moisture content (30% to 52%) and degree of compaction (80% to 100%) at a hydraulic gradient of 200. This is followed by the conduct of consolidated undrained triaxial tests under monotonic loading on specimens, with suffusion, and without suffusion, to compare the impact of suffusion on the shear strength, stress path, and dilatancy properties. Next, the degree of compaction of the suffusion-induced soil is correlated with the maximum deviator stress, brittleness, critical state, and dilatancy. Then, the results are compared to those of specimens without suffusion.

It is observed from the results that the extent of suffusion increases with a decrease in the degree of compaction during the specimen preparation due to changes in the constriction size and inter-particle grain-to-grain contact. Furthermore, for equally compact specimens, the degree of saturation (or moisture content) during the

specimen preparation adversely affects the extent of suffusion due to the formation of an inter-pore network called macro-porosity in specimens with low saturation. Moreover, the hydraulic conductivity, shear strength, stress paths, and dilatancy are all noticeably affected by suffusion. Suffusion increases the hydraulic conductivity of pumice sand by enlarging the flow path due to the removal of fines. The specimens with suffusion exhibit an increase in residual shear strength and maximum deviator stress under shearing and experience an earlier occurrence of phase transformation from contraction to dilation during shearing. This tendency implies that suffusion has no significant negative impact on the deterioration of earthfill made of pumice sand and that it persists at degrees of compaction between 80% to 100%.

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Symbols and Notations

A	Area of the specimen
CSL	Critical state line
CP	Cell pressure
C_c	Coefficient of curvature
C_u	Coefficient of uniformity
d_{85}^f	Representative grain size of the finer fraction
D	Soil particle diameter
D_c	Degree of compaction attained after specimen preparation
D_{ci}	Constriction size when i % of constrictions are smaller
$D_{c,t}$	Target degree of compaction
D_i	Soil particle diameter at which i % of the soil is finer
D_L	Deviator load
D_r	Relative density
ESP	Effective stress path
F	Mass of fraction smaller than diameter, D of soil at any point on the grain size distribution curve
FC	Fines content
h'	Slope of the coarse fraction
h''	General slope of soil gradation curve
H	Mass of fraction measured between D and $4D$
i	Target point in stereophotogrammetry
i	Hydraulic gradient
I_B	Brittleness index
k	Hydraulic conductivity
k_{i50}	Hydraulic conductivity at $i = 50$
k_{i200}	Hydraulic conductivity at $i = 200$
m_d	Dry mass of specimen
$M_{\max}$	Maximum slope of ESP
$M_{q_{\max}}$	Slope of ESP at $q_{\max}$
n	Number of stages

OMC	Optimum moisture content
p'	Effective mean stress
PTP	Phase transformation point
PWP	Pore water pressure
q	Deviator stress
$q_{\max}$	Maximum deviator stress or peak deviator stress
q_R	Residual shear strength
Q	Flow rate
S	Secant slope of soil gradation curve
S_f	Value of percentage finer at the junction of coarse and fine fractions
S_r	Degree of saturation
t	Flow duration
T	Turbidity
u, v	2-dimensional coordinates in image space
U	Voltage
V	Volume of specimen
V_n	Volume of specimen at any stage n
V_e	Volume of effluent
w	Moisture content
WoS	Without suffusion
WS	With suffusion
X_w, Y_w, Z_w	3-dimensional coordinates of target point in real physical space
Δm_d	Change in dry mass of specimen
ΔV	Change in volume of specimen
Δu	Excess pore water pressure
$\Delta u_{15\%}$	Excess PWP at 15% strain
Δu_{PTP}	Excess PWP at PTP
ε_a	Axial strain
$\varepsilon_{a,PTP}$	Axial strain at PTP
ε_r	Radial strain
ε_v	Volumetric strain

ρ_d	Dry density
ρ_{dmax}	Maximum dry density
ρ_{max}	Maximum density
ρ_{min}	Minimum density
σ'_c	Effective consolidation stress
ϕ_{max}	Maximum shear resistance angle
ϕ_{qmax}	Shear resistance angle at q_{max}

Chapter 1

INTRODUCTION

1.1 Background

Satozuka district of Sapporo city, Japan, stands on a layer of pumice sand of volcanic origin (pyroclastic flow deposit or spfl) derived approximately 40,000 years ago from Mount Shikotsu (currently caldera of the Lake Shikotsu) (Fig. 1.1). The suburb of Satozuka located in Sapporo city's south-east part was developed by excavating the hills made of pumice sand and filling into the valleys in preparation to meet the growing demand for residential land particularly in the 1970s (Fig. 1.2).

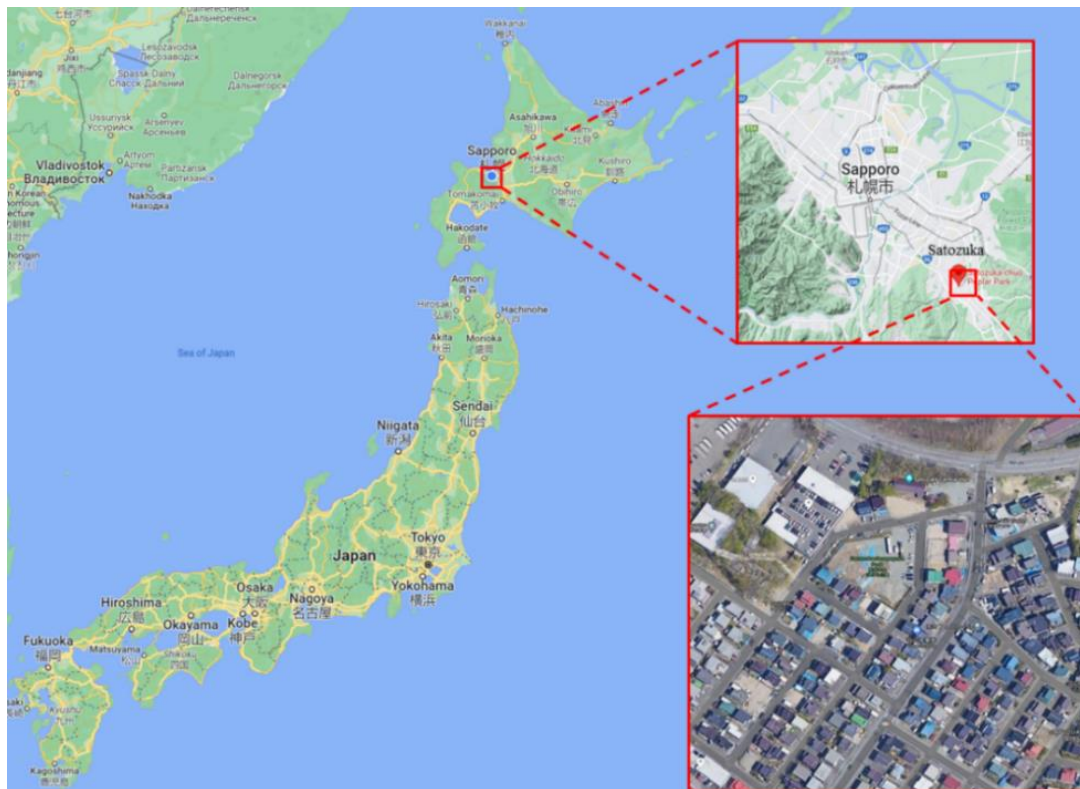
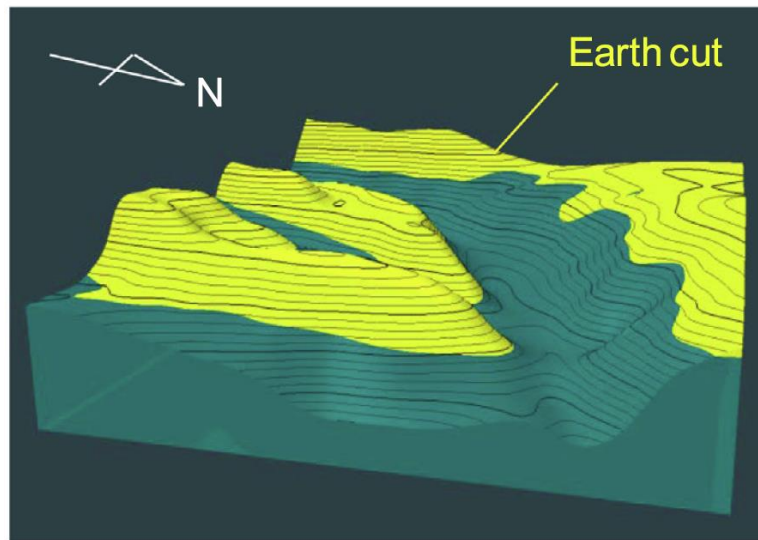
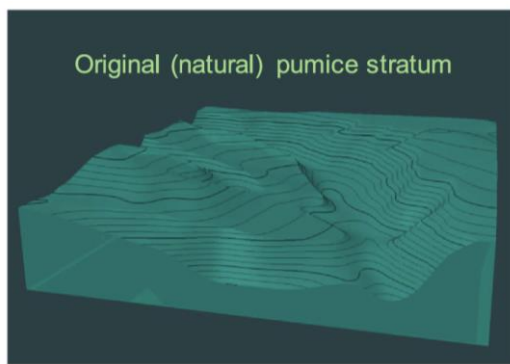


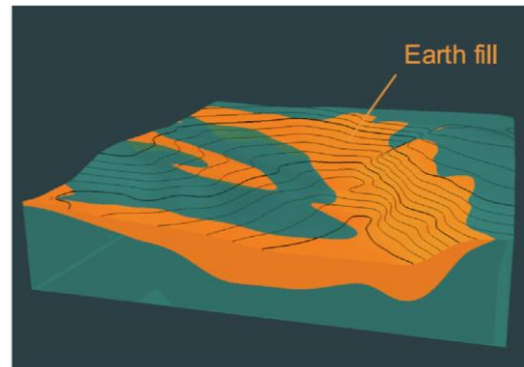
Fig. 1.1. Location and aerial view of the artificially developed Satozuka suburb, Japan
(source: <https://www.google.co.jp/maps>).



(a) Before site development



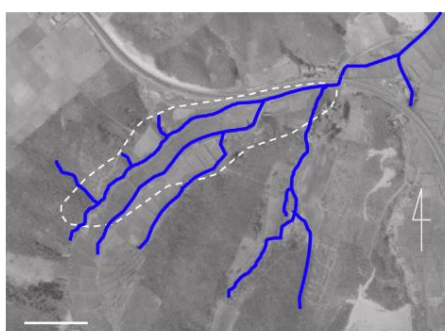
(b) After earth cut



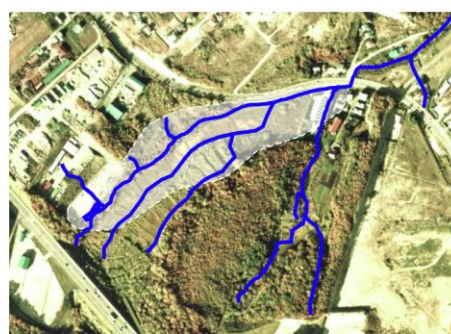
(c) After earth fill

Fig. 1.2. Estimated history of site development in Satozuka based on elevation data (Watabe and Nishimura, 2020).

An underground conduit network was installed in this area during the land development process to regulate the flow of ground water (Fig. 1.3). Evidence of suffusion in the form of accumulation of fines around drainage pipe outlets and layers of muddy cake was observed inside the underground drainage pipes installed, during the geotechnical investigation conducted after the Hokkaido Eastern Iburu Earthquake occurred in 2018 (Fig. 1.4).



(a) Years 1961-1969



(b) Years 1974-1978



(c) Years 1984-1987

Fig. 1.3. Aerial photographs of Satozuka suburb with location of underground conduit and drain pipelines (Watabe and Nishimura, 2020).



(a) Accumulation of fines around drainage pipe outlet



(b) Layers of muddy cakes

Fig. 1.4. Evidence of suffusion inside the underground drainage pipes installed beneath the artificially developed area.

1.2 Motivation and Scope

The pumice sand particular to Satozuka suburb is originated from the pyroclastic flow deposit during the volcanic eruption of Mount Shikotsu. Pyroclastic flow deposit or Spfl is formed due to deposition of fast-moving currents of hot lava, ash and volcanic rock fragments (known as pyroclastic flow); as it losses momentum or comes to a halt due to presence of obstructions; when rushing down the sides of a volcano during eruption. Pumice sand from satozuka region is a coarse-grained volcanic soil characterized by porous and brittle particles and typically contains more than 40% non-plastic fine particles. Such high non-plastic fine content can significantly affect the density, porosity, permeability, strength, and stability of a soil ultimately influencing the mechanical behavior of a soil.

Suffusion is a sort of internal erosion, which is the process of migration of fine soil particles through coarse soil particles under the action of a hydraulic gradient. Suffusion is dependent upon various factors such as soil type, soil density, hydraulic conductivity, degree of saturation etc. Watabe et al. (2021) experimented on erodibility of clayey sand (with grain size distribution similar to pumice sand with plastic fines) and established that the degree of saturation (a function of moisture content and compaction condition) affects the erodibility in soil. The influence of variation in degree of saturation on suffusion characteristics has scarcely been studied for well graded volcanic soil with high non-plastic fine content denoting a deficient in understanding the role of soil microfabric in suffusion.

The suffusion phenomenon can alter the microstructure of natural and artificial soil deposits, thus changing the geo-mechanical behavior of soils in different stress states (Chang and Zhang, 2011; Xiao and Shwihyat, 2012). Geo-mechanical behavior

of natural and compacted soils is dependent on the arrangement of solid soil components and soil pores, referred to as soil microstructure (Romero et al., 1999; Tang et al., 2020). Therefore, suffusion can compromise the stability and integrity of artificially compacted soil structures or earthen constructions, such as earth fills, embankments, dams etc.; by changing the mechanical behavior of soil; thus making it an important consideration in geotechnical engineering and soil mechanics. Instances of experimental studies on effect of suffusion on mechanical behavior of different types of soil are available in the literature. These previous studies imply that the mechanical behavior of suffusion-induced soil is very complex and differs depending on factors such as soil type, gradation, packing, test conditions etc. The occurrence of suffusion in volcanic soil will change its mechanical behavior. It is therefore crucial to explore how suffusion affects the mechanical behavior of pumice sand deposits.

Tatsuoka et al. (2016) recommended that artificially compacted soils should achieve a degree of compaction over 90%, preferably over 95% for design and construction of earth fills. As Sapporo is the central hub in the northern part of Japan that is still growing, the future construction sites in expanded Sapporo city will have a requirement of high degree of compaction. The prior study on internal erosion of volcanic ash by Chitravel et al. (2021) was in loose packing (relative density upto 80%) which was similar to in-situ conditions in the Satozuka region as it subsists at present (artificially compacted approximately 50 years ago). The influence of suffusion on the mechanical behavior of densely packed volcanic soil meeting the construction management practice recommended above (degree of compaction over 90% or 95%) still remains a key issue. This study aims to investigate whether densely compacted pumice sand containing high fraction of fines is secured against suffusion

or not on the basis of undrained shear strength, as the effect of suffusion on mechanical properties of pumice sand in this type of compaction condition has not been previously studied.

More specifically, the following research questions will be addressed in this study-

1. Is the pumice sand originated from pyroclastic flow deposit susceptible to suffusion? If yes, what will be the influence of compaction conditions and degree of saturation on suffusion characteristics of pumice sand?
2. What will be the effect of suffusion on the mechanical properties of the pumice sand prepared with different compaction conditions?
3. How suffusion will change the microstructure of the soil leading to change in its mechanical behavior?

1.3 Objectives

To overcome the above-mentioned limitations, this study describes the results of a laboratory study of the mechanical behavior of suffusion-induced Satozuka pumice sand, particularly focusing on high-density specimens with moisture variation with the following objectives-

1. To assess the suffusion susceptibility of pumice sand after material characterization.
2. To analyze the suffusion characteristics of pumice sand with respect to the influence of degree of saturation, and degree of compaction.
3. To evaluate the effect of suffusion on mechanical behavior of pumice sand with the help of consolidated undrained triaxial tests on specimens with and without suffusion; in consideration to degree of saturation, and degree of compaction.

This is followed by establishment of correlations between variation in degree of compaction with changes in normalized maximum deviator stress, and normalized excess pore water pressure for with suffusion and without suffusion specimens.

4. To interpret the changes in soil microstructure due to suffusion from the results of the experimental study.

1.4 Organization of the Thesis

This thesis is organized into seven chapters in total. Chapter 1 introduces the background, motivation, scope, objectives of this study, and organization of the thesis. Chapter 2 introduces the importance of suffusion with a literature review of the existing theoretical, experimental as well as numerical studies on the topic of suffusion, factors controlling suffusion, criteria for assessment of suffusion susceptibility of soils, and its influence on the physical as well as mechanical properties of soil. Chapter 3 introduces the testing material with a description its physical properties followed by assessment of internal instability of pumice sand. Chapter 4 describes the testing apparatus and testing program for different specimens. Chapter 5 presents the suffusion characteristics of specimens with varying degrees of saturation and degree of compaction. Chapter 6 presents the experimental results obtained from the consolidated undrained triaxial shear test for with suffusion and without suffusion specimens. The results were analyzed and interpreted based on shear strength, stress paths, and dilatancy properties. The resulting observations were then interpreted to infer the changes in soil microstructure. Chapter 7 summarizes the conclusions drawn from this study, recommendations from this study, and the scopes for future study are also suggested.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

Internal erosion phenomena occur in soils when solid particles are detached from the solid phase and transported by the hydromechanical force of seepage flow due to a hydraulic load that exceeds the erosion resistance of the material. These phenomena are widely observed in hydraulic earth structures, such as embankment dams and levees, and imply a particular threat (Stewart and Watts, 2000; Zhang and Chen, 2006; Fell and Fry, 2007).

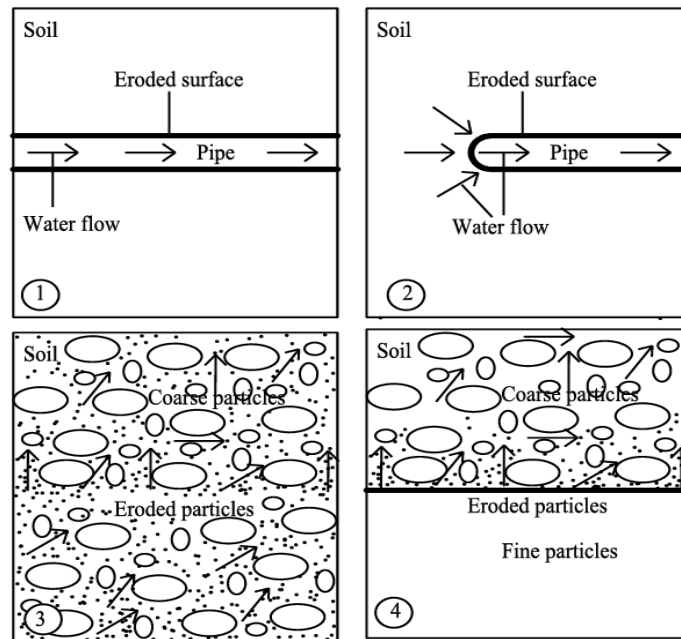


Fig. 2.1. The mechanisms of internal erosion (1) Concentrated leaks, (2) Backward erosion, (3) Suffusion, and (4) Contact erosion (Fell and Fry, 2007).

Fell and Fry (2007) classified internal erosion into four main types as concentrated leak erosion, backward erosion, contact erosion, and suffusion (Fig. 2.1).

Concentrated leak erosion occurs when the water flowing through cracks or openings erodes the soil particles surrounding crack. Backward erosion occurs when hydraulic gradients at the toe or exit of an embankment exceeds the critical hydraulic gradient thus the hydraulic force detaching soil particles through small erosion channels at downstream face. Contact erosion occurs at the boundary of two different soil layers where the fine particles pass through the coarser layer. Suffusion occurs when the finer fraction of a soil is transported by the mechanical action of a seepage flow, leaving behind a coarse soil skeleton.

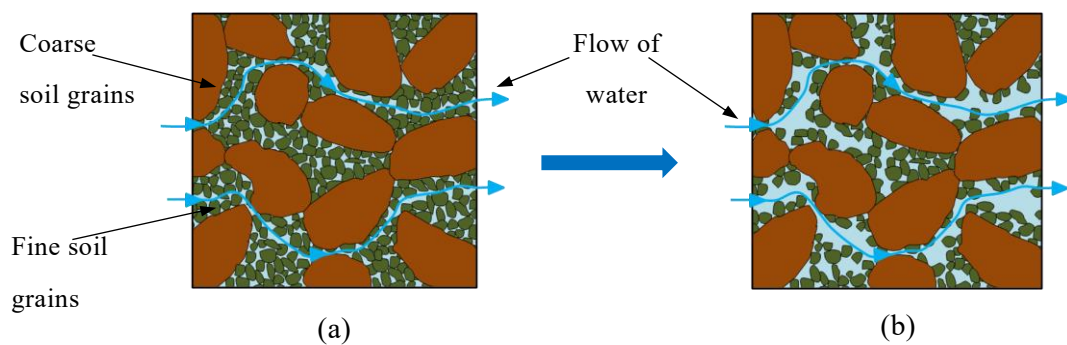


Fig. 2.2. Illustration of the suffusion phenomenon (a) before suffusion, and (b) after suffusion.

To put another way, suffusion is a type of internal erosion process in soil, occurs when finer fraction is transported through coarse soil particles due to seepage force generated by a difference in hydraulic head between two points (Fig. 2.2). Soil particles passing through 75 μm size sieve are referred to as finer fraction in this context. At some critical hydraulic gradient, the force of the seepage flow can be large enough to alter the micro structure of the primary framework and create larger pore constrictions leading to macro failure.

2.2 Earthen Constructions Susceptible to Suffusion

Artificially compacted grounds such as earth fills, slopes and cuts, roadway embankment etc. can get damaged by suffusion resulting in sinkholes or change in mechanical properties. Moreover, suffusion is one of the prominent cause of failure of hydraulic earth structures such as dams, levees and dikes. Statistical analysis showed that internal erosion is the governing failure mode of approximately half of the failures of embankment dams (Foster et al., 2000; Richards and Reddy, 2007).

The modes of failure prevalent for earthen constructions subjected to suffusion causing damage to lives and properties are-

- a) Foundation failure, slope failure due to reduction in shear strength,
- b) settlement of the structure, and
- c) formation of sinkholes due to loosening of the eroded soil.

2.3 Evolution of Studies on Suffusion Phenomena

2.3.1 Theoretical studies

Theoretical or analytical studies are useful because of their systematic approach and quantitative results to compute soil suffusion. Terzaghi and Peck (1948) first introduced the possibility of theoretical/analytical calculation of suffusion from their laboratory work, following which other researchers developed various theoretical methods to measure suffusion considering various suffusion related parameters (Kezdi, 1979; Kenney and Lau, 1985; Wan and Fell, 2008; VNIIG, 1976; VODGEO, 1982). In the recent years, researchers are trying to develop theoretical

models to simulate mechanical behavior of suffusion induced soils (Liang et al., 2013; Chen et al., 2015; Bianchi et al., 2018; Huang et al., 2022).

2.3.2 Experimental studies

In 1930s, with the development of the study on the mechanics of soil erosion and sediment transport (Rouse, 1939; Shields, 1936) which are key aspects of suffusion, topic of soil suffusion came to light. Terzaghi (1943) suggested that laboratory tests can be conducted to evaluate the exposure of soils to suffusion which was of significant importance to the field of geotechnical engineering. Since early 80s, the advancement on the experimental investigation on the suffusion continued to develop where rigid-walled and flexible walled permeameter tests were conducted to observe erosion to calculate the local variations in density under one-dimensional seepage flow condition moving either upward or downward (Sherard et al., 1984; Kenney et al., 1985, 1986; Keney and Lau, 1986; Lafleur et al., 1989; Tanaka and Toyokuni, 1991; Skempton and Brogan, 1994; Garner and Sobkowiz, 2002; Wan and fell, 2008). However, these tests were not able to replicate the effective stress in soil deposits at a certain depth below the ground surface and incapable to examine the changes in stiffness and strength of eroded soils which introduced the suffusion experiments under drained uniaxial compression and undrained triaxial compression. Moffat and Fannin (2006) studied internal instability in a gap graded soil with the help of a modified permeable cell with the provision of applying axial load during the erosion process. Further study by Moffat and Fenning (2011) using the same apparatus suggested that critical hydraulic gradient for internal erosion in soil is dependent upon the effective stress.

Attempts to study the suffusion characteristics of soil under confining stress with the help of modified triaxial apparatus were started from the beginning of 21st century (Richards and Reddy, 2008; 2009; and 2012; Bendahmane et al., 2008; Marot et al., 2009; 2012; and 2016; Nguyen et al., 2012; Sibille et al., 2015).

Chang and Zhang (2011) developed a stress-controlled apparatus (Fig. 2.3) to investigate the development of suffusion under complex stress state on cohesionless specimens (a mixture of sand and granite). They studied the influence of internal erosion on 100 mm diameter and 100 mm high gap graded soil specimens on soil stress-strain behavior under 50 kPa effective stress at a hydraulic gradient of 8. They observed that more fine particles were eroded from top layer as compared to the bottom layer and the original dilative stress-strain behavior changed to contractive with decrease in peak stress.

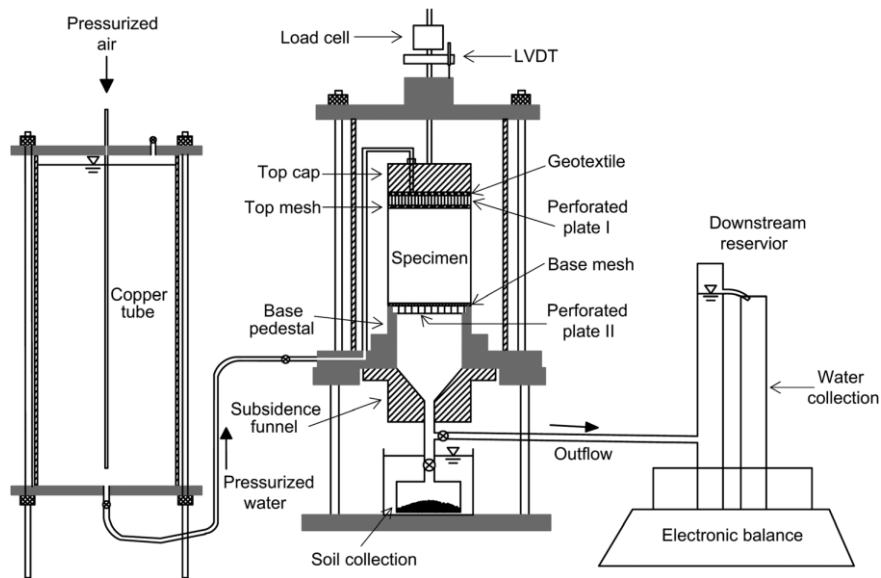


Fig 2.3. Schematic of the testing apparatus used by Chang and Zhang (2011).

Xiao and Shwiyhat (2012) studied the effects of suffusion on the physical and geo-mechanical properties of soil using a modified triaxial apparatus (Fig. 2.4). The

soil under this study was poorly graded river sands (cohesionless) mixed with 15% to 18% fine content. The specimens were 51 mm diameter and 102 mm height with 85% degree of compaction under an effective stress of 34.5 kPa (5 psi) at a hydraulic gradient of 20.8. The authors commented that suffusion causes reduction in permeability of the soil and influenced the mechanical behavior of soil depending upon the fine content.



Fig 2.4. Suffusion experimental set up used by Xiao and Shwiyhat (2012).

Ke and Takahashi (2014) modified a triaxial apparatus to conduct all triaxial test phases to an eroded specimen, including saturation, consolidation, erosion and shearing in a triaxial cell; with a new system to collect the discharged water and the eroded particles without loss of saturation (Figure 2.5). Their research focused on the drained and undrained behavior of a cohesionless gap-graded soil 70 mm diameter and 150 mm high specimens; with fine contents of 35%, 25% and 15% and 50 kPa,

100 kPa and 200 kPa effective stresses under monotonic and cyclic loadings pre- and post-erosion. They observed that the post-erosion drained shear strength decreased, while the eroded specimen reached a higher undrained peak shear strength at small strains.

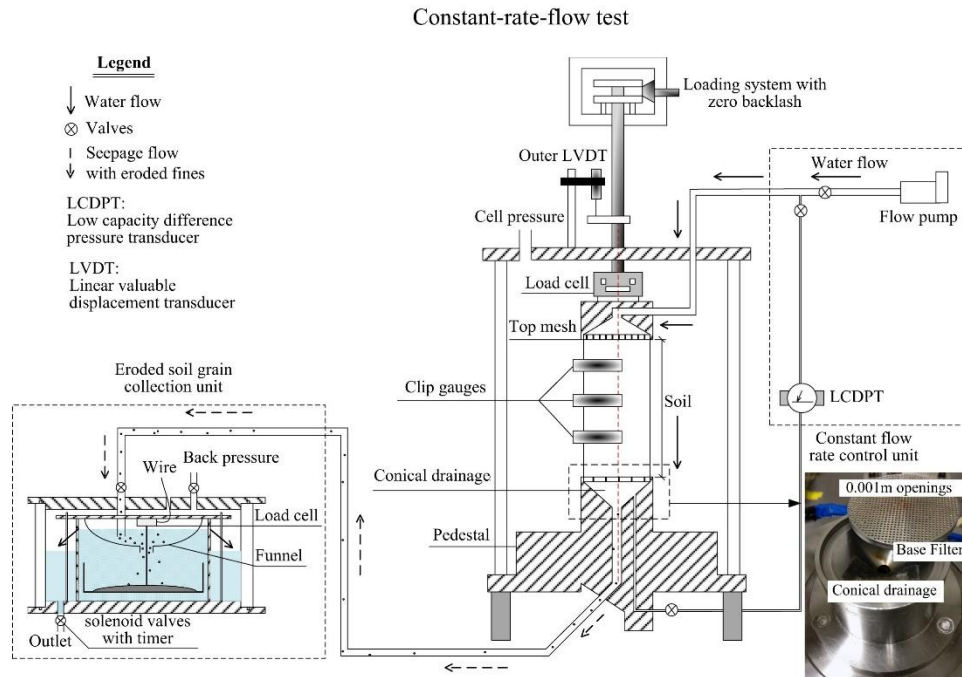


Fig 2.5. Schematic diagram of apparatus assembly developed by Ke and Takahashi (2014).

Ouyang and Takahashi (2015) used the same apparatus developed by Ke and Takahashi (2014) to study the influence of fine content on the fabric of soils subjected to internal erosion. All the cohesionless soil specimens were prepared at 30% relative densities with 15%, 25% and 35% fine contents. Seepage tests were conducted at flow rate of 5.2×10^{-6} m/s under effective stress of 50 kPa. They observed that the specimens with higher fine contents lost their fine particles more during the erosion phase. Additionally, eroded specimens showed drop in shear strength for low fine

contents and larger undrained secant stiffness compared to the non-eroded specimens at a relatively small axial strain level.

Mehdizadeh et al. (2017a; 2017b; and 2018) studied the post-erosion mechanical and geometric behavior of gap-graded cohesionless soil, with a modified triaxial apparatus developed independently (Fig. 2.6). They conducted suffusion at 92 mm/min and 52 mm/min inflow velocities in 50 mm diameter and 115 mm height gap graded soil specimens subjected to 150 kPa effective stress and carried out undrained shear test under monotonic and cyclic loadings thereafter. They suggested that eroded specimens displayed increase in shear strength and higher resistance to liquefaction during cyclic loadings.

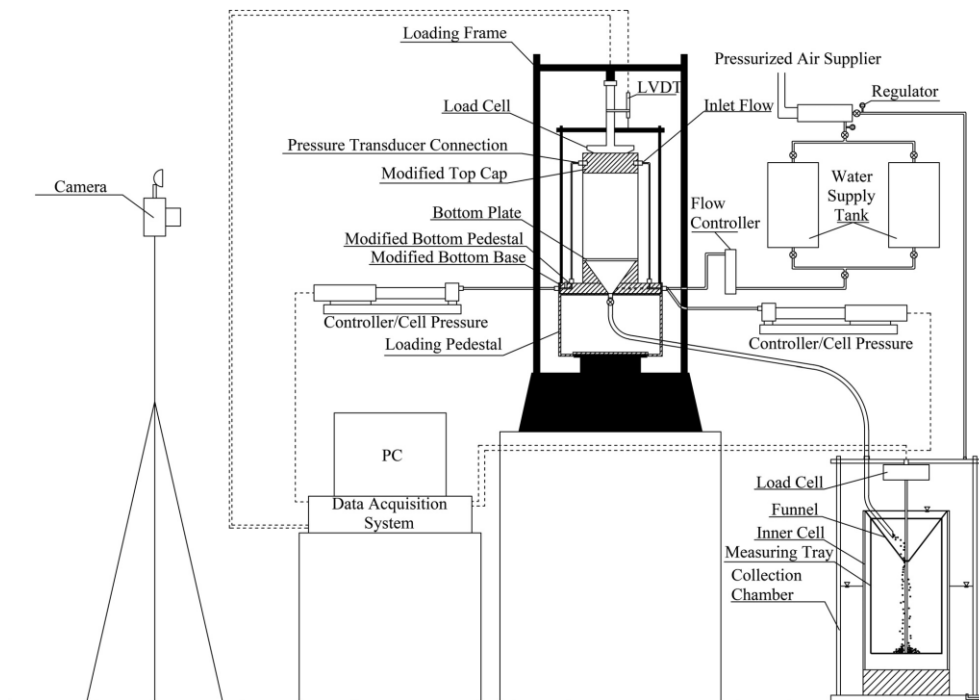


Fig. 2.6. Schematic of testing apparatus developed by Mehdizadeh et al. (2017a).

Sato and Kuwano (2018) studied the effect of small degree of suffusion due to downward seepage flow on stiffness and deformation of well graded natural sand

containing plastic clay particles using the testing apparatus shown in Fig. 2.7. They established that there was decrease in stiffness which was occurred due to increase in void ratio and reconstitution of the soil packing after suffusion.

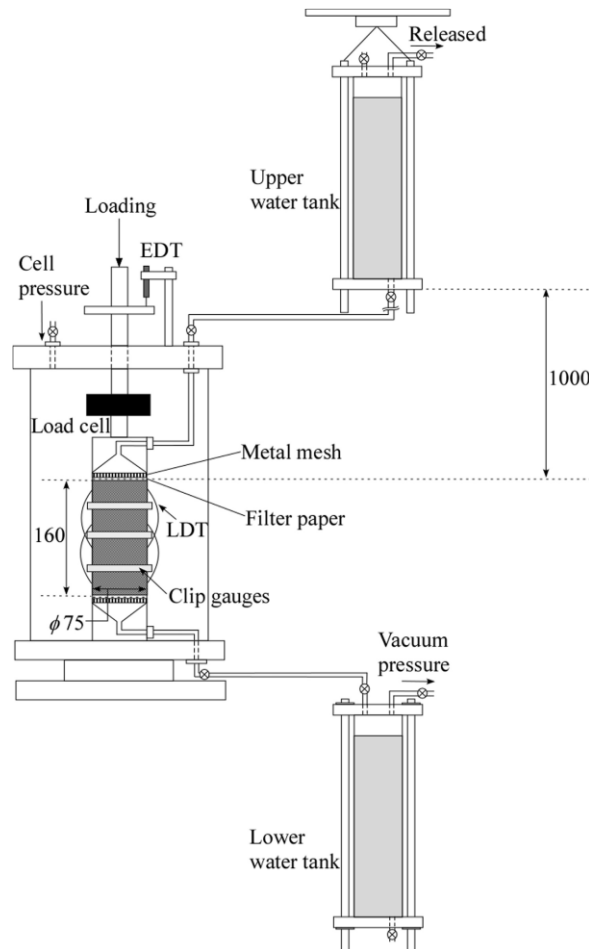


Fig. 2.7. Testing apparatus for simulating erosion under triaxial compression by Sato and Kuwano (2018).

Kuwano et al. (2021) studied the post-suffusion volume change in gap graded silty sand (Silica) specimens with 20% cohesive (DL clay) fine content in a triaxial apparatus modified for the provision of suffusion originally developed by Sato and Kuwano (2018) and later modified by the authors (Fig. 2.8). The size of the specimen was 75 mm diameter and 150 mm height. They applied a flow rate of 750 ml/min and

erosion was continued upto effluent volume of 5 L was collected. They commented that the erosion process varied depending on the initial density of the soil and higher density specimens showed relatively lower rates of erosion, degree of erosion, and compressive straining during erosion as compared with loose specimens.

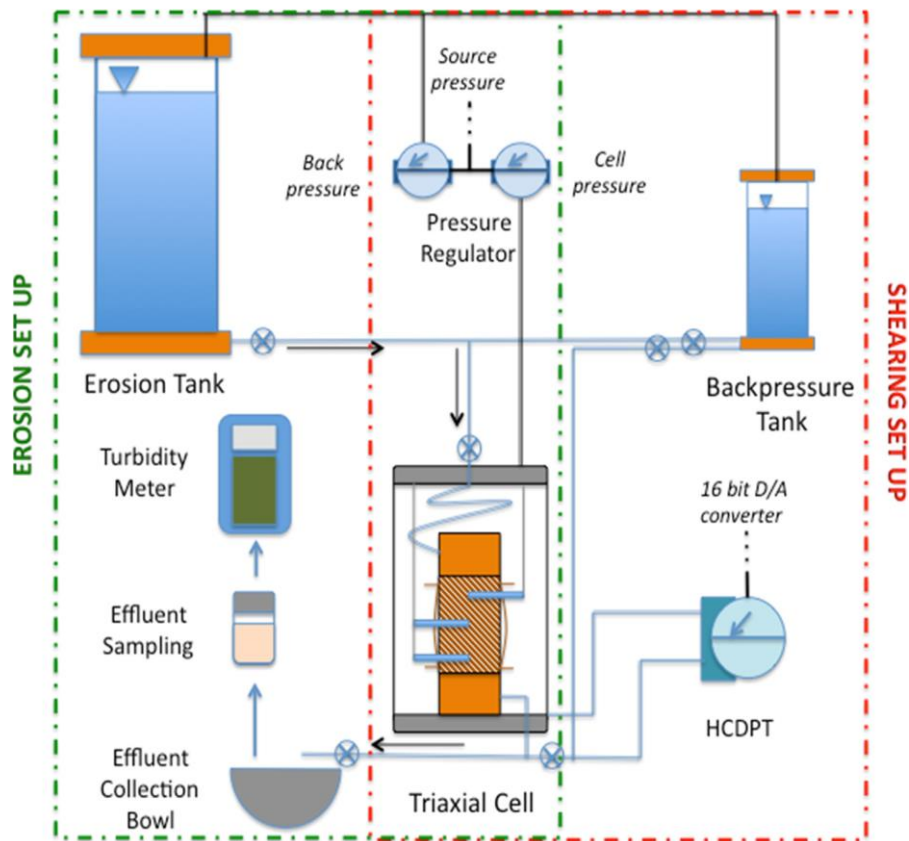


Fig. 2.8. Schematic diagram of the triaxial test set up developed by Kuwano et al. (2021).

Chitravel et al. (2021) explored the tendency of internal erosion in pumice sand and evaluated the effect of internal erosion on the stiffness degradation and monotonic response by conducting undrained and drained triaxial tests on 75 mm diameter and 150 mm high specimens with varying relative densities of 30%, 50% and 80%, i.e., loose packing at hydraulic gradient 50 using the apparatus developed by Kuwano et

al (2021). They concluded that internal erosion influences positively on the dilatancy response (observed indirectly from stress paths) of relatively dense specimens, as well as brittleness and peak strength. Furthermore, they inferred from the drained triaxial tests that elastic properties such as maximum shear modulus was increased and elastic yield surface found to be expanded for eroded soils.

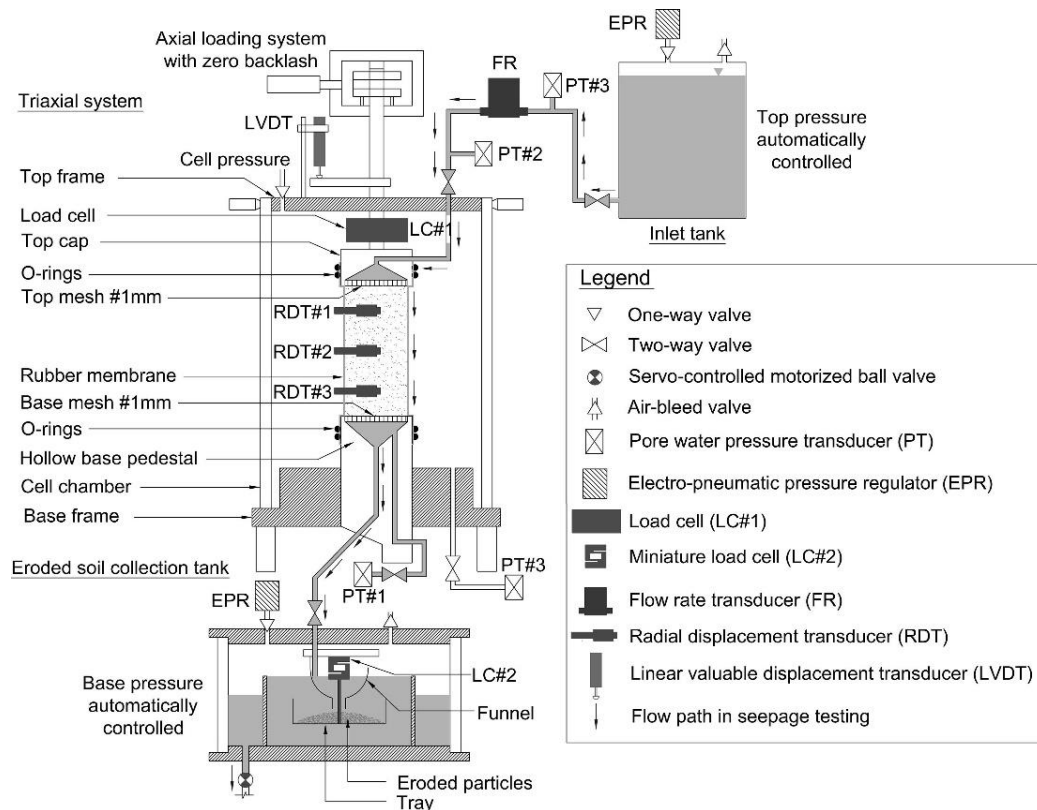


Fig. 2.9. Modified triaxial testing apparatus used by Prasomsri and Takahashi (2020).

Prasomsri and Takahashi (2020; and 2021) observed the changes in monotonic and cyclic loading behavior of eroded gap graded mixture of silica sand (cohesionless soil) using a modified triaxial erosion apparatus originally developed by Ke and Takahashi (2014) and later modified by the authors (Fig. 2.9). The 150 mm height and 75 mm diameter specimens were prepared with 15%, 25% and 35% fines content and were subjected to 50 kPa effective stress during erosion and shear. They observed

reduction in shear strength under monotonic compression and decrease in liquefaction resistance under cyclic loading.

2.3.3 Numerical studies

Conducting laboratory experiments is problematic at times since it may not be possible to consider all the factors affecting suffusion. Moreover, most of the experiments are conducted on gap graded soils, therefore, the results may not be fully understood or applicable to other soils (Wan and Fell, 2008; and 2014; To et al., 2018). As a solution to this problem and to study the effect of complex micro-scale factor such as stress and material structure on suffusion with theoretical studies and laboratory experiments; studying the initiation of suffusion and particle detachments have been tried through computational approach and mathematical models (Locke et al., 2001; Cividini and Gioda, 2004; Scheuermann et al., 2010; Sari et al., 2011; Maeda et al., 2012; Federico et al., 2013; Abdelhamid and Shamy, 2015). In this approach, the probability of movement of particles are determined by the particle scale basic mechanism and coupled discrete element method (DEM) and computational fluid dynamics (CFD) simulations. The mechanical behavior of suffusion induced soil has been studied through numerical solution by Chang and Hicher (2005), Hicher and Chang (2009), Hicher (2013), Wang and Li (2015), and Kawano et al. (2017).

2.4 Factors Controlling Suffusion Susceptibility of Soil

Initiation of suffusion mainly depends on three criteria namely- hydraulic criteria (hydraulic gradient, duration), geometric criteria (particle shape, and soil gradation) and transportable criteria (soil type, density, confining stress) as explained in Fig. 2.10 (Garner and Fannin, 2010; Fell and Fry, 2013; Marot et al., 2016).

Table 2.1. Recent studies on factors controlling suffusion.

Reference	Assessment type	Factors considered								Remarks
		Soil type	Particle shape	Grada-tion	Fine content	Soil density	Hydraulic gradient	Confining stress	Dura-tion	
Xiong et al., 2021	Numerical (CFD-DEM simulation)		✓	✓						Reduces suffusion rate with deviation from spherical shape.
Qian et al., 2021	Numerical (CFD-DEM simulation)		✓	✓						Angularity intensifies suffusion resistance.
Bendahmane et al., 2008	Experimental	✓			✓		✓	✓		Suffusion in clay specimens showed no volume change. Rate of erosion increases with decrease in fine content and confining stress. Hydraulic gradient has a positive influence on rate of erosion.
Zhou et al., 2023	Experimentl and numerical studies			✓						Higher aspect ratio results in higher resistance to erosion and severe loss of fines with greater bedding
Bi et al., 2021	Analytical and Numerical			✓			✓			A new method was proposed to evaluate suffusion in consideration to soil gradation and hydraulic

Table 2.1.(contd.) Recent studies on factors controlling suffusion.

Reference	Assessment type	Factors considered								Remarks
		Soil type	Particle shape	Grading	Fine content	Soil density	Hydraulic gradient	Confining stress	Durability	
Wan and Fell, 2008	Experimental	✓		✓						Minor differences in the shape of the particle size distribution affect the internal instability of soil.
VNIIG, 1976	Analytical			✓						Loose particles must be significantly smaller than constrictions to avoid clogging.
Kenney and Lau, 1985	Analytical			✓			✓			A hydraulic gradient >5 ensures beginning of suffusion for soil with $C_u < 20$.
Marot et al., 2012	Experimental	✓	✓							angularity of coarse fraction grains contributes to increase of suffusion resistance.
Zhang et al., 2023	Experimental			✓						Suffusion and contact erosion occur simultaneously when the retention ratio $(D_{35}/d_{85SA}) < 1$.
Kuwano et al., 2020	Experimental			✓		✓	✓			Density influences suffusion directly. Influence of hydraulic gradient on suffusion is dependent on the initial density of soil.

Table 2.1.(contd.) Recent studies on factors controlling suffusion.

Reference	Assessment type	Factors considered								Remarks
		Soil type	Particle shape	Grada-tion	Fine content	Soil density	Hydraulic gradient	Confining stress	Dura-tion	
Sibille et al., 2016	Experimental			✓			✓		✓	Initial fine content and hydraulic loading are key parameters for suffusion development.
Maroof et al., 2021	Experimental		✓	✓						Increase in the sphericity and roundness causes an increase in the loss of fine grains. Gap graded and poorly graded soils has higher suffusion susceptibility compared to well graded soil.
Bedja et al., 2022	Experimental					✓				Erosion is inversely proportional to density.
Prasomsri et al., 2021	Experimental				✓					soil with fines content less than 30% is vulnerable to suffusion at a relatively small hydraulic gradient.
Ke and Takahashi, 2014	Experimental				✓			✓		The larger effective confining pressure would lead to the less extent of suffusion. With larger initial fines content, more fines would be eroded away.

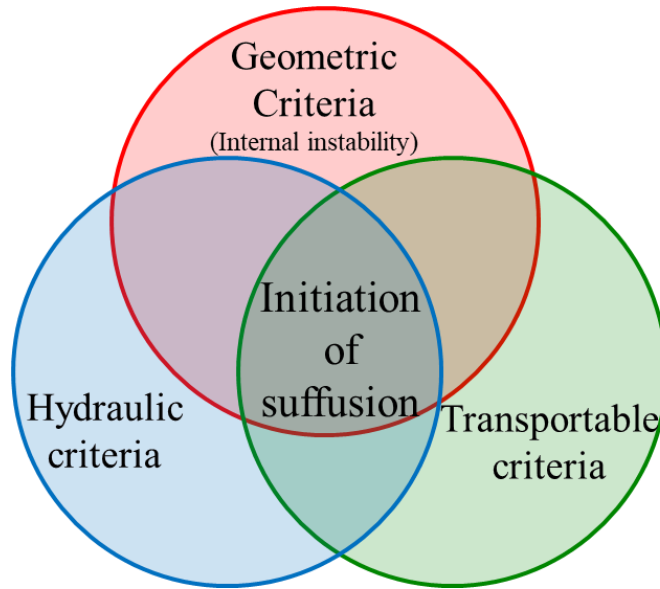


Fig. 2.10. Criteria for initiation of suffusion.

The influence of these factors such as soil type (cohesive and cohesion less), particle shape, soil density, hydraulic gradient, confining stress, and duration need special attention since they may vary during the transportation. Recent studies on the influence of these factors are described in Table 2.1 with references.

2.5 Assessment of Internal Stability in Soils Through Geometric Criteria

The internal instability or the geometric criterion refers to a soil's inclination towards detachment of the finer fraction from coarser fraction by seepage forces, i.e., soil gradation and fine contents. The geometric condition was further divided into three types which are geometric key size, geometric separation and geometric constriction. The dimensions of a soil's constrictions depend on the size, layout, and number of particles forming it (Kenney et al., 1985). It is therefore expedient to draw internal stability criteria by relating the dimensions of the constrictions of the soil to

the geometric characteristics of its particle distribution curve. Overall, granular noncohesive soils which are gap-graded or poorly graded with a flat tail (finer fraction) and a steep front (coarse fraction) are most prone to be internally unstable (Wan and Fell, 2008). Poorly graded soils like glacial tills are used in the construction of dam cores, and gap-graded soils may occur locally in earth filled structures due to mishandling of soil and poor construction practices (Xiao and Shwiyhat, 2012).

Various geometric criteria to assess the internal stability of soils proposed by different researchers from theoretical, experimental studies are explained in the following-

U.S. Army (1953) classified a soil as internally stable if its coefficient of uniformity,

$$C_u (= D_{60}/D_{10}) < 20 \quad \dots (2.1)$$

Instomina (1957) modified this criterion stating that the coefficient of uniformity, should be less than or equal to 20 for a soil to be internally stable. Mathematically,

$$C_u \leq 20 \quad \dots (2.2)$$

Based on Terzaghi's classical retention criteria for granular filters, Kezdi (1969) proposed a new method to investigate the soil internal stability by dividing it into fine and coarse fractions. He stated that a soil gradation becomes internally unstable when

$$\frac{D_{15}'}{d_{85}'} < 4 \quad \dots (2.3)$$

The values of D_{15}' and d_{85}' can be determined from the gradation curve at $0.85S_f$ and $(0.85S_f + 15)$ as shown in Fig. 2.11 where S_f is the value of percentage finer at the junction of coarse and fine fractions.

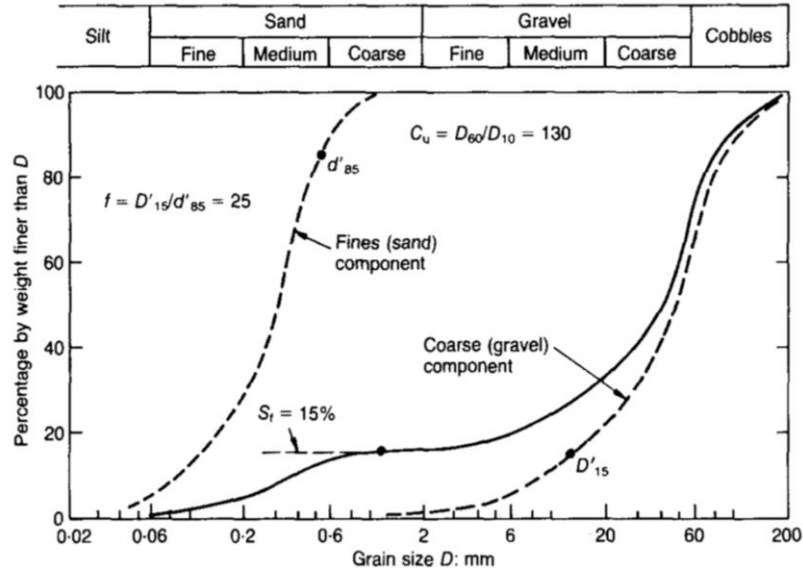


Fig. 2.11. Definitions of D'_{15} and d'_{85} by Kezdi (1969).

Sherad (1979) modified equation 2.3 to define an internally stable soil when

$$\frac{D'_{15}}{d'_{85}} < 5 \quad \dots (2.4)$$

Based on experimental results and theoretical analysis, Kenney and Lau (1985) proposed that a soil is internally stable when

$$H > 1.3F \quad \dots (2.5)$$

$$\text{or } H > F \quad \dots (2.6)$$

where F is the mass of fraction smaller than diameter, D of soil at any point on the grain size distribution curve, and H is the mass of fraction measured between D and $4D$ as shown in Fig. 2.12.

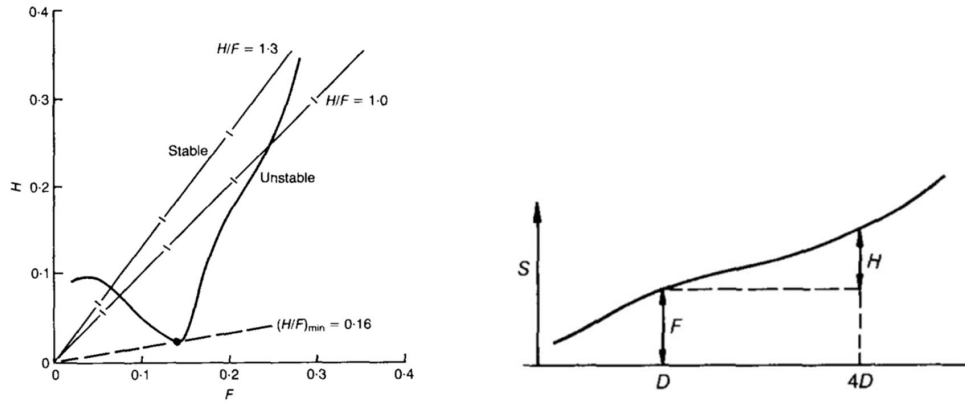


Fig. 2.12. Kenny and Lau's (1985) method with definitions of F and H .

Chapuis (1992) expressed Kezdi (1969), Sherard (1979) and Kenny and Lau's (1985) method mathematically as

$$S < 24.9\% \text{ per cycle } (0.15/\log 4) \text{ (Kezdi criterion)} \quad \dots (2.7)$$

$$S < 21.5\% \text{ per cycle } (0.15/\log 4) \text{ (Sherard criterion)} \quad \dots (2.8)$$

$$S < 1.66F \text{ for each particle size } \leq D_{FC} \text{ (FC } \leq 30\%) \quad \dots (2.9)$$

Where S is the secant slope of soil gradation curve (Fig. 2.13).

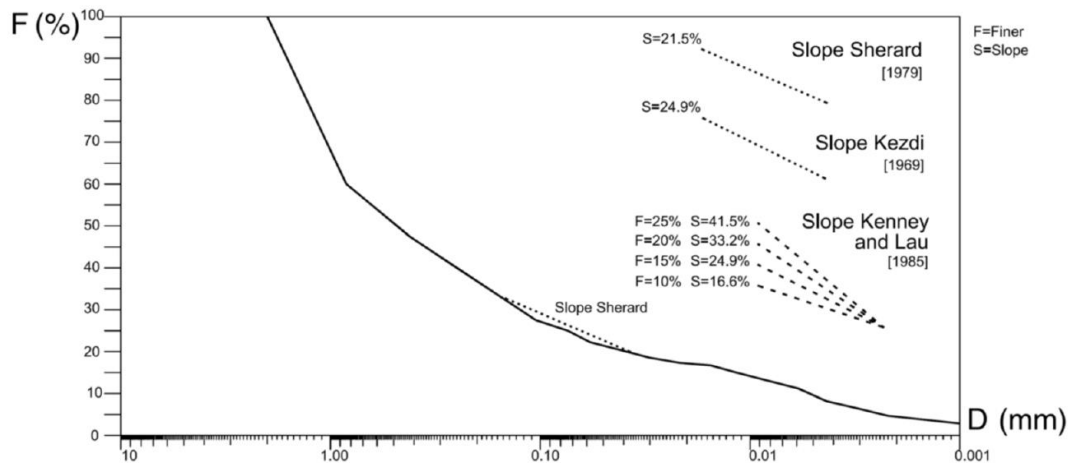


Fig. 2.13. Secant slope of the gradation curve as defined by Chapuis (1992).

Burenkova (1993) stated that a soil is stable if

$$0.76 \log(h'') < h' < 1.86 \log(h'') + 1 \quad \dots (2.10)$$

Where $h' = \frac{d_{90}}{d_{60}}$ is the slope of the coarse fraction and $h'' = \frac{d_{90}}{d_{15}}$ is the general slope.

According to Mao's Criteria (2005), depending upon fine content FC and porosity n , a soil is internally stable if:

$$4FC(1 - n) \geq 1 \quad \dots (2.11)$$

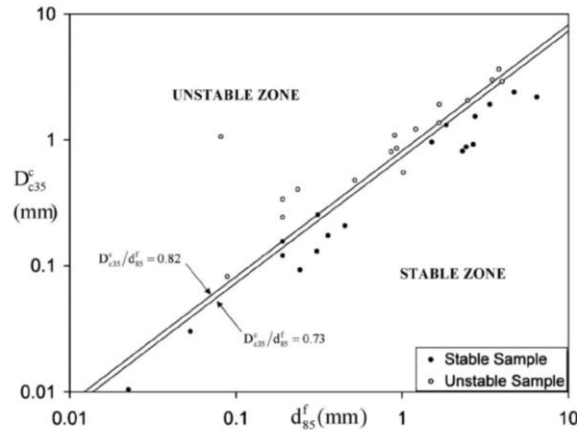


Fig. 2.14. Soil internal stability zones according to Indraratna et al. (2011).

Indraratna et al. (2011) proposed three internal stability zones for soils (Fig 2.14), based on the constriction size D_{ci} , as

Internally unstable zone if

$$\frac{D_{c35}}{d_{85}^f} > 0.82 \quad \dots (2.12)$$

Internally stable zone if

$$\frac{D_{c35}}{d_{85}^f} < 0.73 \quad \dots (2.13)$$

Transitional zone where further experimental investigation is required if

$$0.73 \leq \frac{D_{c35}}{d_{85}^f} \leq 0.82 \quad \dots (2.14)$$

where D_{c35} is the constriction size when 35% of constrictions are smaller, and can be calculated using Eq. 2.15 proposed by Locke et al. (2001).

$$D_{c35} = D_{c35(\text{densest})} + 0.35(1 - D_r)(D_{c35(\text{loosest})} - D_{c35(\text{densest})} \quad \dots (2.15)$$

Dallo et al. (2013) defined the stability zones (Fig. 2.15) based on Indraratna et al. (2011), which states a soil is

Internally unstable if

$$\frac{D_{c35}}{d_{85}^f} > 1.05 \quad \dots (2.16)$$

Internally stable if

$$\frac{D_{c35}}{d_{85}^f} < 1 \quad \dots (2.17)$$

Transitional zone where further experimental investigation is required if

$$1 < \frac{D_{c35}}{d_{85}^f} < 1.05 \quad \dots (2.18)$$

However, this method is not applicable if $d_{85}^f > 1.18$ mm.

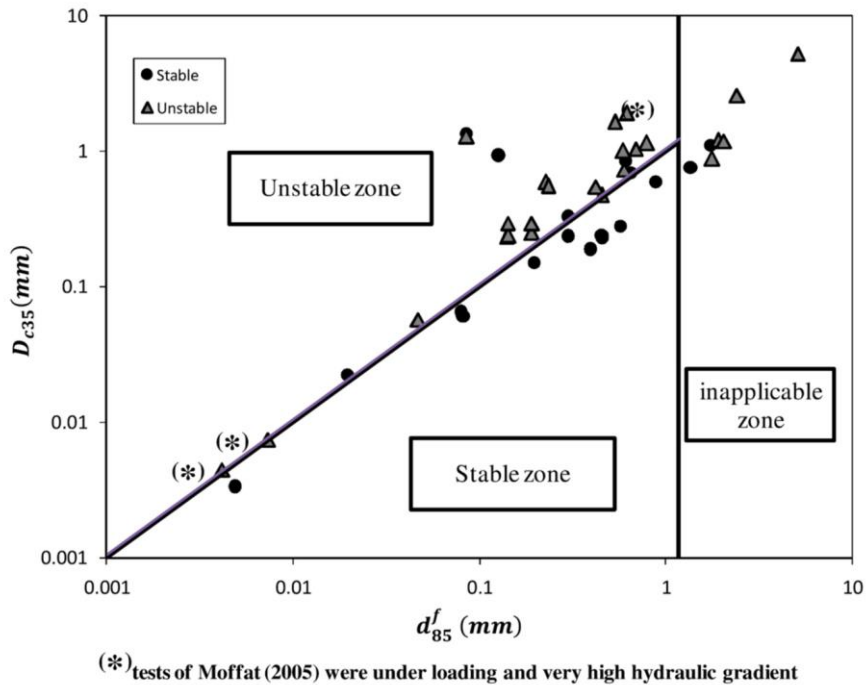


Fig. 2.15. Stability zones defined by Dallo et al. (2013).

2.6 Effect of Suffusion on Physical and Mechanical Properties of Soil

The observations of previous experimental studied relevant to this research is discussed below-

2.6.1 Volume change

Fannin and Slangen (2014) defined suffusion as a seepage-induced erosion of the finer particles from the skeleton of the coarser particles that are in contact; the finer particles are removed via the voids between the coarser particles by seepage flows, leaving the coarser skeleton intact.

Recent experimental studies on volume change due to suffusion are presented in Fig. 2.16. Xiao and Shwiyhat (2012) reported that the changes in volume due to fine migration were within the range of 0.26% to 0.75% and commented it as minor change. Bedja et al. (2022) studied the post-suffusion volume change in gap graded silty sand specimens with 20% fine content in a triaxial apparatus modified for the provision of suffusion. They observed that loose specimens developed higher volumetric strain than that of dense specimens on average. However, the volume change was limited to 0.682% maximum indicating non-destructive volume change. Nguyen et al. (2019) observed 1.07 % volume change on a gap graded soil specimen. Prasomsri and Takahashi (2021) observed volume change in a gap graded soil with 25% fine content during suffusion using a modified triaxial erosion apparatus. Mehdizadeh et al. (2017b) observed 0.59% and 0.61% volumetric strains undergoing suffusion at flow velocities of 52 mm/min and 208 mm/min respectively. Chitravel et al. (2021) observed volumetric strain minimum of 0.4% in dense specimens and 2.05% in loose specimens constituted of well graded volcanic soil subjected to loose compaction.

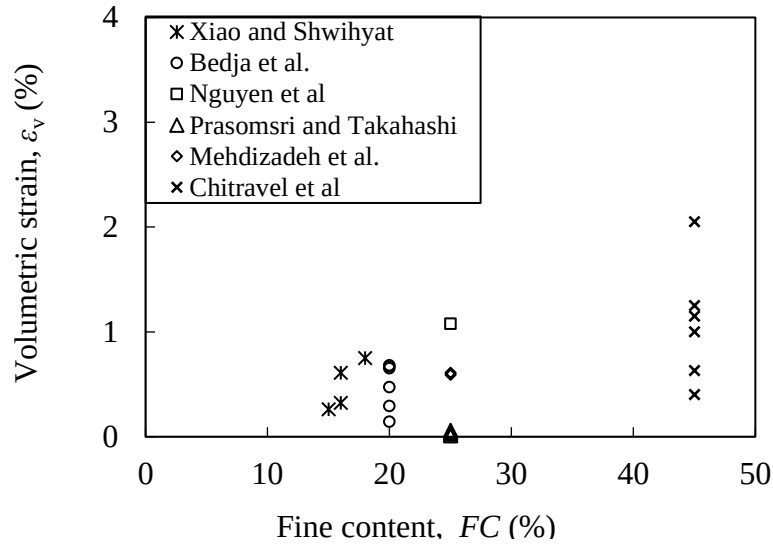


Fig. 2.16. Relationship between volumetric strain corresponding to fine content in recent experimental studies on suffusion.

From these studies it can be inferred that although the volumetric deformation of soil during suffusion is dependent upon various factors but not limited to fine content, soil gradation, hydraulic gradient, effective stress, specimen size etc.; these studies support the notion of Fannin and Slangen (2014) that suffusion causes no or negligible/non-destructive changes in volume due to the migration of finer particles.

2.6.2 Permeability

Figure 2.17 shows the relationship between hydraulic conductivity corresponding to relative density observed in recent literatures related to experimental studies on suffusion (Ke and Takahashi, 2014; Prasomsri and Takahashi, 2021; Chitravel et al., 2021). All the studies showed that for the same specimen, the post erosion hydraulic conductivity was higher than the post consolidation hydraulic conductivity.

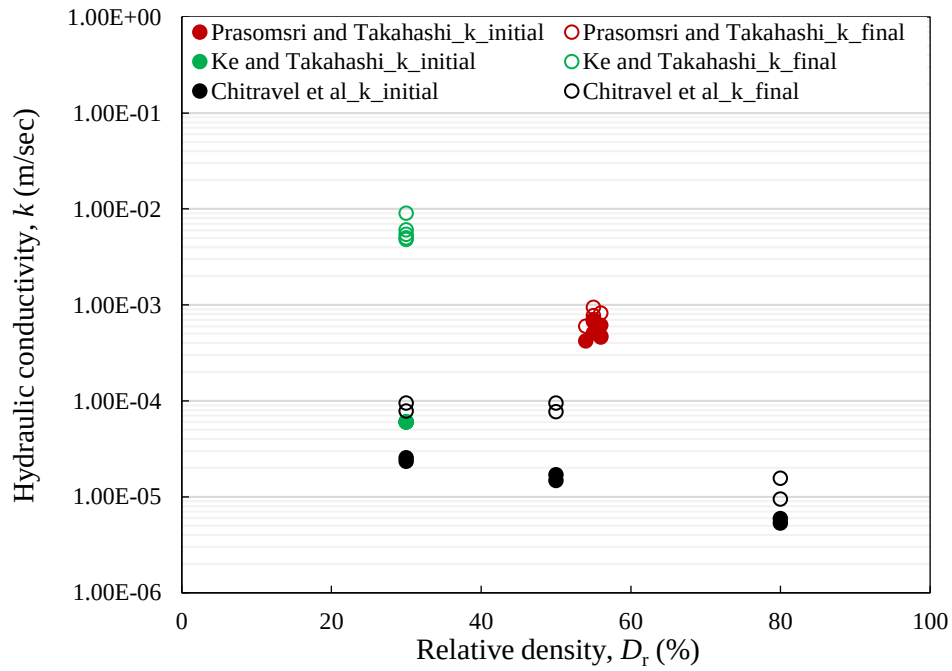


Fig. 2.17. Relationship between hydraulic conductivity corresponding to relative density in recent experimental studies on suffusion.

2.6.3 Shear strength

Observations on influence of suffusion on peak or maximum shear strength by experimental studies are presented in Fig. 2.18. The test conditions for these literatures are mentioned in section 2.3.2. Chang and Zhang (2011) reported a decrease in the peak shear strength of the eroded specimens. Xiao and Shwiyhat (2012) detected decrease in peak shear strength in well graded eroded specimens whereas decrease in shear strength for gap graded eroded specimens prepared with silty sand. Ke and Takahashi (2014) reported an increase in the maximum deviator stress of suffusion-induced cohesionless soil specimens subjected to high effective stress (200 kPa) with low fine content and reverse phenomena for high fine content. Opposite trend was observed by Oyung and Takahashi (2015) for soil specimens subject to low effective stress (50 kPa). Mehdizadeh et al. (2017) and Prasomsri and Takahashi (2021)

observed that suffusion reduces the peak shear strength in cohesionless gap graded specimens. Sato and Kuwano (2018) observed minor decrease in peak shear strength in high density specimens and increase in peak shear strength in low density specimens; whereas Kuwano et al. (2021) reported opposite pattern for a cohesive soil mixture (silica and DL clay). Chitravel et al. (2021) reported that suffusion increases the shear strength of well-graded volcanic soil at loose compaction condition with a variation in 50% relative density.

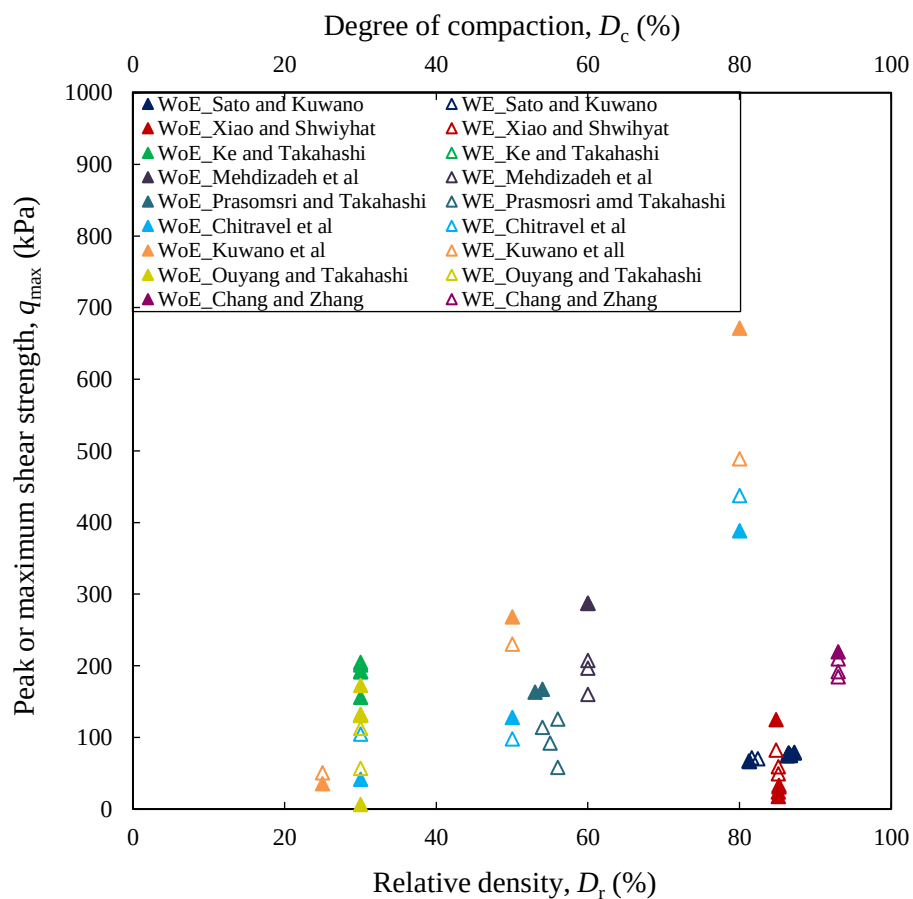


Fig. 2.18. Relationship between peak or maximum shear strength corresponding to relative density/degree of compaction in recent experimental studies on suffusion.

These experimental studies indicate that the change in peak or maximum shear strength of eroded specimens does not follow a specific trend, i.e., increase or decrease. Rather it is very complex in nature and dependent upon various factors such as soil type (cohesive or cohesionless), fine content, gradation, density, effective stress etc.

Chapter 3

TESTING MATERIAL

3.1 Sampling Site and Soil Collection

Pumice sand particular to Satozuka region in Kiyota ward, Sapporo City, Hokkaido, Japan, was used in this study. Encompassing a vast expanse of 1120 square kilometers, Sapporo boasts of a diverse array of geographical features, ranging from southern mountainous terrains to northern alluvial plains and peaty wetlands. The downtown Satozuka area is situated atop the alluvial fan of the Toyohira River, positioned on the western fringes of the Ishikari Lowland Belt and nestled between Hokkaido's primary mountain belts, primarily consists of hilly terrain.

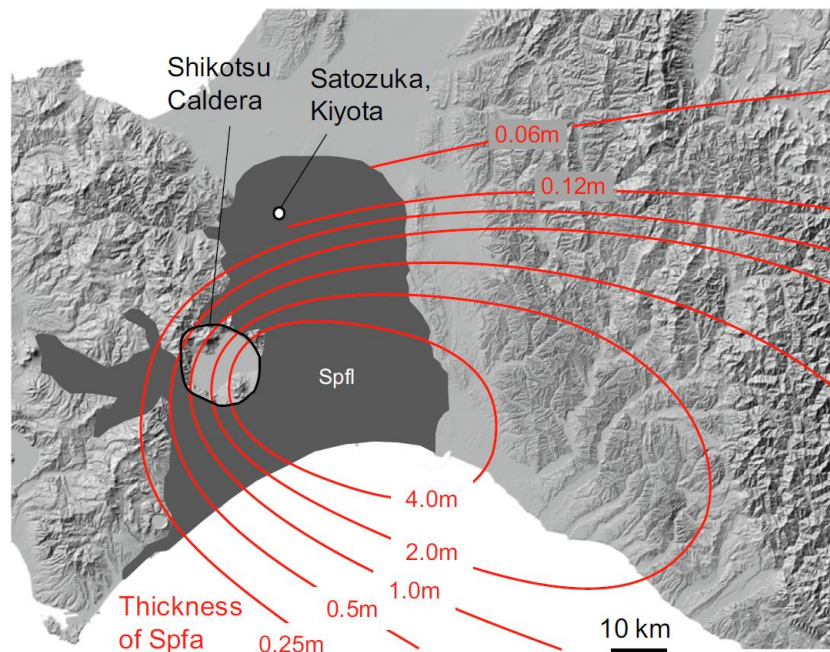


Fig. 3.1. Extent of Shikotsu Pumice deposits, Spfl and Spfa (Watabe and Nishimura, 2020).

Figure 3.1 gives a detailed depiction of the topographic characteristics of the sample collection area. This region lies in close proximity to the northern boundary of the ancient Mount Shikotsu, dating back 30,000 to 40,000 years. Figure 3.1 also provides an illustration of the estimated extent of Shikotsu Pumice-Flow (Spfl) and Shikotsu Pumice-Fall (Spfa), which represent predominant pumice deposit types in the Ishikari Lowland. Spfl is the deposition of volcanic lava flowing above the ground surface due to gravity and other physical obstruction whereas Spfa is the deposition of wind driven volcanic gas containing ash and lava due to gravity and air resistance (Matsumura, 2014). Due to their mode of formation, Spfl has well graded and Spfa has uniform grain size distribution. Satozuka region itself is situated on the northern periphery of the elevated pumice-flow hills, as discernible from the elevation map depicted in Fig. 3.2. This positioning underscores the geological intricacies of the area, shaped by the historical events surrounding Mount Shikotsu's volcanic activity.

The mineralogical constituents of Spfl include primarily plagioclase, hornblende, augite, hypersthene, and site-specific presence of quartz phenocrysts (Minato et al., 1972; Sato and Kagawa, 1956; Katsui, 1961; and 1963). Chemical composition of this sand include SiO_2 ($\approx 70\%$), Al_2O_3 ($\approx 15\%$), TiO_2 (0-0.5%), FeO and Fe_2O (3-4%), MnO (0-0.2%), MgO (0.5-1%), CaO (3%), Na_2O (3-4%), K_2O (1-2%), P_2O_5 (0-1%), and H_2O (2.5-3%) (Katsui, 1963).

This research consists of two experimental phases of material characterization and erosion triaxial test. Disturbed soil sample was collected from the Satozuka-Chuo Poplar Park (Fig. 1.1) and physical properties were determined with the help of laboratory tests. Soil sample were excavated at depths 0.9 m, 1.1 m, 1.5 m and 2.0 m depths (Fig. 3.3) and mixed to create a homogeneous sample (Fig. 3.4). The mixed

pumice sand collected appeared to be dark greyish brown in color with a shiny smooth texture.

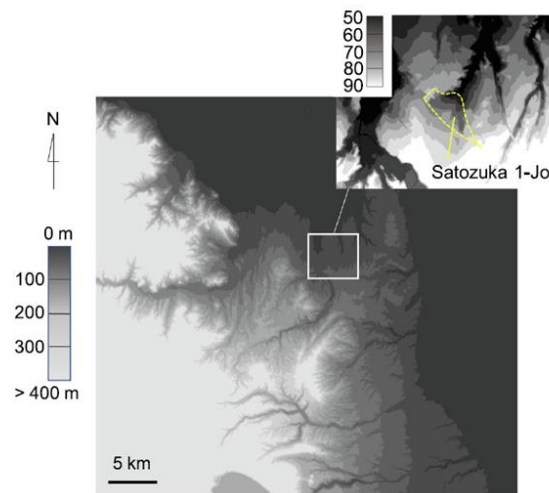


Fig. 3.2. Elevation (in m) and topographic features around Satozuka district (Watabe and Nishimura, 2020).

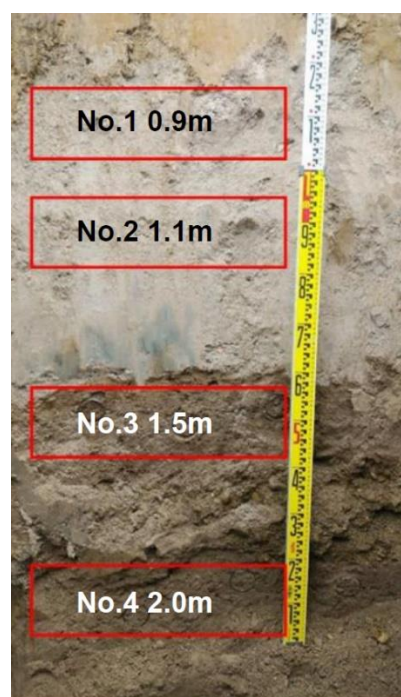


Fig. 3.3. Depths of sample collection in the boring pit.



Fig. 3.4. Pumice sand collected from Satozuka region used in this study at in-situ moisture content.

3.2 Material Characterization

Laboratory tests adopted for assessing the physical properties of the soil mixture is shown in Table 3.1. In addition, optical microscopic photo, SEM image and Computer Tomography (CT) images were also studied to evaluate the grain characteristics and soil microfabric. Soil particles passing through 9.5 mm size mesh were used for determination of physical properties and preparation of the specimens according to JIS A 1201.

Table 3.1. Laboratory tests adopted for assessing the physical properties of the soil mixture.

Sl No.	Laboratory test	Standard
1	Practice for preparing disturbed soil samples for soil testing	JIS A 1201
2	Soil particle density	JIS A 1202
3	Moisture content of soils	JIS A 1203
4	Grain size distribution analysis	JIS A 1204
5	Proctor compaction test	JIS A 1210
6	Maximum density and minimum density test	JIS A 1224
7	Specimen preparation for triaxial tests	JGS 0520-209
8	Microscopic image study	-
9	CT image analysis	-
10	Erosion susceptibility	-

3.2.1 Soil particle density

Even though all the experiments in this study are conducted exclusively on a single soil type, determination of soil particle density is essential, as it is used in the calculation of grain size, degree of compaction and degree of saturation. The soil particle density was obtained as 2.395 Mg/m^3 from the laboratory test according to JIS A 1202.

3.2.2 Grain size distribution analysis

Grain size distribution analysis provides an initial idea for engineering behavior of soils. It is also used for soil classification systems, such as Unified Soil Classification System (USCS). Grain size distribution is also essential for assessment of suffusion susceptibility of soil using geometric criteria. Figure 3.5 shows the grain size distribution of the pumice sand prepared using combination of sieve analysis and hydrometer analysis. The soil was found to be well graded with approximately 45% non-plastic fine content (>0.075 mm) in its natural state. The coefficient of uniformity, C_u and coefficient of curvature, C_c were obtained as 97 and 2.95, respectively, which indicates the presence of well graded soil particles with different size ranges. The soil is classified as “volcanic coarse-grained soil” with $FC < 50\%$ according to the classification by Japanese Geotechnical Society and as “silty sand (SM) of volcanic origin” according to the Unified Soil Classification System (USCS).

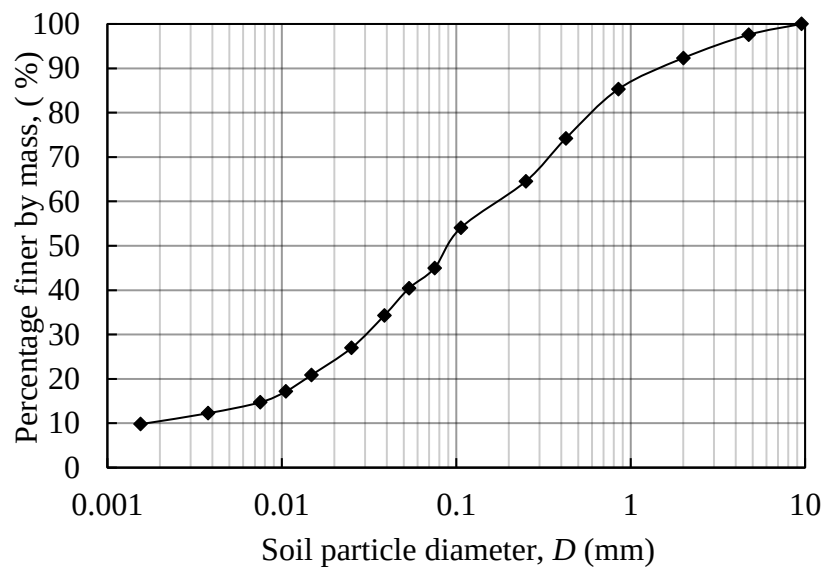


Fig. 3.5. Grain size distribution curve of pumice sand.

3.2.3 Proctor compaction test

Degree of compaction is one of the important components in assessing the state of the compactness for cohesionless soils. The moisture content - dry density relationship of the soil obtained from the proctor compaction test forms the basis for specifications and field compaction control (Sharma et al., 2016). The optimum moisture content and maximum dry density were observed as 42.7% and 1.068 Mg/m³, respectively, from standard proctor compaction test (JIS A 1210) as shown in Fig. 3.6.

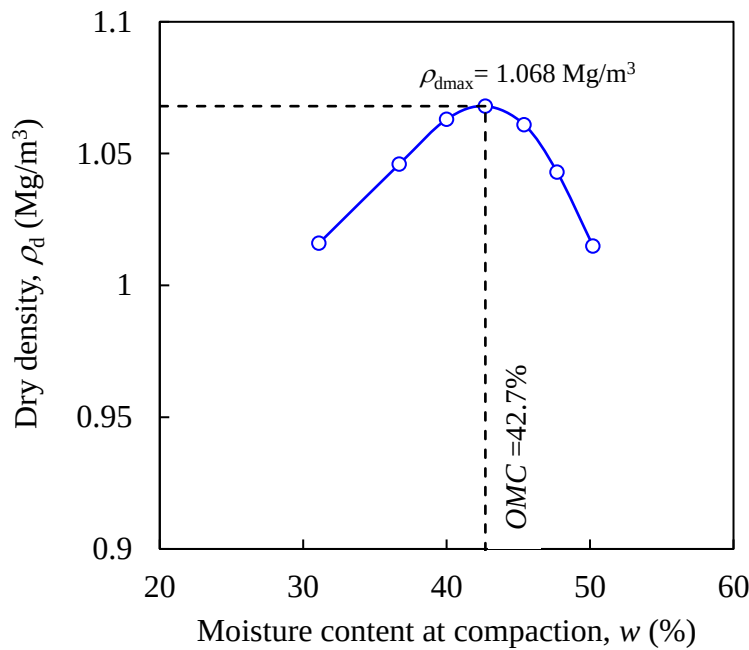


Fig. 3.6. Compaction curve of pumice sand.

3.2.4 Maximum and minimum densities

Relative density is related to the achievable maximum and minimum densities, which can only be measured in the laboratory. The maximum and minimum densities of the pumice sand used in this study were obtained as 0.988 Mg/m³ and 0.680 Mg/m³, respectively, from minimum and maximum densities test according to JIS A 1224.

The physical properties of pumice sand obtained from the initial site investigation immediately after the Hokkaido Eastern Iburi Earthquake by Watabe and Nishimura (2020) are included in Table 3.2 (represented as Series A). The physical properties of pumice sand used in this study collected from Satozuka-Chuo Poplar Park samples are summarized in Table 3.2 (represented as series B). The properties of Series A and Series B appear different most likely because of the heterogeneity of the soil material due stratification of pyroclastic flow deposit (Spfl) over different time span and the non-uniformity of the compacted ground.

Table 3.2. Physical properties of Satozuka pumice sand.

Physical property	Series A	Series B
Soil particle density, ρ_s (Mg/m ³)	2.391	2.395
Coefficient of uniformity, C_u	24	97
Coefficient of curvature, C_c	2.17	2.95
Maximum density, $\rho_{\max}$ (Mg/m ³)	1.051	0.988
Minimum density, $\rho_{\min}$ (Mg/m ³)	0.749	0.680
Optimum moisture content, OMC (%)	42.6	42.7
Maximum dry density, $\rho_{d\max}$ (Mg/m ³)	1.094	1.068

3.2.5 Microscopic image

The optical microscopic photo and scanning electron microscopic (SEM) image (Fig. 3.7) analyzed by Watabe and Nishimura (2020) revealed the presence of

glassy plate-like sand particles with high angularity and the aggregation of fine particles.

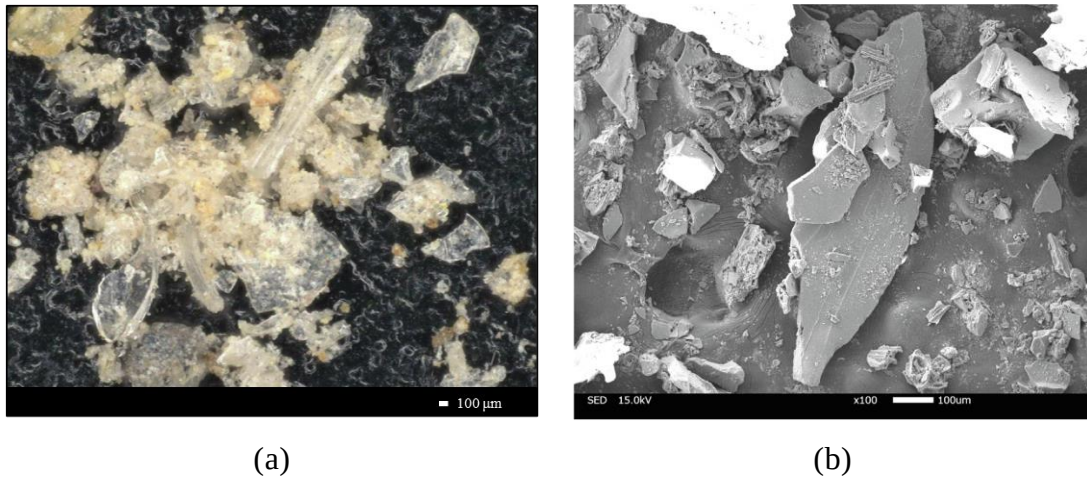


Fig. 3.7. (a) Optical microscopic photo, and (b) Scanning Electron Microscopic (SEM) image of Satozuka pumice sand (Watabe and Nishimura, 2020).

3.2.6 CT Image Analysis

Soil specimens of 10 mm diameter were prepared under different degrees of compaction at *OMC* and X-ray CT (Computer Tomography) scan was conducted to observe the variation in microfabric of soil with degree of compaction. Figure 3.8 shows the reconstructed cross sectional CT images of soil specimens at 75% and 100% degree of compaction at natural moisture content (47%). The images were reconstructed with white background to observe the interconnected voids appropriately. The gray scale in this CT image is white to black corresponding to high density to low density. This gray scale is opposite from normal CT images. Aggregation of fine particles was observed in the specimens. The loose specimen shows the formation of a macropore network (macro-porosity) due to the aggregation of fine particles.

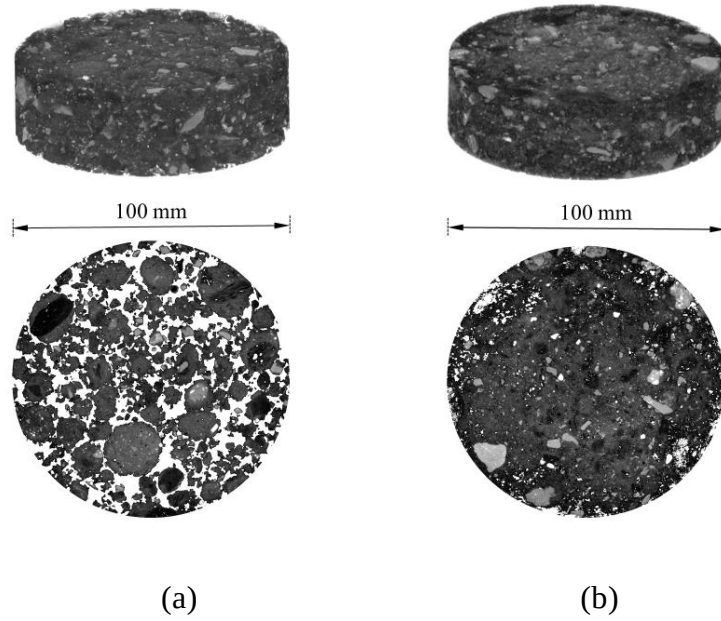


Fig. 3.8. CT images of pumice sand at (a) $D_c \approx 75\%$, and (b) $D_c \approx 100\%$.

3.3 Assessment of Internal Instability of Pumice Sand Based on Geometric Criteria

From the physical properties and the grain size distribution curves, the geometric properties of the pumice sand were determined as given in Table 3.3 and stability of the pumice sand is assessed in Table 3.4 using the suitable soil internal stability criteria stated in Chapter 2.

Table 3.3. Geometric properties of pumice sand.

Property	Value (mm)
D_{10}	0.0018
D_{15}	0.0081
D_{60}	0.1730
D_{90}	1.466

Table 3.4. Internal stability assessment of pumice sand.

Criteria	Evaluation	Inference
U.S. Army criteria (1953)	$C_u = \frac{D_{60}}{D_{10}} = 97 \not\leq 20$	Unstable
Instomina criteria (1957)	$C_u = 97 \not\leq 20$	Unstable
Kenney and Lau's method (1985)	For mass fraction smaller than $D = .03$ mm, $F = 29.73$ and $H = 32.08$ Hence, $\frac{H}{F} = 1.08 \not\geq 1.3$ or For mass fraction smaller than $D = .02$ mm, $F = 24.32$ and $H = 22.21$ Hence, $\frac{H}{F} = 0.913 \not\geq 1$	Unstable
Burenkova (1993)	$h' = \frac{D_{90}}{D_{60}} = 8.474$ $h'' = \frac{D_{90}}{D_{15}} = 180.98$ Now, $0.76 \log(h'') = 1.716$ $1.86 \log(h'') + 1 = 5.2$ Hence, h' is not within the prescribed limit i.e. $0.76 \log(h'') < h' \not\leq 1.86 \log(h'') + 1$	Unstable

The results indicate that the pumice sand under assessment is internally unstable against suffusion in spite of its well-graded nature.

Chapter 4

EXPERIMENTAL SETUP AND TESTING PROGRAM

4.1 Testing Equipment

The experimental setup is modelled after the suffusion apparatus by Sato and Kuwano (2018) and Kuwano et al. (2020) with several distinct instruments. It comprises a triaxial testing apparatus with a strain-controlled bottom loading frame, pressurized water supply system (erosion tank), and effluent collection container (Fig. 4.1).

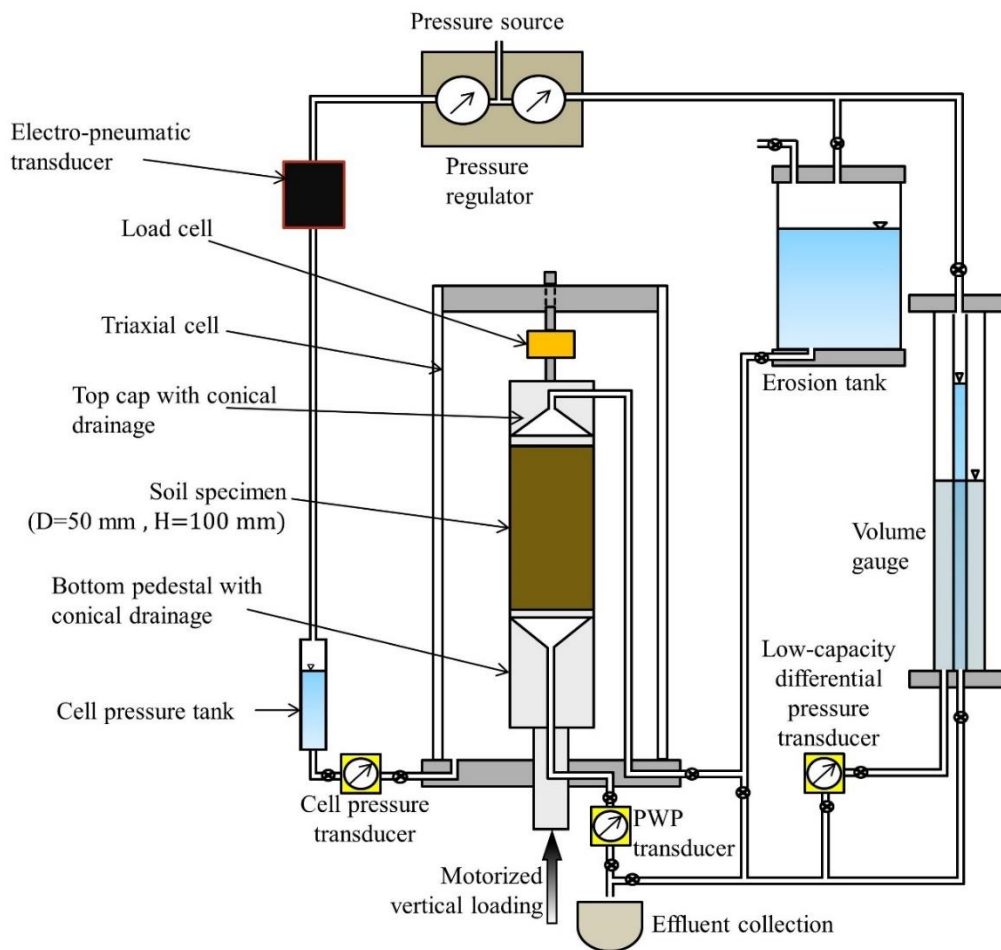


Fig. 4.1. Schematic of experimental setup.

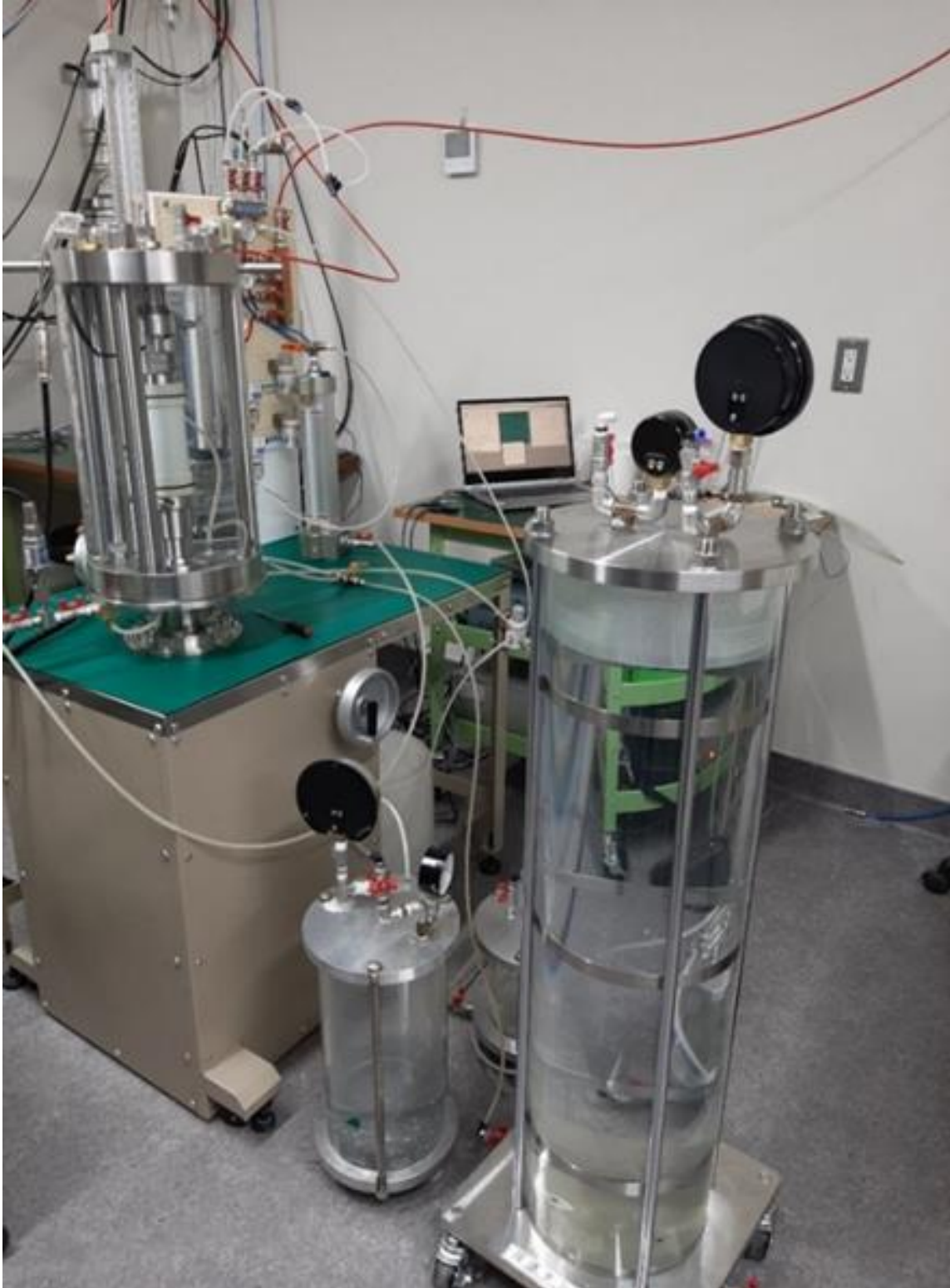


Fig. 4.2. Complete triaxial test setup with erosion tank.

The triaxial apparatus consists of a loading frame, triaxial acrylic cell, submergible load cell to measure the deviator load, two gauge-type pressure transducers to measure pore water pressure and cell pressure (Fig. 4.2). The change in volume of the specimen can be measured with the help of a low-capacity differential pressure transducer (LCDPT) which was connected to the soil specimen and a double tube burette or volume gauge. Four amplifiers were used for channels connected to transducers (load cell, pore water pressure transducer, cell pressure transducer and LCDPT). The vertical displacement of the specimen was measured in terms of very precise motor displacement, and the lateral displacement (radial strain) was calculated from the correlation between volume change and vertical displacement. The cell pressure during the test was maintained with the help of an electro-pneumatic (EP) transducer that converts a voltage input into proportional pressure output. Analogue to digital and digital to analogue (AD/DA) converter was used to make interaction between the testing apparatus and the computer program in terms of voltage. A computer program developed in the laboratory in Visual Basic programming language was used to operate the apparatus and obtain and record the test results.

The top cap and bottom pedestal were modified to make provision for suffusion. To facilitate the migration of fines, the main body of the top cap and bottom pedestal were tapered into a funnel shape at top leading to a hole in the center. A porous plate with 2 mm holes was placed above the funnel and was attached to the main body with screws. To allow the finer particles to erode, filters made of metal mesh and plastic mesh (1 mm openings each) were placed above the porous plate (Fig. 4.3 and Fig. 4.4). The top cap was connected to a de-aerated water tank (erosion tank) which helped in

the development of hydraulic gradient. An erosion tank was connected at top with a gauge type pressure meter capable of measuring both positive and negative pressure.

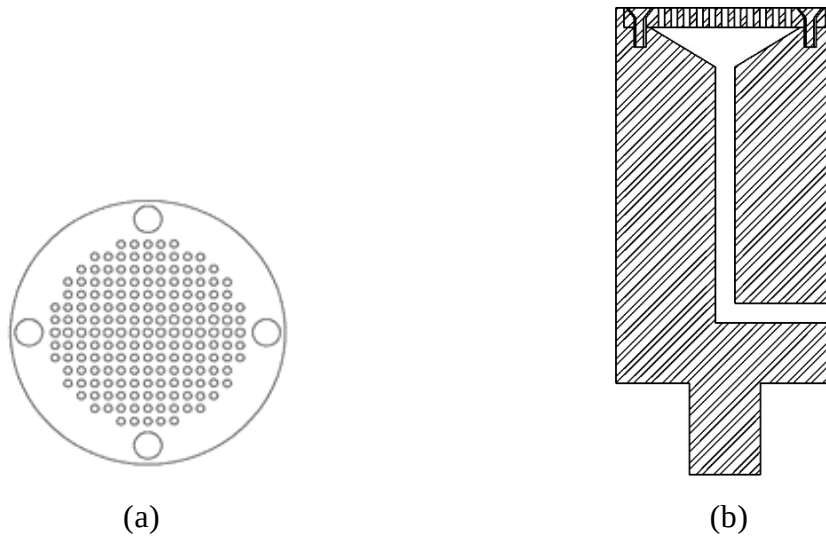


Fig. 4.3. (a) Top view of the porous plate, and (b) cross-section of the modified pedestal with porous plate attached.

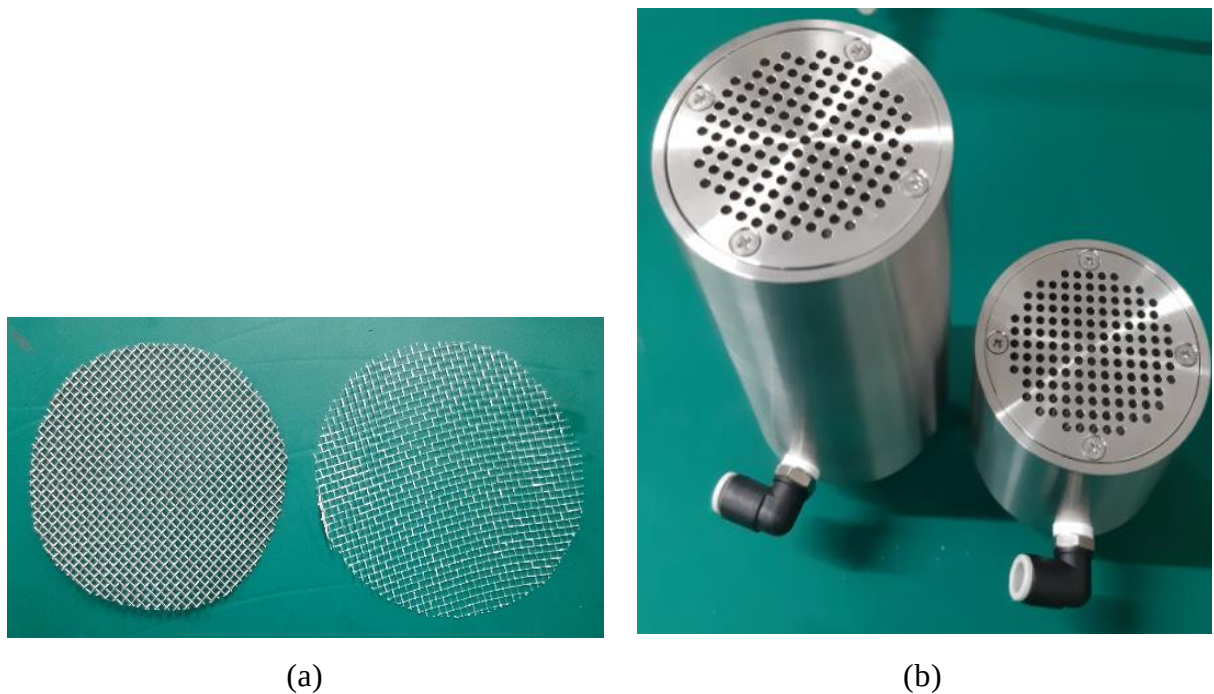


Fig. 4.4. (a) Metal mesh (left) and plastic mesh (right) used to control size of eroded particle, and (b) Modified pedestal and cap with porous plate attached.

The quantity of the eroded particles was measured in terms of turbidity using a portable digital nephelometric turbidimeter and converted to equivalent dry mass of soil sample using a calibration chart prepared in the laboratory.

The calibration curves of submersible load cells, pore water pressure transducer, cell pressure transducer, LCDPT, and turbidimeter are presented in Fig. 4.5, 4.6, 4.7, 4.8, and 4.9 respectively.

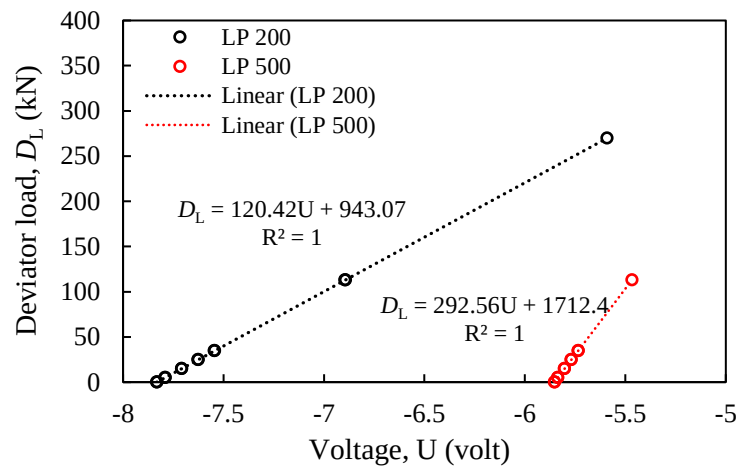


Fig. 4.5. Calibration curves of 2 kN (LP 200) and 5 kN (LP 500) capacity submersible load cells.

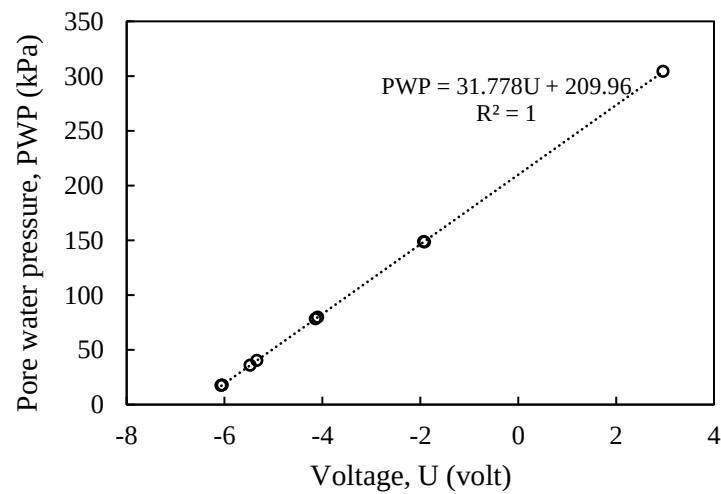


Fig. 4.6. Calibration curve of pore water pressure transducer.

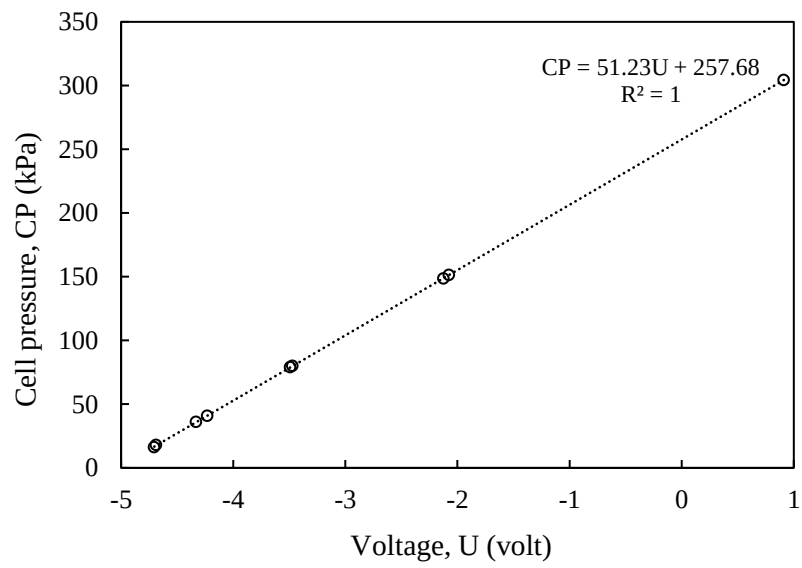


Fig. 4.7. Calibration curve of cell pressure transducer.

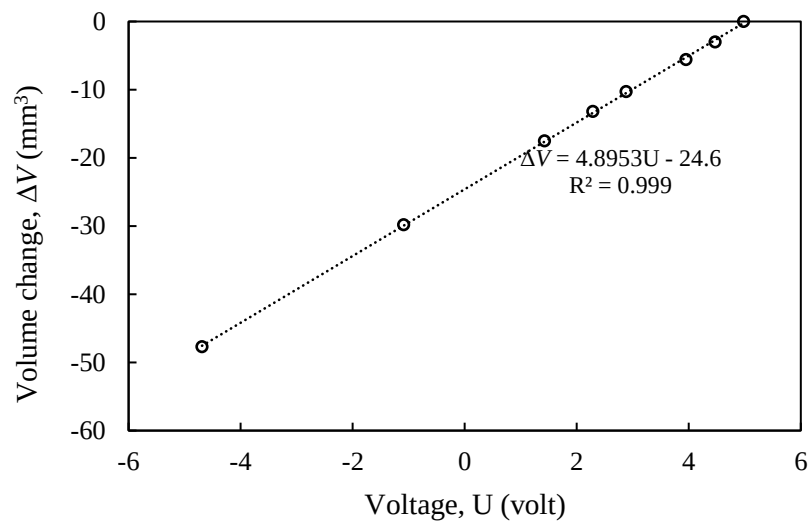


Fig. 4.8. Calibration curve of LCDPT.

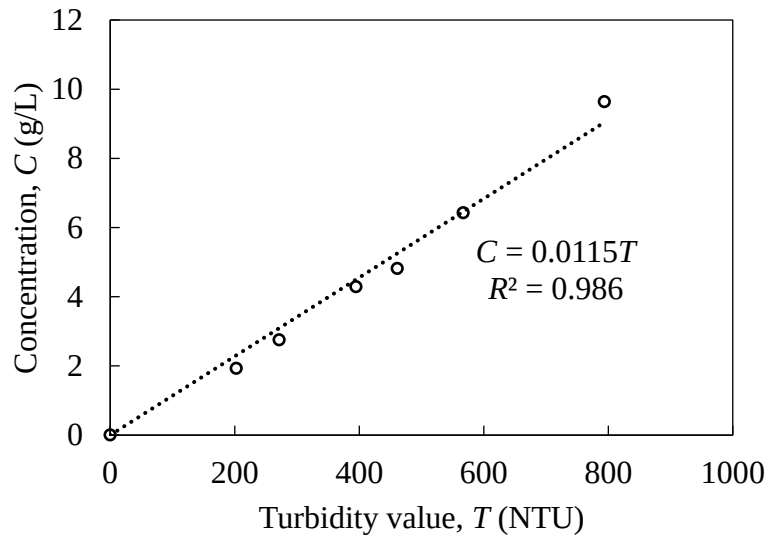


Fig. 4.9. Calibration curve of turbidimeter.

4.2 Testing Program and Testing Procedure

The experimental program consisted of five test stages, namely, specimen preparation, saturation, consolidation, suffusion by hydraulic gradient, and shearing.

4.2.1 Specimen preparation, saturation and consolidation

Table 4.1. Number of layers and number of blows given to specimens.

D_c	No. of layers	No. of blows
80%	5	16
90%	5	18
95%	8	18
100%	8	20

At first the soil was mixed with de-aerated water and kept in an air-tight container for 24 hours to attain the desired moisture content. Soil specimens, 100 mm in height and 50 mm in diameter, were prepared by compacting the soil in layers

(specified in Table 4.1) using a mold and a tamping rod at 80%, 90%, 95% and 100% degrees of compaction (D_c) of maximum dry density (ρ_{dmax}) on dry and wet sides of the optimum moisture content (OMC) of the compaction curve (Fig. 4.10).

According to Tamura and Sakai (1992), the degree of compaction is defined as the ratio between soil density measured in situ condition (ρ_d) and maximum dry density of soil (ρ_{dmax}) obtained from the standard proctor compaction test. Mathematically, D_c at a specific moisture content can be expressed as

$$D_c = \frac{\rho_d}{\rho_{dmax}} \quad \dots (4.1)$$

where ρ_d is the dry density of specimen at that moisture content and ρ_{dmax} is the maximum dry density of the soil at OMC . The test specimen identity with target and attained compaction condition are presented in Table 4.2.

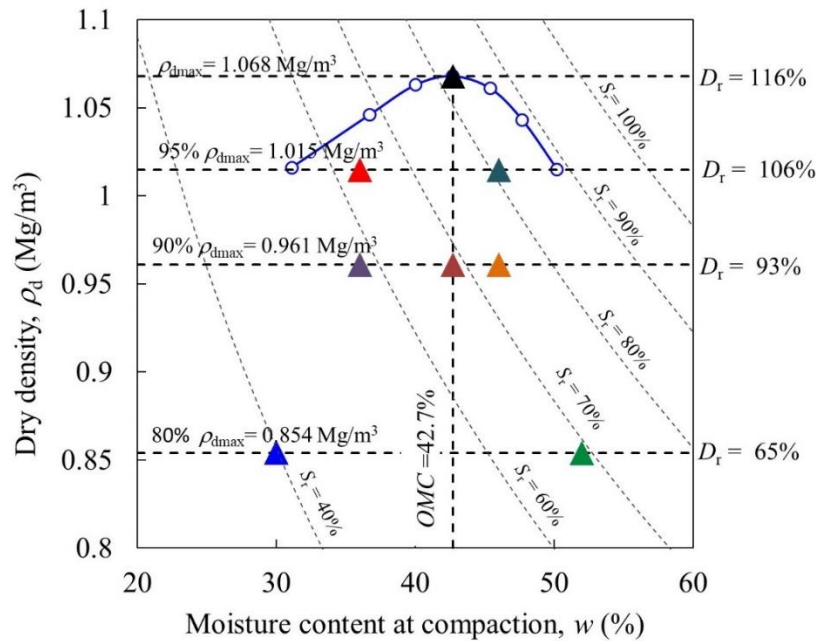


Fig. 4.10. Compaction curve with desired compaction conditions of specimens.

Table 4.2. Test specimen identities with target and attained degrees of compaction as well as moisture content during specimen preparation.

Specimen identity (Test case)	Target degree of compaction, $D_{c,t}$ (%)	Attained degree of compaction, D_c (%)	Moisture content, w (%)	Degree of saturation, S_r (%)	Suffusion occurred
WS_80Dc_30w	80	81.53	29.9	41	Yes
WoS_80Dc_30w	80	81.87	29.9	41	No
WS_80Dc_52w	80	81.67	52.4	72	Yes
WoS_80Dc_52w	80	81.69	50.8	70	No
WS_90Dc_36w	90	90.94	35.8	59	Yes
WoS_90Dc_36w	90	90.30	36.4	59	No
WS_90Dc_omc	90	91.46	43.2	71	Yes
WoS_90Dc_omc	90	91.38	42.9	71	No
WS_90Dc_46w	90	90.41	45.9	74	Yes
WoS_90Dc_46w	90	91.32	45.2	73	No
WS_95Dc_36w	95	94.65	36.0	63	Yes
WoS_95Dc_36w	95	94.20	36.7	63	No
WS_95Dc_46w	95	95.84	46.4	83	Yes
WoS_95Dc_46w	95	96.04	45.9	82	No
WS_100Dc_omc	100	99.71	42.5	82	Yes
WoS1_100Dc_omc	100	99.82	42.4	81	No
WoS2_100Dc_omc	100	100.78	41.5	81	No

* WS and WoS stand for “with suffusion” and “without suffusion” conditions, respectively.

Prepared specimens were mounted on the bottom pedestal with 1 mm opening meshes at top and bottom. The triaxial cell was assembled and filled with water till the load cell was submerged. The pressure inside triaxial cell and specimen were lowered to –70 kPa and –80 kPa, respectively until the air bubble keeps escaping from

the soil specimen. Then de-aired water was allowed to flow through the specimen from bottom to top, by keeping a pressure difference of 10 kPa between bottom and top of the specimen, till the volume of water about 1.5 times volume of specimen (equivalent to approximately 2.5 times pore volume) flows through the specimen, concurrently increasing the pressures inside and outside the specimen. The pressure in the specimen and triaxial cell were increased back to zero, triaxial cell was filled with water and the specimen was connected to volume gauge thereafter. The pressure inside the specimen was raised to 300 kPa and allowed to saturate till the volume gauge reading attains a constant value.

A pressure difference of 10 kPa was maintained between specimen and triaxial cell during the saturation by infiltration process. The specimen was considered to be saturated when the value of Skempton's pore pressure parameter (B -value) becomes more than 0.95 (Skempton, 1954). After achieving the desired B -value, the specimens were consolidated to an effective stress of 80 kPa, which corresponds to a representative overburden effective stress in the 9 m thick earth fill at Satozuka.

4.2.2 Suffusion test and shearing

Saturated and consolidated specimens were eroded under 80 kPa effective stress, which corresponds to the overburden effective stress. To facilitate the erosion process, the radial stresses outside the specimen (cell pressure) and inside the specimen (back pressure) were gradually reduced to 280 kPa and 200 kPa, respectively. Thereafter, the top of the specimen was connected to a water tank; and radial pressures outside the specimen and inside the specimen were gradually reduced to 130 kPa and 50 kPa; respectively. The hydraulic gradient along the specimen was introduced by connecting the top cap to a water tank under 50 kPa pressure and

keeping the bottom pedestal at the atmospheric pressure by connecting with effluent collection bowl via a 6 mm diameter transparent tube. For each specimen, water pressure in the top cap was increased by 50 kPa in steps and in each step, water carrying eroded soil was collected for 30 minutes. The cell pressure was also increased simultaneously to maintain 80 kPa effective stress throughout. The increase in cell pressure and water pressure in the top cap was continued after every 30 minutes till the pressure difference in the top cap and bottom pedestal reaches 200 kPa. This allowed suffusion to occur at a maximum hydraulic gradient of 200 considering a condition of 20 m water column height.

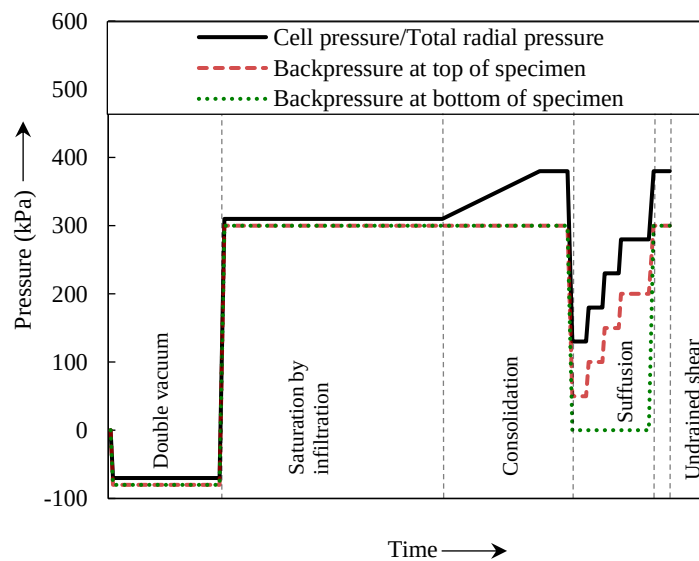


Fig. 4. 11. Changes in pressure inside and outside specimens with time.

Figure 4.11 shows the changes in pressure inside (back pressure) and outside (cell pressure/total radial pressure) the specimen with time. After the completion of the suffusion tests, the back pressure on the specimen was increased to 300 kPa before the shearing stage to maintain identical test conditions for all specimens (both WS-

series and WoS-series) during shearing.

While a hydraulic gradient of 200 may seem too large, it can still be achieved under practical conditions, such as the presence of a small crack or opening on the surface of a drainage pipe leading to leakage. For such conditions, the equipotential lines will be concentrated around the leakage point where the pore pressure distribution becomes equivalent to the hydrostatic pressure in the fill ground, and the water will be directed to flow through a thin layer wrapping around the drainage pipe located at the bottom of the compacted earth fill, as shown in Fig. 4.12. Under such conditions, this phenomenon may lead to a high hydraulic gradient of up to 200, although it is highly unlikely to exceed this value.

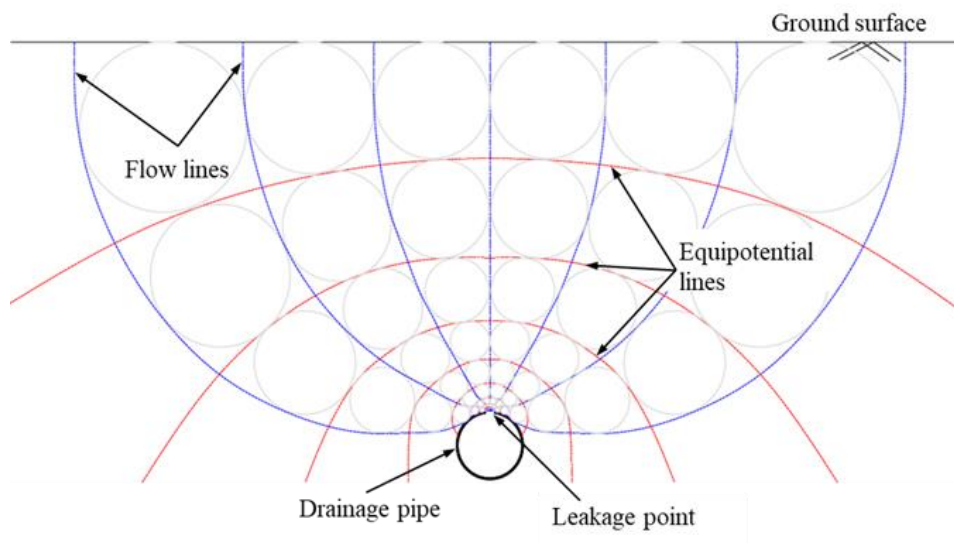


Fig. 4.12. Flow network for a leakage point on a drainage pipe.

The effluent containing eroded fines was collected in a water tank (during saturation) or an effluent collection bowl (during suffusion tests) and each volume was thereafter stirred separately to make a suspension. The quantity of eroded fines present in the effluent was measured in terms of turbidity using a portable digital

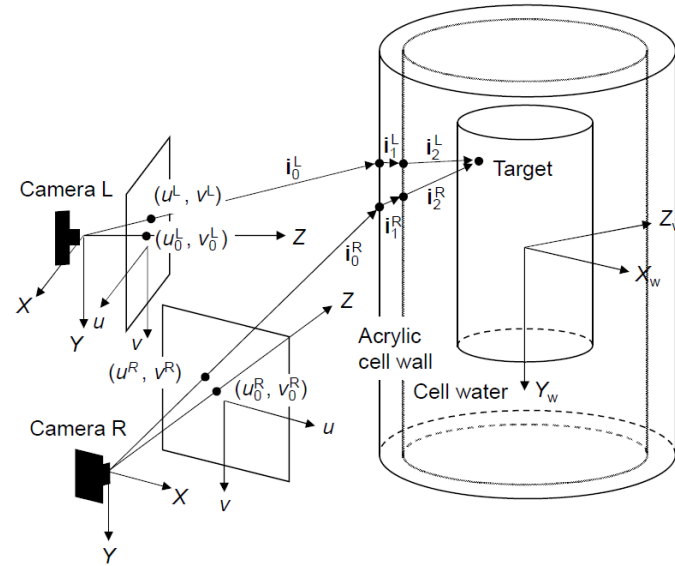
nephelometric turbidimeter (Fig. 4.13) and converted to a concentration of soil in a soil+water sample (in terms of dry mass, g/L). Afterwards, the amount of fines, in terms of the dry mass, was calculated by multiplying the concentration by the effluent volume.



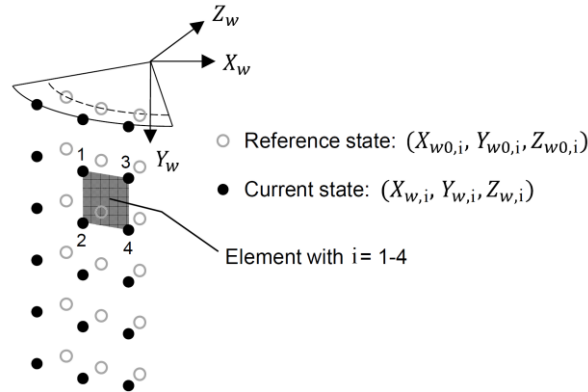
Fig. 4.13. Nephelometric Turbidimeter.

Afterwards, undrained triaxial compression tests ($\overline{CU}$ tests) were conducted at the strain rate of 0.1%/min and terminated when the axial strain (ϵ_a) reached 15%, keeping 380 kPa cell pressure and 300 kPa back pressure. $\overline{CU}$ tests on the specimens without suffusion, prepared in a similar method and subjected to 380 kPa of cell pressure and 80 kPa back pressure, were also conducted to compare the effect of suffusion on the WS-series specimens.

4.3 Estimation of Spatial Deformation During Suffusion Using Stereophotogrammetry



(a)



(b)

Fig. 4.14. (a) Graphical representation of stereophotogrammetry in geometry, and (b) reference state and current state of target points (Nishimura, 2022).

Digital image correlation (DIC) aided stereophotogrammetry strain measurement method was adopted to detect the spatial deformation of the specimen

WS_90Dc_omc during the erosion process. This method of three-dimensional displacement measurement has a precision to order of 10^{-3} mm, i.e., 0.001% strain in common size soil specimens. The strains obtained using this method were used to compute the axial and radial strains.

In this method, the 3-dimensional strain of soil specimens can be measured with the help of two sets (front and rear) of two digital cameras [camera L and camera R], as shown in Fig. 4.14. (a). The procedure involves image recording of the target points marked with black dots at 5-mm intervals (and a reference target line with red dots) on the rubber membrane.

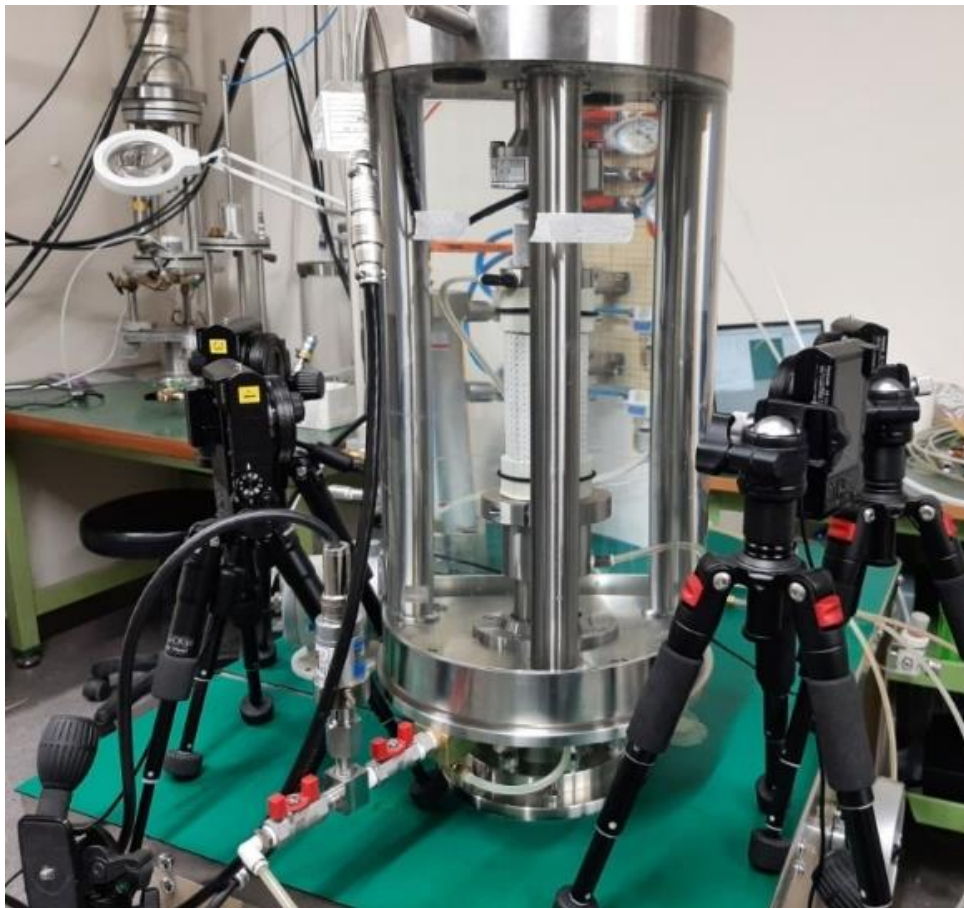


Fig. 4.15. Camera setup for stereophotogrammetry used in this study.

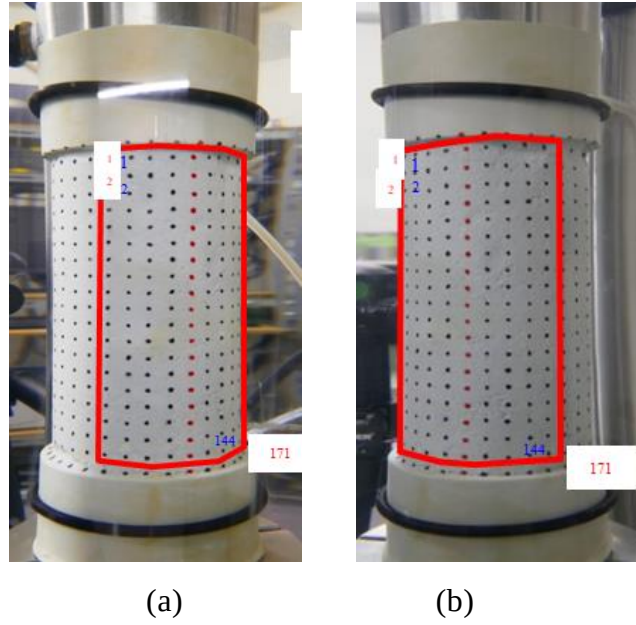


Fig. 4.16. Stereo images captured by front camera set (a) camera L, and (b) camera R.

The two sets of cameras (front camera set and rear camera set) at an approximate angle of 45° were placed on two sides of the triaxial cell at a distance of 200–250 mm (Fig. 4.15). The ray-tracing technique was adopted to correct the ray refraction due to interfaces (air and cell wall, cell wall, and water). The cameras were calibrated using a calibration sheet with targets at the known intervals. The images were recorded at 1-minute intervals (e.g., Fig. 4.16). The cameras can identify a common target point with u, v coordinates in image space (stereo-matching) using a pinhole camera model. The target points determined from the obtained images were converted into binary pixels, and the coordinates were computed in $u-v$ space with the aid of digital image correlation (DIC). The strain values on an element with $i = 1-4$ were obtained from the reference state and current state of the target points (1, 2, 3, and 4) using Eqs. 4.2 and 4.3 (Nishimura, 2022), followed by the reconstruction of the 3-dimensinal images of the specimens in terms of the elements in Python.

$$\text{Axial strain, } \varepsilon_a = -\frac{1}{2} \sum_{i=1}^2 \frac{(Y_{w,2i} - Y_{w,2i-1}) - (Y_{w0,2i} - Y_{w0,2i-1})}{(Y_{w0,2i} - Y_{w0,2i-1})} \quad \dots (4.2)$$

$$\text{Radial strain, } \varepsilon_r = -\frac{1}{4} \sum_{i=1}^4 \frac{\sqrt{(X_{w,i}^2 + Z_{w,i}^2)} - \sqrt{(X_{w0,i}^2 + Z_{w0,i}^2)}}{\sqrt{(X_{w0,i}^2 + Z_{w0,i}^2)}} \quad \dots (4.3)$$

where $X_{w0,i}$, $Y_{w0,i}$, and $Z_{w0,i}$ are the reference states, and $X_{w,i}$, $Y_{w,i}$, and $Z_{w,i}$ are the current states of each target point i , as shown in Fig. 4.14.(b).

Chapter 5

SUFFUSION CHARACTERISTICS OF PUMICE SAND

5.1 Effect of Compaction Condition on Degree of Suffusion

The results of the suffusion tests are shown in Table 5.1 in terms of the dry mass of the eroded fines. The specimens eroded during saturation even though the difference in pressure between the top and bottom of the specimen was, at most, 20 kPa. The amount of fines that eroded during saturation was measured for the collected effluent using the turbidimeter. The results show that the specimens lost a substantial amount of fine particles due to migration during this stage. During the suffusion tests (by the application of the hydraulic gradient), the eroded fines were sparse. However, the results are found to be consistent with the erosion occurring during the actual suffusion tests, namely, within a similar range (0.22% to 1.76%), which indicates the reliability of the tests. The small amount of erosion during the suffusion tests can be attributed to high density, high angularity, the small size of the specimens, and the fact that the majority of suffusion occurred involuntarily during the saturation stage, particularly the migration of the finer particles, which was unavoidable (Ke and Takahashi, 2014; Marot et al., 2012; Zhong et al., 2018). However, the total amount of soil that eroded during the saturation and suffusion processes can be regarded as the particles that eroded due to suffusion.

Moreover, after the completion of each test, a considerable mass of fine particles was observed to have accumulated on the pedestal and porous plate surface (Fig. 5.1). In an attempt to prevent the trapping of fines on the pedestal, the cone angle of the pedestal was increased at a later stage and tests were performed on the

specimens with $D_c \approx 95\%$. Although the trapping of fines was minimized, it could not be eliminated completely. The fine particles trapped on the pedestal and porous plate were collected carefully in a beaker after the completion of the experiment and the oven-dried mass was obtained.



Fig. 5.1. Fines trapped in (a) pedestal, and (b) porous plate.

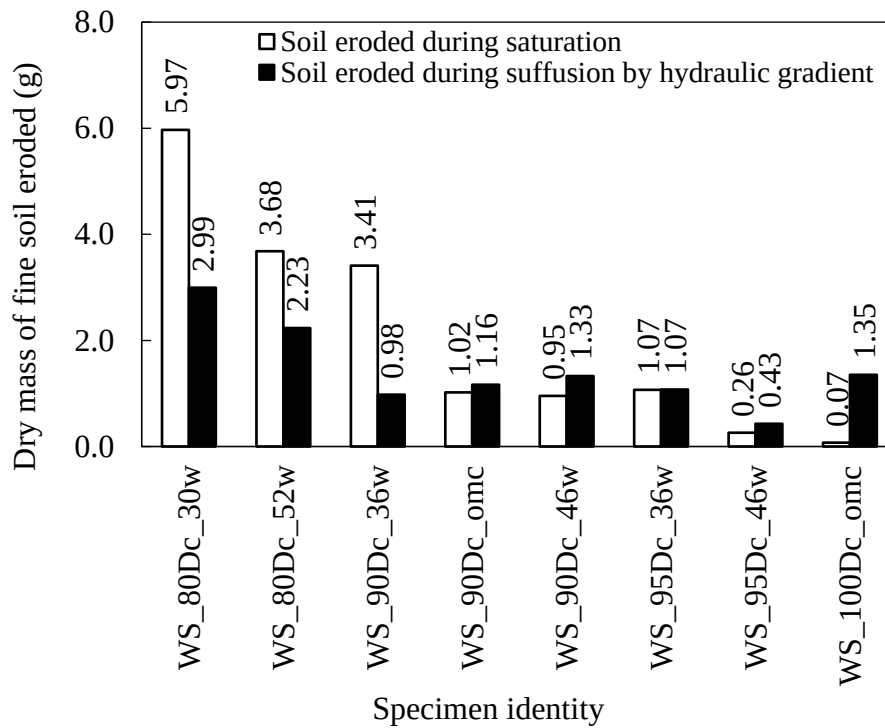


Fig. 5.2. Fractional distribution of particles that migrated in terms of dry mass in grams.

Thereafter, the amount of soil that eroded during suffusion by the hydraulic gradient is expressed as the combination of the dry mass of the soil that eroded during the suffusion tests (obtained from the effluent collected using the turbidimeter) and the oven-dried mass of the soil that was trapped on the pedestal. Finally, in this research, the degree or extent of suffusion is expressed as the percentage of total dry mass of soil eroded (= total dry mass of soil eroded/dry mass of specimen during preparation) during the complete experiment (i.e., summation of the two elements shown in Fig. 5.2).

Table 5.1. Results of suffusion tests under various compaction conditions and moisture contents.

Specimen identity	Dry mass of soil eroded during saturation (g)	Dry mass of soil eroded during suffusion by hydraulic gradient (g)	Total dry mass of soil eroded (g)	Percentage of total fines eroded (%)	Hydraulic conductivity at $i = 50$, k_{i50} (10^{-6} m/s)	Hydraulic conductivity at $i = 200$, k_{i200} (10^{-6} m/s)
WS_80Dc_30w	5.97	2.99	8.96	5.27	7.34	3.28
WS_80Dc_52w	3.68	2.23	5.92	3.58	2.98	1.87
WS_90Dc_36w	3.41	0.98	4.39	2.29	2.91	1.74
WS_90Dc_omc	1.02	1.16	2.18	1.16	3.35	1.86
WS_90Dc_46w	0.95	1.33	2.28	1.21	2.55	1.49
WS_95Dc_36w	1.07	1.07	2.14	1.08	14.6	4.35
WS_95Dc_46w	0.26	0.43	0.69	0.35	3.39	1.63
WS_100Dc_omc	0.07	1.35	1.42	0.68	0.95	0.85

The relationship between degree of saturation S_r at compaction with the percentage of fines that eroded during saturation and the percentage of total fines that eroded are plotted in Fig. 5.3. If the suffusion tests (by the application of the hydraulic gradient) had been conducted directly without the saturation or consolidation of the specimen, this would also have led to a high extent of erosion. Therefore, these two phenomena cannot be separated, and the total amount of erosion is seen as an important parameter to be considered when analyzing suffusion characteristics. For the case of $D_c \simeq 80\%$ (i.e., relatively loose specimens), a comparatively high percentage of total fines is eroded from the specimens on both dry and wet sides of the compaction curve. Loose specimens possess a larger constriction size and decreased inter-particle grain to grain contact. This causes the easy detachment of the fines leading to the initiation of suffusion at a lower hydraulic gradient, thereby leaving a lesser amount to erode during actual suffusion. Moreover, the presence of higher effective confining stress leads to a lesser extent of erosion during suffusion (80 kPa) than the extent of erosion occurring during the saturation process (10 kPa) (Ke and Takahashi, 2014). For these reasons, the amount of erosion during saturation becomes larger than that during suffusion (by application of the hydraulic gradient) in loose specimens. In addition, it is observed that S_r adversely influences the extent of suffusion as lower saturation results in more erosion for equally compact specimens. This increase in the amount of migrated fines can be ascribed to the change in the soil microfabric due to the variation in S_r . The spatial arrangement of the soil particles and voids (patterns of soil constituents), studied in thin sections with the help of an optical microscope, scanning electron microscope, computer tomography, etc., is referred to as the soil microfabric (Miedema and Oort, 1990). Differences in the S_r and D_c of the

soil alter the pore geometry, resulting in a change in the soil microfabric. Soil with low S_r at compaction (i.e., on the dry side of the *OMC*) develops a visible inter-pore network in the soil microfabric, called macro-porosity, while soil with high S_r at compaction (i.e., on the wet side of the *OMC*) develops invisible intra-pores in the fine particles, called micro-porosity. Macro-porosity allows more water to pass through the soil, increasing the kinetic potential of the soil grains and influencing the detachment of fines from the soil matrix. For this reason, the presence of macro-porosity makes the soil more erodible. For the specimen with $D_c \approx 100\%$ at the *OMC*, it was found that the erosion occurring during saturation is trivial and that the majority of the fines migrated during suffusion by the application of the hydraulic gradient. This is due to the fact that the dense packing of soil creates a smaller constriction size, allowing a lesser amount of fines to detach and migrate under the low hydraulic gradient that developed during saturation.

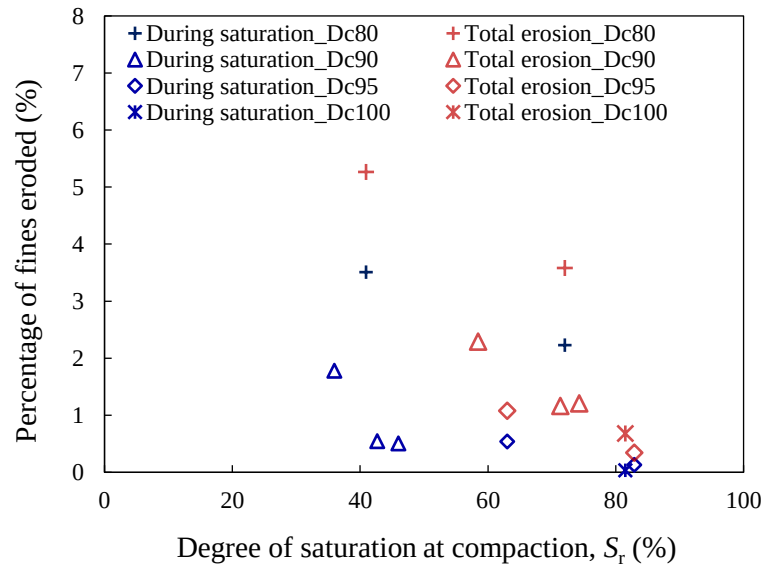
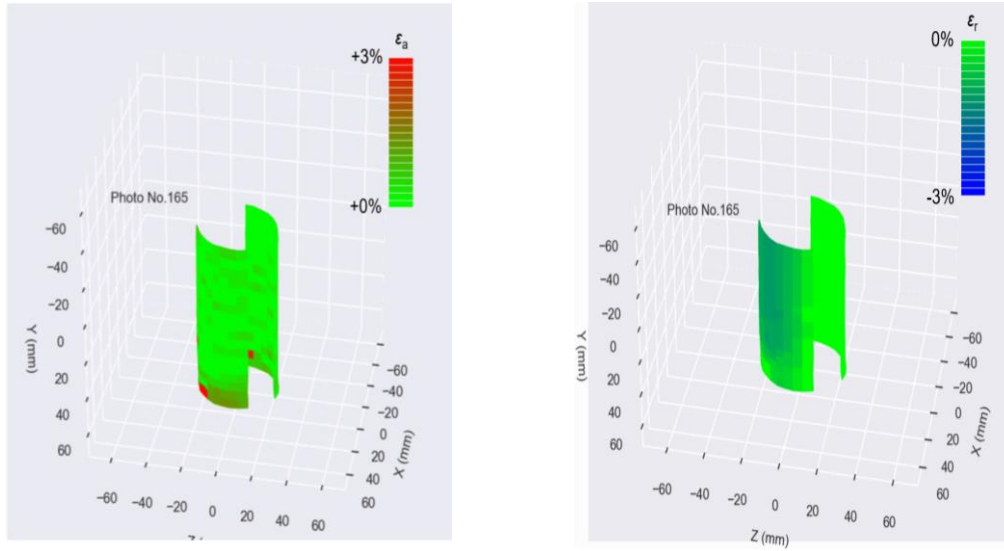


Fig. 5.3. Percentage of fines that eroded during saturation and suffusion corresponding to degree of saturation at compaction.

5.2 Spatial Deformation due to Suffusion

The deformation of the specimens subjected to suffusion was found to be non-destructive and was not feasible to detect by simple visual observation. The spatial deformation of specimen WS_90Dc_omc, during suffusion by the hydraulic gradient, was determined using the stereophotogrammetry technique developed by Nishimura (2022). Figure 5.4(a) presents the axial strain, while Fig. 5.4(b) presents the radial strain at the end of suffusion in the reconstructed image of the 3-dimensional space with compression as positive strain. At the end of suffusion, a maximum axial strain of 3% is spotted in a few elements in the lower part of the specimen, whereas axial deformation of less than 1% is detected in the upper and middle parts of the specimen. These observations indicate that the bottom surface of the specimen is more affected by suffusion than the other parts of the specimen. The radial strains were detected as less than -1% by front camera set and +1% by rear camera set. A maximum radial strain of +1.4% is detected near the top surface of the specimen. The marginal axial deformation and radial deformation on the top and middle parts of the specimen suggest that the erosion began in the upper part of the specimen, then the fines migrated to the lower part and clogged the pores, thus affecting the extent of suffusion in the lower part of the specimen (with the bottom surface as the exception). The face averages of the axial strain (ϵ_a) and radial strain (ϵ_r) are estimated as 0.110% and 0.295%, respectively, from the strain of each element obtained from the stereophotogrammetry analysis which resulted in 0.7% volumetric strain (ϵ_v). Details on the determination of spatial deformation using a stereophotogrammetry analysis are provided in Section 4.3.



(a) Axial strain

(b) Radial strain

Fig. 5.4. Deformation observed at the end of suffusion.

5.3 Volume Change due to Migration of Fines

The volumetric strain of the with suffusion (WS) specimens at different degrees of compaction due to migration of fines during saturation and during suffusion are shown in Fig. 5.5. The volume change during saturation was measured from the difference in the volume gauge reading after saturation for WS compared to the volume gauge reading after saturation for WoS specimens at similar compaction condition. The volume change during suffusion was measured from the difference in volume gauge reading for WS specimens before and after suffusion. Although conservative in nature, the values were found to be in agreement with the volume change calculated from the image analysis, for the specimen WS_90Dc_omc, as mentioned in section 5.2. It was observed that the volume changes of the densely compacted with suffusion specimens were within the range of 0.01% to 0.70% during saturation stage, and 0.04% to 1.02% during the suffusion test by the application of hydraulic gradient, which are not significant and can be deemed as non-destructive.

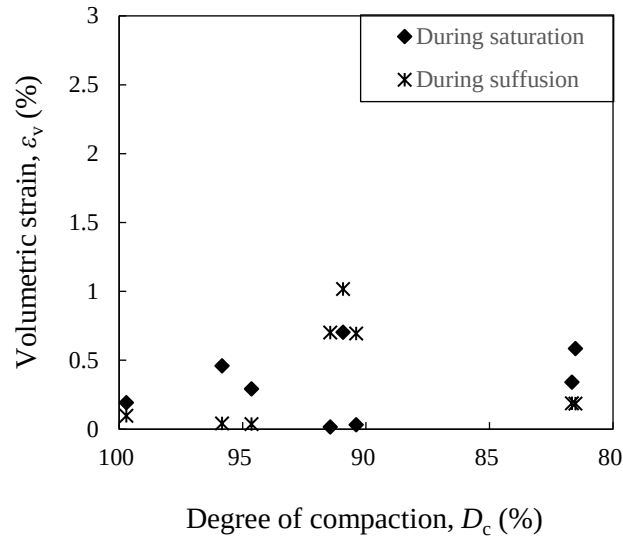


Fig. 5.5. Volumetric strain in WS specimens during saturation and suffusion.

5.4 Effect of Suffusion on Hydraulic Conductivity

Hydraulic conductivity, k , of the WS-series specimens is determined from the volume of the effluent collected during the suffusion tests using Darcy's law which states

$$Q = \frac{V_e}{t} = kiA \quad \dots (5.1)$$

where Q is the flow rate, V_e is the volume of effluent, t is the flow duration, i is the hydraulic gradient, and A is the area of the specimen. The hydraulic conductivity at the start of suffusion ($i = 50$) and that at the end of suffusion ($i = 200$) are listed in Table 5.1. The complex network of pores has made the flow path of the WS-series specimens twisted, causing tortuosity. Moreover, the flow path has become narrower due to clogging by the migrated fines which have been trapped at a bottleneck of the flow path. The flow path is further elongated by the random movement of fast-moving water through the narrow, bottlenecked pore network which has resulted in a decrease in hydraulic conductivity with an increase in hydraulic gradient. A similar range in

hydraulic conductivity after suffusion ($i = 200$), within the range of 8.5×10^{-7} to 4.5×10^{-6} m/s, is observed for all densities. Watabe and Nishimura (2020) determined the hydraulic conductivity of the specimens without suffusion at very high densities, i.e., D_c ranging from 96% to 101% along Proctor's normal compaction curve, as shown in Fig. 5.6. The hydraulic conductivities of the very high-density WS-series specimens (WS_95Dc_36w, WS_95Dc_46w, and WS_100Dc_omc) at $i = 200$ have been significantly affected by suffusion compared to those in Watabe and Nishimura (2020), causing the hydraulic conductivity of the WS-series specimens to increase significantly.

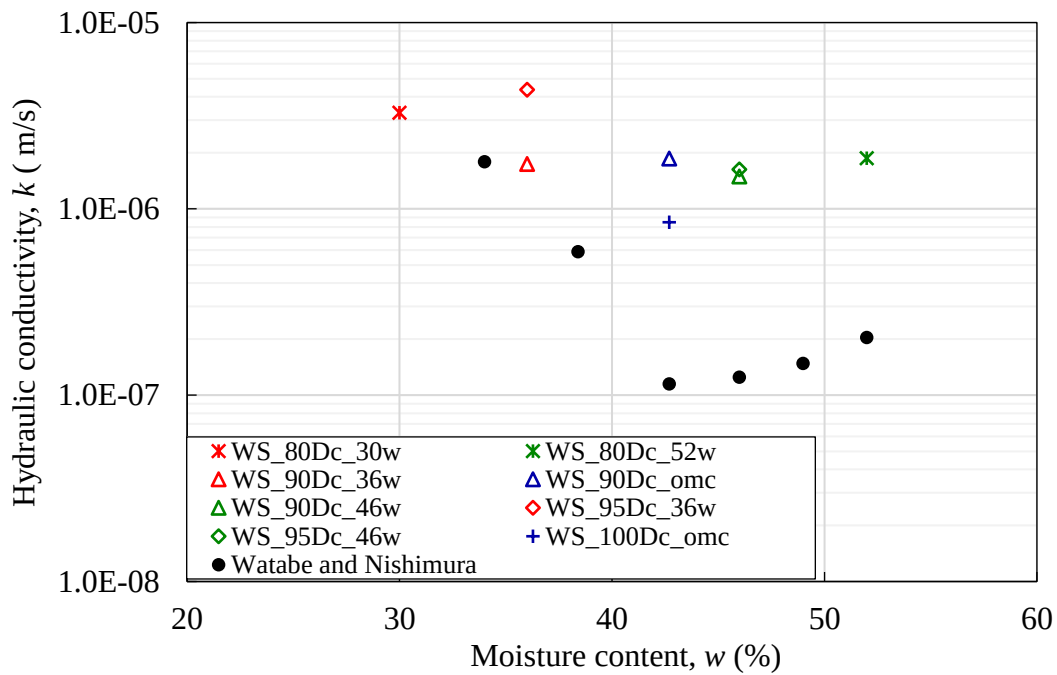


Fig. 5.6. Hydraulic conductivity corresponding to moisture content.

In summary, the removal of fine particles during the saturation process causes a significant increase in hydraulic conductivity, while the trapped fine particles clogged the flow path during the application of the hydraulic gradient, resulting in a

slight decrease in final hydraulic conductivity. However, the absolute value of hydraulic conductivity before shear remains higher for the WS-series specimens which indicates an increase in voids, i.e., porosity, due to the removal of the fines.

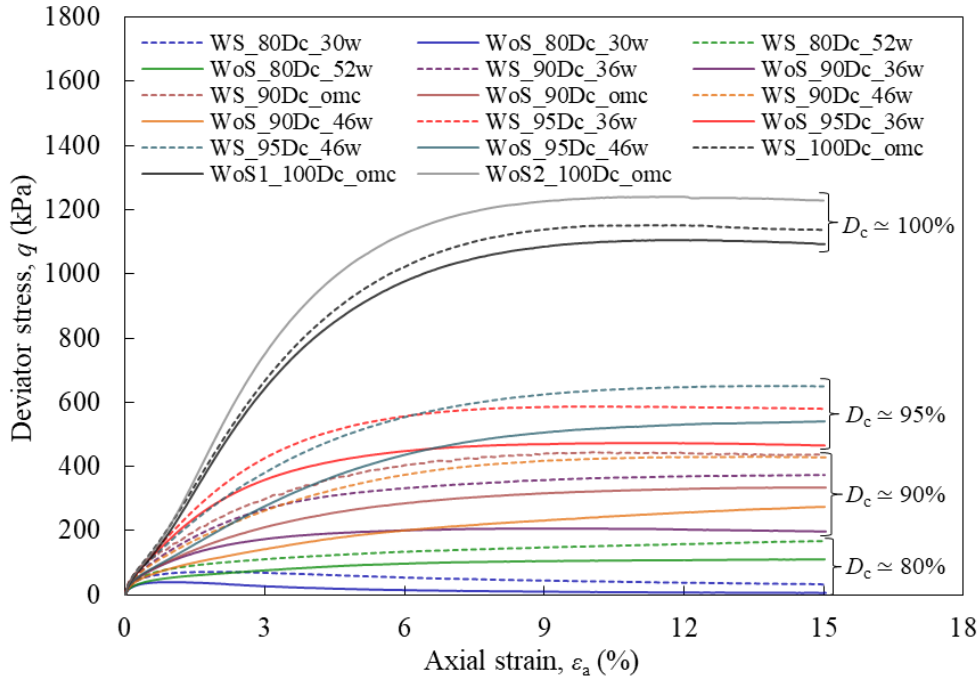
Chapter 6

INFLUENCE OF SUFFUSION ON MECHANICAL PROPERTIES OF PUMICE SAND

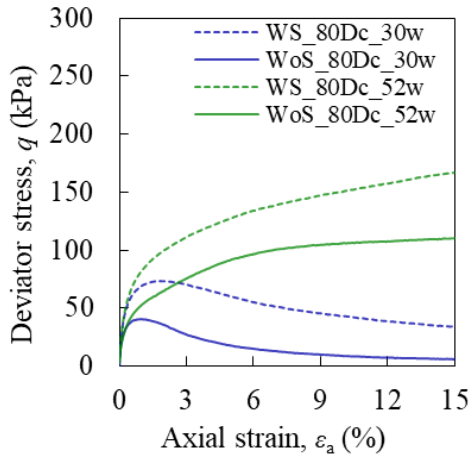
6.1 Shear Strength Behavior

The relationship between deviator stress q and axial strain ε_a under different degrees of compaction and moisture contents for both WS- and WoS-series specimens is depicted in Fig. 6.1. At low density, the WoS-series specimens on the dry side of the *OMC* show strain-softening behavior, whereas the WS-series specimens with similar compaction conditions display a similar tendency with the improvement in residual shear strength q_R . The WS-series specimens on both the wet side of the *OMC* and at the *OMC* exhibit strain-hardening behavior with a significant increase in q_R , although the D_c values of WS_90Dc_46w and WS_95Dc_46w are marginally smaller than those of WoS_90Dc_46w and WoS_95Dc_46w. Moreover, the value of the residual strength q_R for the WS_100Dc_omc specimen ($D_c = 99.71\%$) is found to be 7% lower than the WoS-series specimen with $D_c = 100.78\%$. An additional experiment with another WoS-series specimen at $D_c = 99.82\%$ was conducted to confirm this observation, and q_R is found to have increased by 4% compared to the WS-series specimen. These two test results apparently do not coincide with each other; however, this can be explained by taking into account the very high undrained residual shear strength of such densely packed specimens and their sensitivity to a minute variation in D_c . Therefore, at the very high degree of compaction of $D_c \simeq 100\%$, the influence

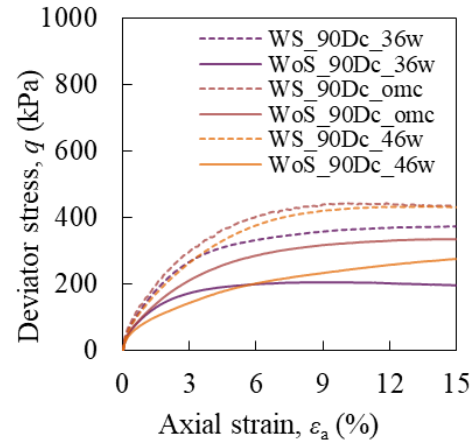
of suffusion on the residual shear strength is complex, yet not detrimental to pumice sand owing to its high strength.



(a)



(b)



(c)

Fig. 6.1. Relationship between deviator stress and axial strain (a) all specimens combined, (b) $D_c \approx 80\%$ specimens, (c) $D_c \approx 90\%$ specimens.

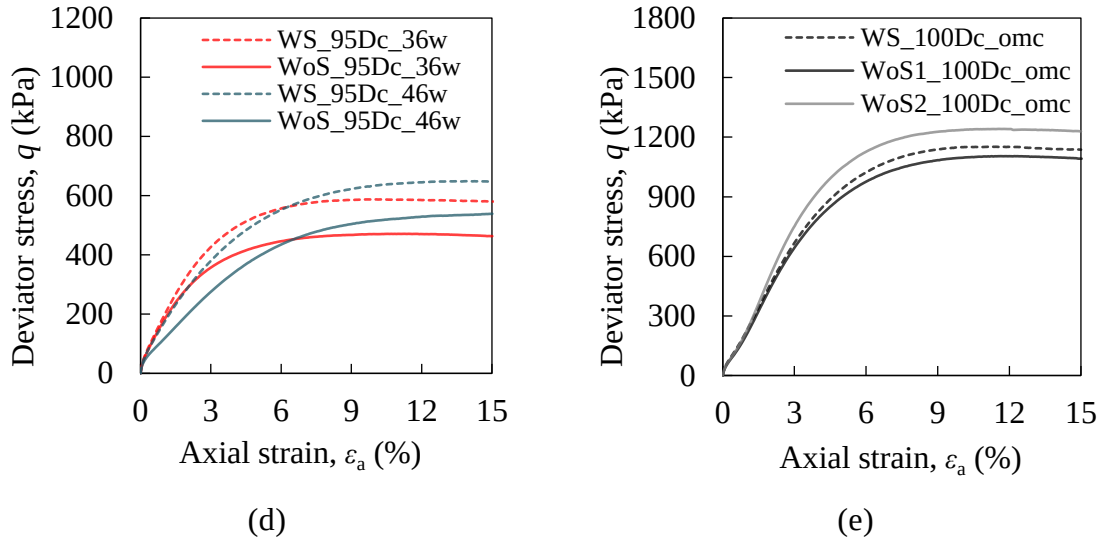


Fig. 6.1.(contd.) Relationship between deviator stress and axial strain (d) $D_c \approx 95\%$ specimens, and (e) $D_c \approx 100\%$ specimens.

The relationship between the brittleness index and the degree of compaction is shown in Fig. 6.2. Brittleness index I_B is an indicator of the soil's collapsibility during undrained triaxial shear and is defined as

$$I_B = \frac{q_{\max} - q_R}{q_{\max}} \quad \dots (6.1)$$

where $q_{\max}$ is the maximum deviator stress and q_R is the residual shear strength of the specimens (Bishop, 1967; Yoshimine et al., 1999). The value of $I_B = 1$ indicates the complete loss of strength, while $I_B = 0$ signifies dilative behavior and the absence of strain softening. The WoS_80Dc_30w specimen shows collapsible or brittle behavior with a high brittleness index value which is reduced in the WS_80Dc_30w specimen, indicating lowered collapsibility [Fig. 6.1.(b)]. Brittleness index values of nearly zero are observed in the case of the remaining specimens with no or only trivial changes in brittleness index for the WS-series specimens compared to the WoS-series specimens.

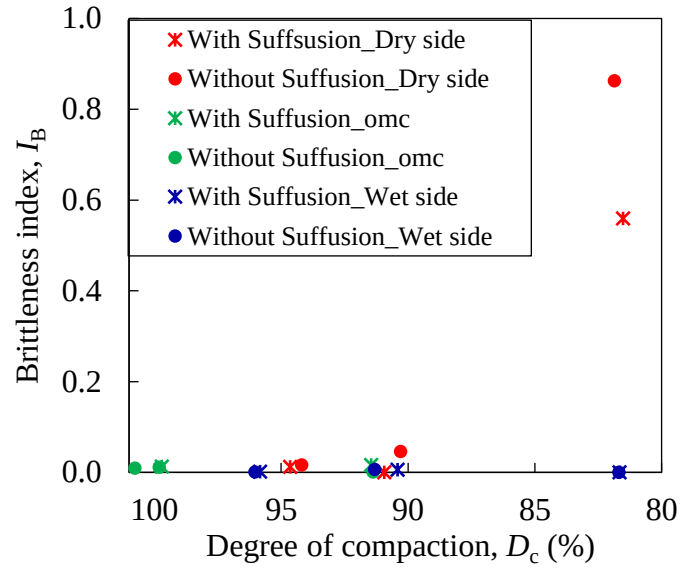


Fig. 6.2. Brittleness index corresponding to degree of compaction.

Figure 6.3 depicts a comparative study on the effect of suffusion on the maximum (or peak) deviator stress, $q_{\max}$, corresponding to density and moisture content conditions, as well as the existing research on suffusion on a similar soil. Suffusion tends to increase the maximum deviator stress for degree of compaction up to 95%. Specimens on the wet side of the *OMC* exhibit higher maximum deviator stress values than those on the dry side of the *OMC* with similar degree of compaction for both WS and WoS-series specimens. Chitravel et al. (2021) conducted undrained shear tests on suffusion-induced reconstituted volcanic ash using the physical properties of Series A in Table 3.2. For comparison, Fig. 6.3 displays the values obtained by them in terms of degree of compaction recalculated using the physical properties of Series B shown in Table 3.2.

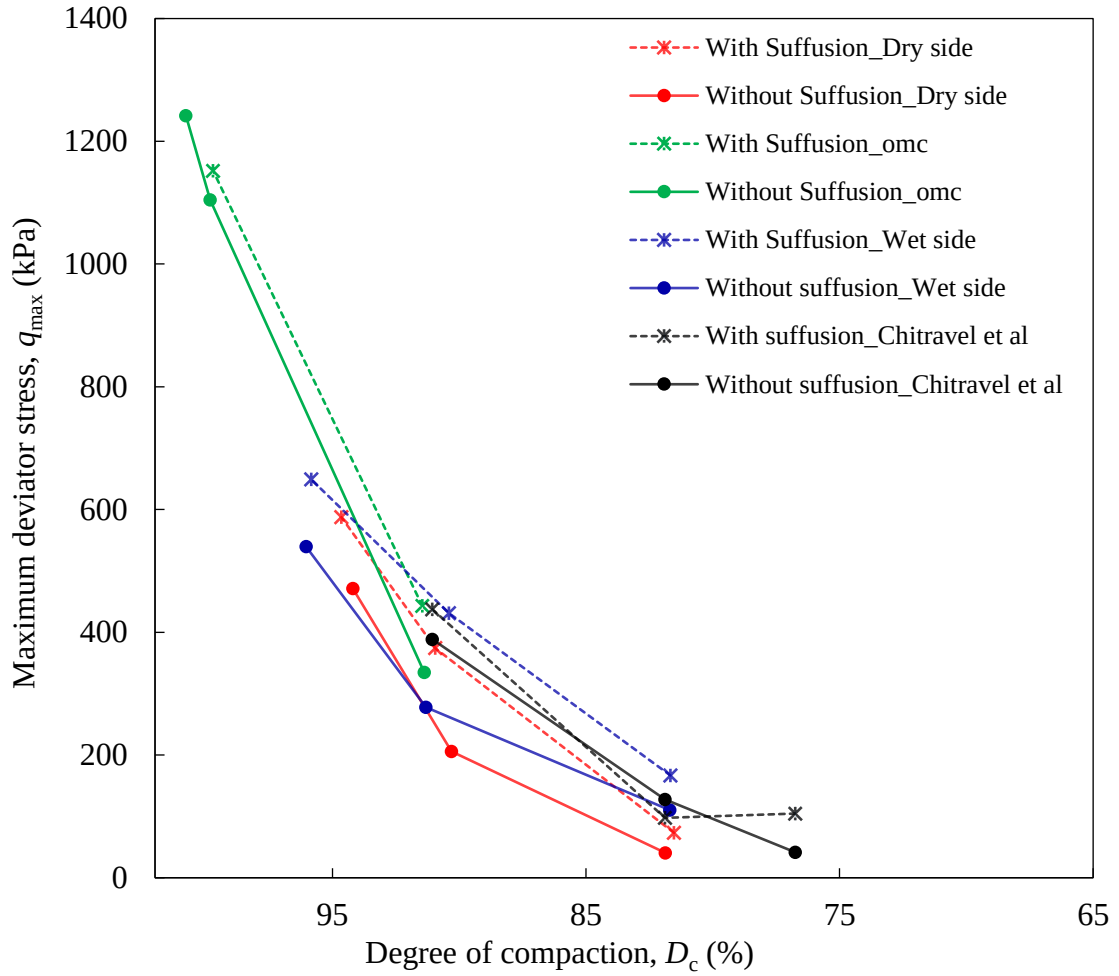


Fig. 6.3. Relationship between maximum deviator stress and degree of compaction.

The normalized maximum deviator stress ($q_{\max}/\sigma'_c$, where σ'_c is the effective consolidation stress), corresponding to the degree of compaction with the moisture content condition, is shown in Fig. 6.4. Both the WS- and WoS-series specimens demonstrate an exponential increase in $q_{\max}/\sigma'_c$ with increasing D_c (i.e., density). The regression coefficients (coefficients of determination, R^2) for the exponential equations of the trend lines, without distinguishing between the dry and wet side specimens, are within an acceptable range (≥ 0.9) with values of 0.925 and 0.937 for the WS and WoS conditions, respectively.

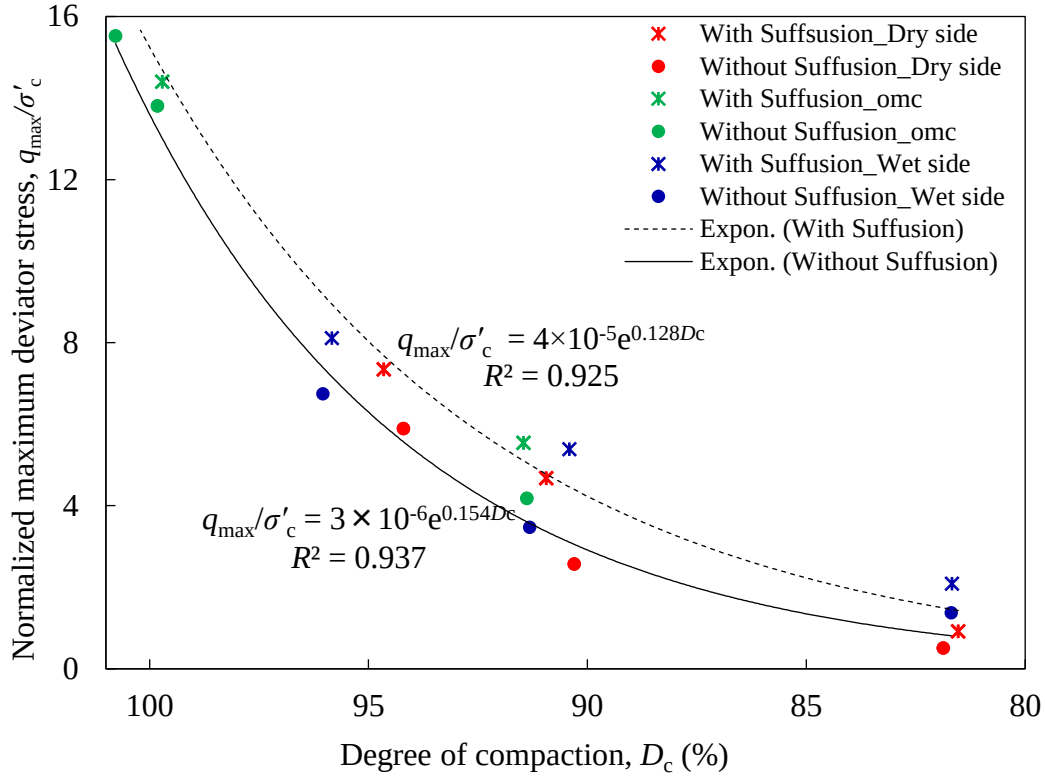


Fig. 6.4. Relationship between normalized maximum deviator stress and degree of compaction.

6.2 Stress Paths

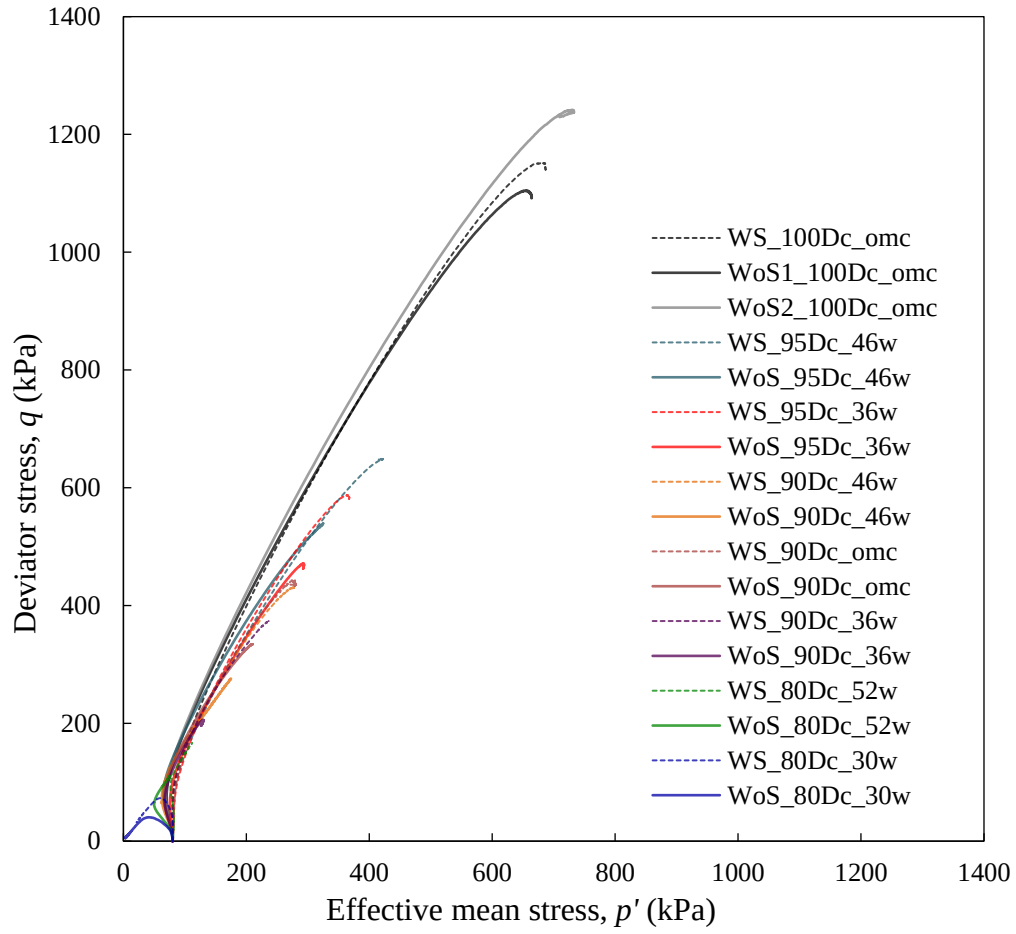
Figure 6.5 presents the effective stress paths (ESPs) of the WS- and WoS-series specimens. The effective stress path of the WoS_80Dc_30w specimen exhibits unstable behavior by moving to the left and displays undrained instability resembling the static liquefaction tendency, which is improved in the WS_80Dc_30w specimen [Fig. 6.5(b)]. Static liquefaction is characterized by zero or negligible undrained shear strength due to the development of large excess pore water pressure (Yamamuro and Lade, 1997; Picarelli et al., 2020). The specimens on the wet side of the *OMC* at $D_c \simeq 80\%$ exhibit unstable behavior at the initial stage, which becomes stable later on, under both WS and WoS conditions. The rest of the specimens display stable behavior

for all the WS- and WoS-series specimens. Moreover, the $D_c \simeq 80\%$ specimens (dry and wet sides) converge at the critical state line (CSL) whereas, for the rest of the specimens with $D_c \geq 90\%$, the effective stress paths do not appear to converge at the critical state line as the slopes of the ESPs continue to decrease until the end of the experiment. This makes the determination of the slope of the critical state line complex. Therefore, the slope of the effective stress path at $q_{\max}$, $M_{q_{\max}}$, and the maximum slope of the effective stress path, $M_{\max}$, which are analogous to the slope of the critical state line, are used in the following sections.

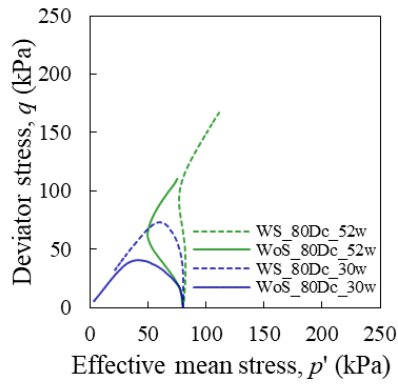
Figure 6.6(a) displays the relationship among the degree of compaction, slope of the ESP at $q_{\max}$, $M_{q_{\max}} = \frac{q_{\max}}{p'}$, and shear resistance angle at $q_{\max}$, $\phi_{q_{\max}}$, for the WS- and WoS-series specimens. The value of $M_{q_{\max}}$ increases in response to an increase in density with values ranging from 0.96 to 1.70. A higher $M_{q_{\max}}$ is observed for the WS-series specimens for $D_c \simeq 80\%$ (wet and dry sides), 90% (*OMC*), and 95% (dry side) compared to the $M_{q_{\max}}$ of the similarly compacted WoS-series specimens. However, suffusion is not shown to generate any significant change in $M_{q_{\max}}$ for $D_c \simeq 100\%$ for the *OMC* specimens and reduces the $M_{q_{\max}}$ marginally for the $D_c \simeq 90\%$ (dry and wet sides) and $D_c \simeq 95\%$ (wet side) specimens. The shear resistance angle at $q_{\max}$, $\phi_{q_{\max}}$, corresponding to $M_{q_{\max}}$, is calculated from the following relationship:

$$M_{q_{\max}} = \frac{q_{\max}}{p'} = \frac{6 \sin \phi_{q_{\max}}}{3 - \sin \phi_{q_{\max}}} \quad \dots (6.2)$$

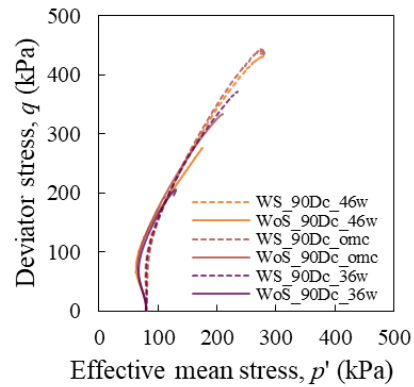
in which a tendency similar to $M_{q_{\max}}$, with values ranging from 24° to 42° , is displayed.



(a)

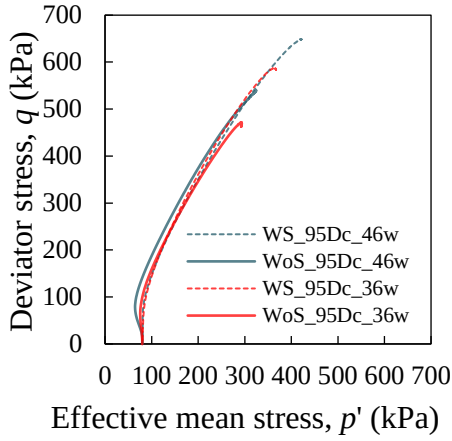


(b)

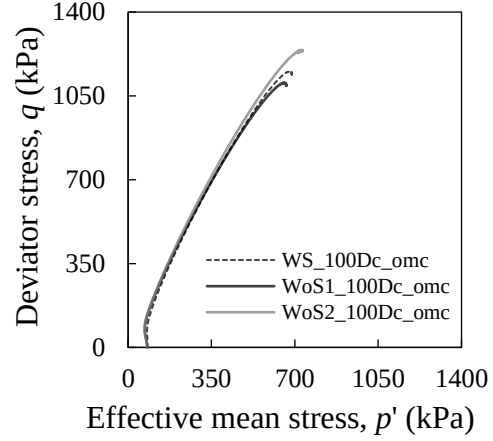


(c)

Fig. 6.5. Effective stress paths of WS- and WoS-series specimens under various degrees of compaction (a) all specimens combined, (b) $D_c \approx 80\%$ specimens, (c) $D_c \approx 90\%$ specimens.



(d)

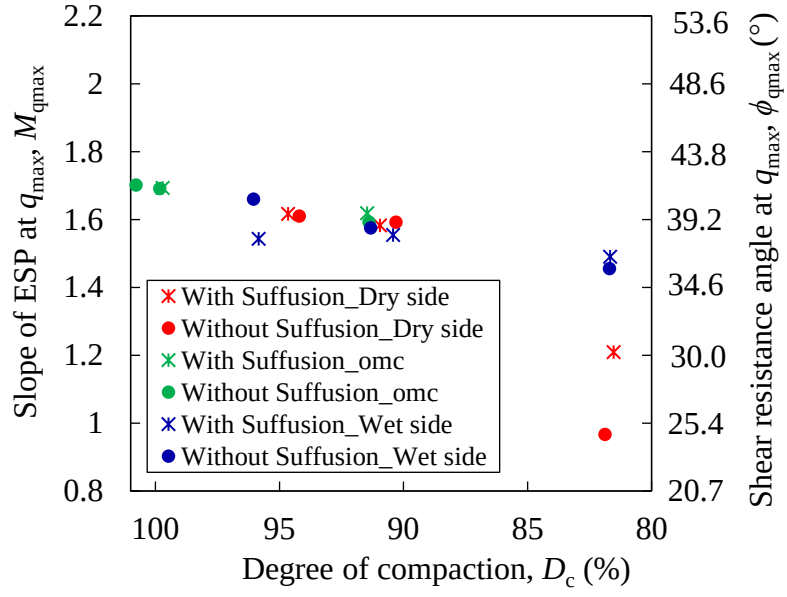


(e)

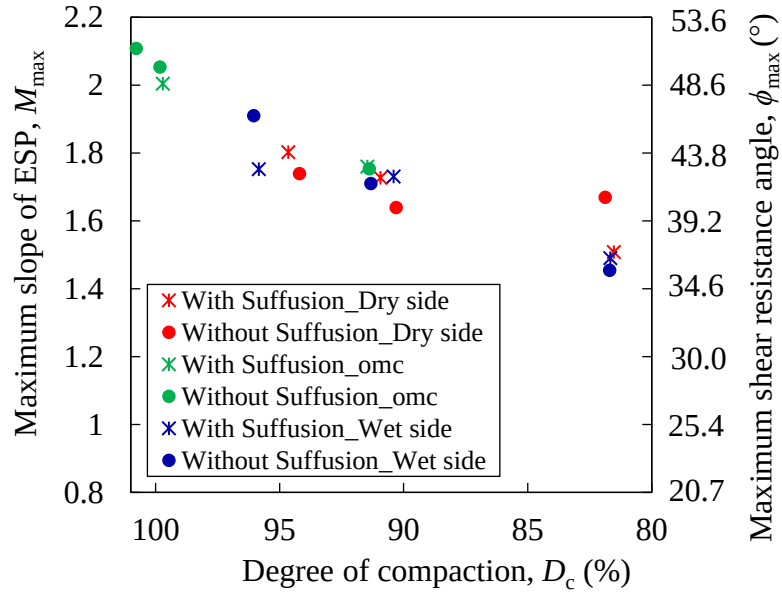
Fig. 6.5.(contd.) Effective stress paths of WS- and WoS-series specimens under various degrees of compaction (d) $D_c \approx 95\%$ specimens, and (e) $D_c \approx 100\%$ specimens.

The maximum slopes of the effective stress paths, $M_{\max} = \left(\frac{q}{p'}\right)_{\max}$, are found to be in the range of 1.46 to 2.11, as shown in Fig. 6.6. (b). For the dry side specimens, suffusion decreases $M_{\max}$ for $D_c \approx 80\%$, but increases it for $D_c \approx 90\%$ and 95% . For specimens at the *OMC*, suffusion does not influence the $M_{\max}$ significantly for $D_c \approx 90\%$ and decreases it for $D_c \approx 100\%$. For the wet side specimens, suffusion increases $M_{\max}$ for $D_c \approx 80\%$ and 90% , and decreases it for $D_c \approx 95\%$. Maximum shear resistance angle $\phi_{\max}$ is calculated from the following relationship:

$$M_{\max} = \left(\frac{q}{p'}\right)_{\max} = \frac{6 \sin \phi_{\max}}{3 - \sin \phi_{\max}} \quad \dots (6.3)$$



(a)



(b)

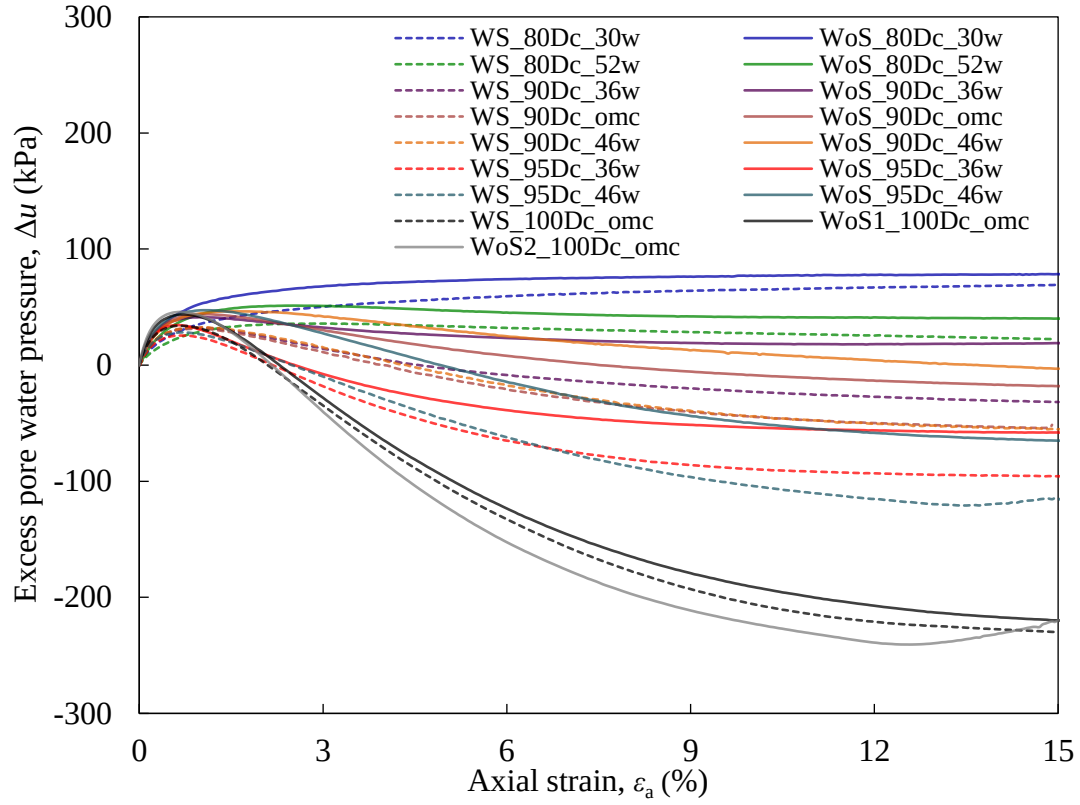
Fig. 6.6. Relationships among (a) slope of ESP at q_{max} , shear resistance angle at q_{max} , and degree of compaction, and (b) maximum slope of ESP, maximum shear resistance angle, and degree of compaction.

The values for $\phi_{\max}$ are all in the range of 24° to 42° and display a tendency similar to that of $M_{\max}$. The values for $\phi_{q\max}$ and $\phi_{\max}$, shown in secondary scale in Figs. 6.6. (a) and (b), are calculated from the values for $M_{q\max}$ and $M_{\max}$ in the respective primary scale using Eqs. (6.2) and (6.3). Therefore, the converted values for $\phi_{q\max}$ and $\phi_{\max}$ in secondary scale are not linear.

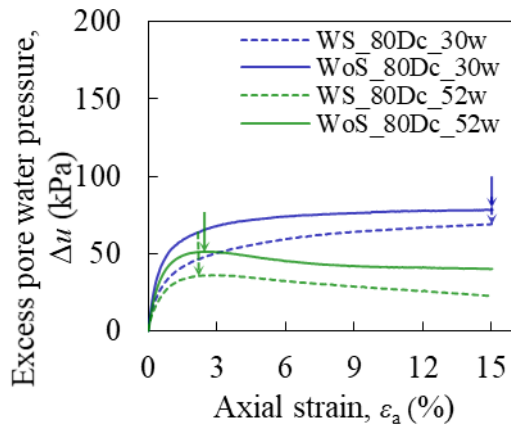
6.3 Dilatancy Behavior

The relationship between excess pore water pressure Δu and axial strain ϵ_a of the WS- and WoS-series specimens is shown in Fig. 6.7. The value of excess pore water pressure continues to increase until it reaches a certain value and then remains constant (thereafter) in the $D_c \simeq 80\%$ specimens on the dry side of the *OMC*. This signifies that the specimens tend to contract during undrained shear until reaching the peak Δu value, after which no change in total volume occurs although the axial strain increases. The rest of the specimens with $D_c \simeq 80\%$ on the wet side of the *OMC*, as well as the $D_c \simeq 90\%$, 95% , and 100% specimens on both the dry and wet sides of the *OMC*, show a decrease in Δu after reaching the peak value under both WS and WoS conditions. This peak signifies a change in the contractive behavior from contraction to dilation which is called the phase transformation point (PTP) (Daouadji et al., 2016). The respective PTPs of every specimen are marked with downward arrows in Figs. 6.7. (b), (c), (d), and (e). Dilation is more significant in the high-density specimens under both WS and WoS conditions. The WS-series specimens exhibit a higher dilation tendency when compared to the WoS-series specimens. The WS-series specimens have lower Δu at the PTPs than the WoS-series specimens under a similar

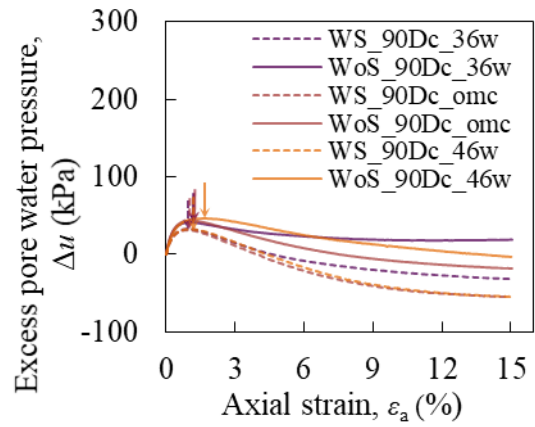
compaction condition (Table 6.1). The change in excess PWP and the normalized change in excess PWP are estimated as



(a) Excess pore water pressure corresponding to axial strain

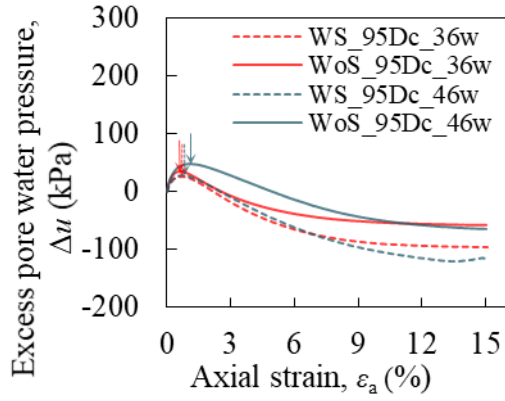


(b) $D_c \approx 80\%$ specimens showing respective PTPs

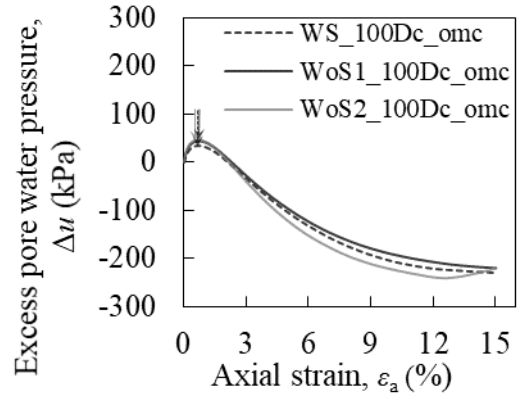


(c) $D_c \approx 90\%$ specimens showing respective PTPs

Fig. 6.7. Relationship between excess pore water pressure and axial strain.



(d) $D_c \approx 95\%$ specimens showing respective PTPs



(e) $D_c \approx 100\%$ specimens showing respective PTPs

Fig. 6.7.(contd.) Relationship between excess pore water pressure and axial strain.

$$\text{Change in excess PWP} = \Delta u_{\text{PTP}} - \Delta u_{15\%} \quad \dots (6.4)$$

$$\text{Normalized change in excess PWP} = \frac{\Delta u_{\text{PTP}} - \Delta u_{15\%}}{\sigma'_c} \quad \dots (6.5)$$

where Δu_{PTP} is the excess pore water pressure at the phase transition point and $\Delta u_{15\%}$ is the excess pore water pressure at an axial strain of 15%. Equation (6.5) corresponds to the absolute value of the change in negative Δu generated due to dilation after the phase transition point. The initiation of suffusion primarily involves the migration of the finest particles, thereby resulting in a reduction in the particle size ratio and a shift of the soil towards homogeneity by the removal of fine grains at the end of suffusion. The particle size ratio is defined as the ratio of the diameter of the largest grains in a sample to the diameter of the smallest grains in the sample (Wiacek and Stasiak, 2018). As the particle size ratio decreases, there is a corresponding increase in the void ratio of the soil matrix. This results in a change in the soil matrix, resulting in an increase in the excess angle (the difference between the shearing resistance angle at q_{max} , $\phi_{q_{\text{max}}}$,

and the maximum shear resistance angle, $\phi_{\max}$, of sand under the application of stress, finally increasing the dilatancy (Bolton, 1986; Xiao et al., 2014).

Table 6.1. Excess pore water pressure and corresponding strain of each specimen.

Specimen identity	Excess PWP at PTP, Δu_{PTP} (kPa)	Axial strain at PTP, $\epsilon_{a,\text{PTP}}$ (%)	Excess PWP at 15% strain, $\Delta u_{15\%}$ (kPa)	Change in excess PWP (kPa)	Normalized change in excess PWP
WS_80Dc_30w	(69.4)*	(15)*	69.4	(0)*	(0)*
WoS_80Dc_30w	(78.3)*	(15)*	78.3	(0)*	(0)*
WS_80Dc_52w	35.9	2.50	22.5	13.4	0.168
WoS_80Dc_52w	51.3	2.75	40.1	11.2	0.140
WS_90Dc_36w	32.4	0.94	-31.7	64.1	0.801
WoS_90Dc_36w	41.3	1.04	19.0	22.3	0.279
WS_90Dc_omc	31.4	0.95	-51.4	82.8	1.035
WoS_90Dc_omc	43.8	1.11	-18.1	61.9	0.774
WS_90Dc_46w	32.8	1.03	-55.3	88.1	1.101
WoS_90Dc_46w	46.4	1.43	-3.0	49.4	0.618
WS_95Dc_36w	25.5	0.68	-95.9	121.4	1.517
WoS_95Dc_36w	34.5	0.62	-58.0	92.5	1.156
WS_95Dc_46w	28.2	0.72	-115.1	143.3	1.791
WoS_95Dc_46w	47.3	1.06	-65.2	112.5	1.406
WS_100Dc_omc	34.0	0.67	-230.0	264.0	3.300
WoS1_100Dc_omc	43.7	0.73	-219.9	263.6	3.295
WoS2_100Dc_omc	45.9	0.66	-219.7	265.6	3.319

* The excess pore water pressure at an axial strain of 15% is regarded as the PTP since it did not appear until the end of the experiment due to contractive behavior.

The relationship between the axial strain at the phase transition point, $\varepsilon_{a,PTP}$, and degree of compaction D_c is shown in Fig. 6.8. The increase in density is accompanied by a decrease in $\varepsilon_{a,PTP}$. The WS-series specimens reach the phase transition point at a lower axial strain, i.e., prior to that of the WoS-series specimens, indicating a lesser amount of contraction during shear. However, it is notable that the $\varepsilon_{a,PTP}$ of the WS-series specimens is lower than that of the WoS-series specimens at $D_c \simeq 95\%$ (dry side), and it is almost the same as that for both the WS- and WoS-series specimens with $D_c \simeq 100\%$ at the *OMC*.

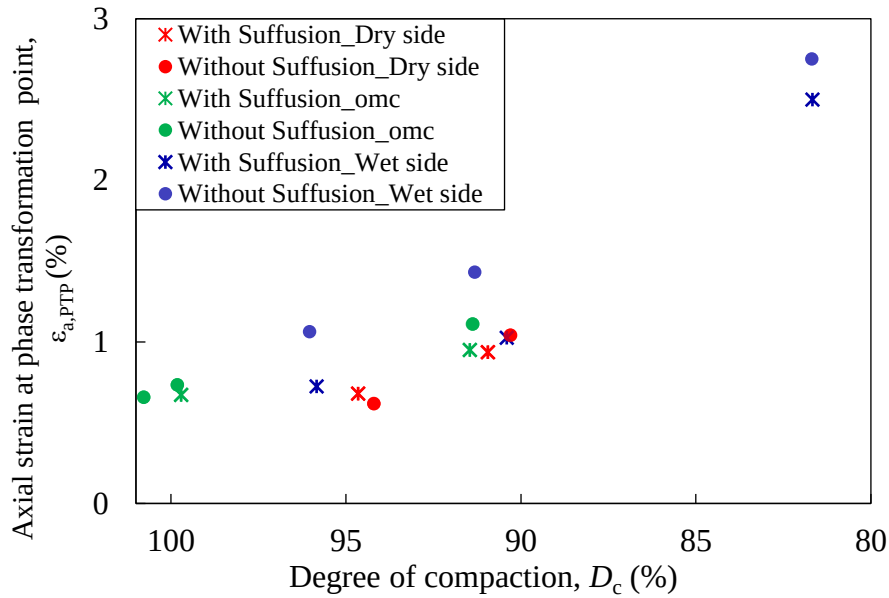


Fig. 6.8. Relationship between axial strain at phase transformation point and degree of compaction.

The relationship between the normalized change in excess pore water pressure and the degree of compaction of the WS- and WoS-series specimens is shown in Fig. 6.9. In the density-dependent changes in Δu or dilation tendency, polynomial increases in the normalized change in excess pore water pressure $[(\Delta u_{PTP} - \Delta u_{15\%})/\sigma'_c]$ are spotted with increasing density under both WS and WoS conditions. The regression

coefficients (coefficients of determination, R^2) for the polynomial equations of the trend lines, without distinguishing between the dry and wet side specimens, are 0.975 and 0.983 for the WS and WoS conditions, respectively.

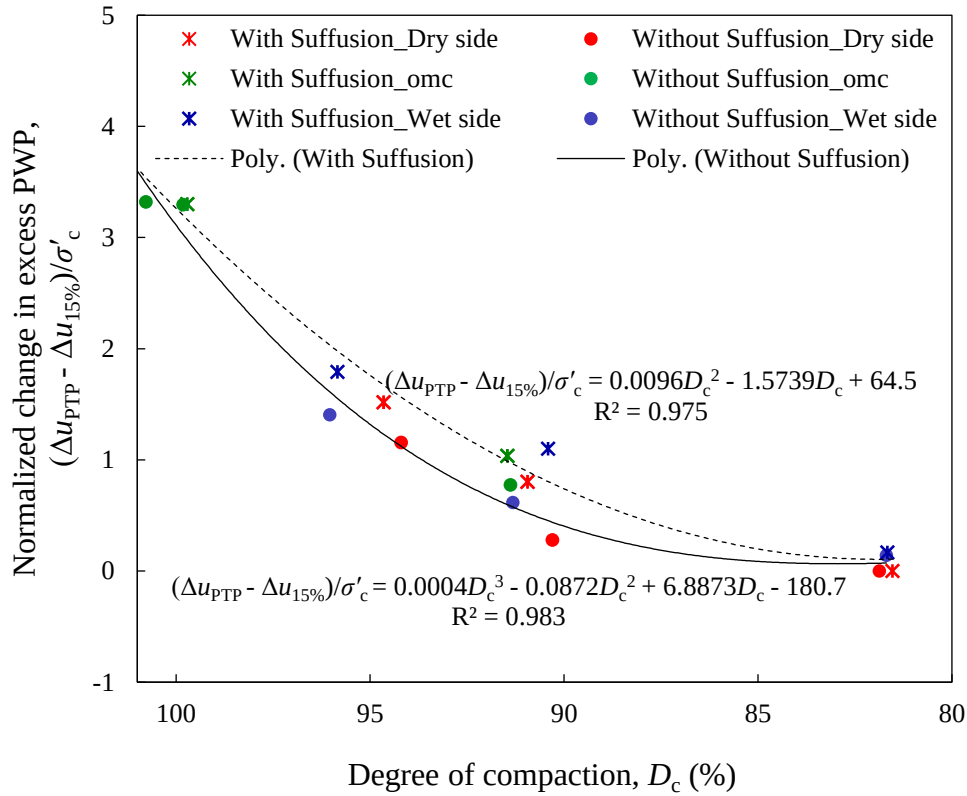


Fig. 6.9. Relationship between normalized change in excess pore water pressure and degree of compaction.

6.4 Microstructural Change in Pumice Sand due to Suffusion

The geomechanical behavior of natural and compacted soil depends on the arrangement of the solid soil components and soil pores, referred to as the soil microstructure (Romero et al., 1999; Tang et al., 2020). Changes in the mechanical behavior of the WS-series specimens, compared to the WoS-series specimens, indicate an alteration in the soil microstructure.

Figure 6.10 shows the variation in dry density, ρ_d , after the four stages of the experiment, namely, specimen preparation (stage 1), saturation (stage 2), consolidation (stage 3), and suffusion by hydraulic gradient/before shear (stage 4). The dry density after any stage n ($=1, 2, 3$, and 4) can be calculated using Eq. 6.6 as follows:

$$\rho_{d,n} = \frac{m_{d,n}}{V_n} = \frac{m_{d,n-1} - \Delta m_{d,n}}{V_{n-1} - \Delta V_n} \quad \dots (6.6)$$

where $\rho_{d,n}$, $m_{d,n}$, and V_n are the dry density, mass of the dry soil, and volume of the soil specimen at stage n , respectively. In addition, $\Delta m_{d,n}$ and ΔV_n are the changes in the dry mass of the soil migrated and volumetric deformation of the soil specimen, respectively, during stage n .

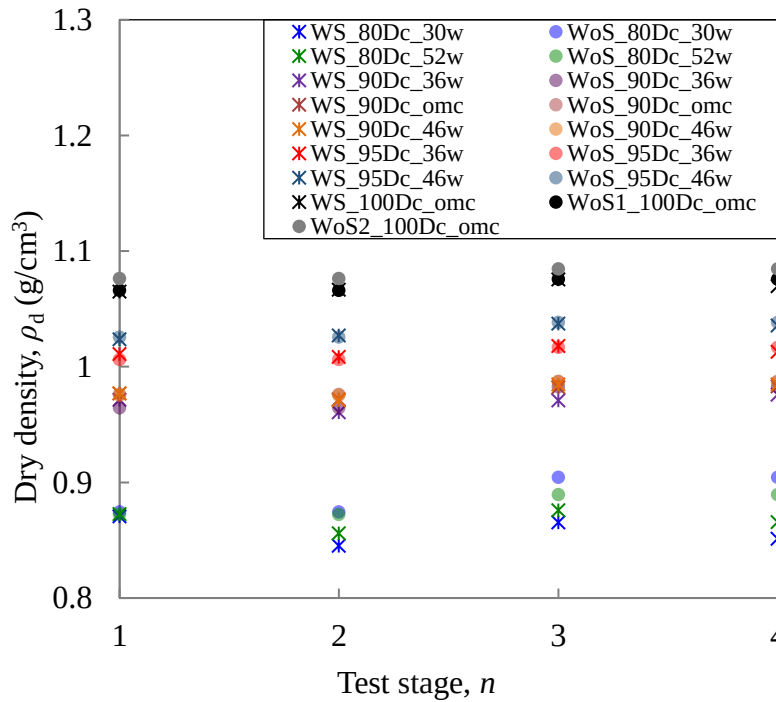


Fig. 6.10. Variation in dry density of specimens at different stages of experiment.

The loss of a major fraction of fines during saturation leads to a decrease in ρ_d of the WS-series specimens after saturation, except for the specimens with $D_c \approx 95\%$ and 100% . A subsequent rise in ρ_d is observed in all specimens after consolidation. As seen in Fig. 5.2, the specimens on the dry side erode more than the specimens on the wet side at similar densities. This results in a drop in ρ_d during the saturation stage in the WS-series specimens on the dry side and a minor increase in ρ_d in the specimens on the wet side, except for the $D_c \approx 80\%$ specimens. Finally, all the WS-series specimens have a lower or equivalent final ρ_d right before shearing than the WoS-series specimens. It is evident, therefore, that densification did not occur after suffusion and that it does not play a role in the change in strength in suffusion-induced densely compacted pumice sand.

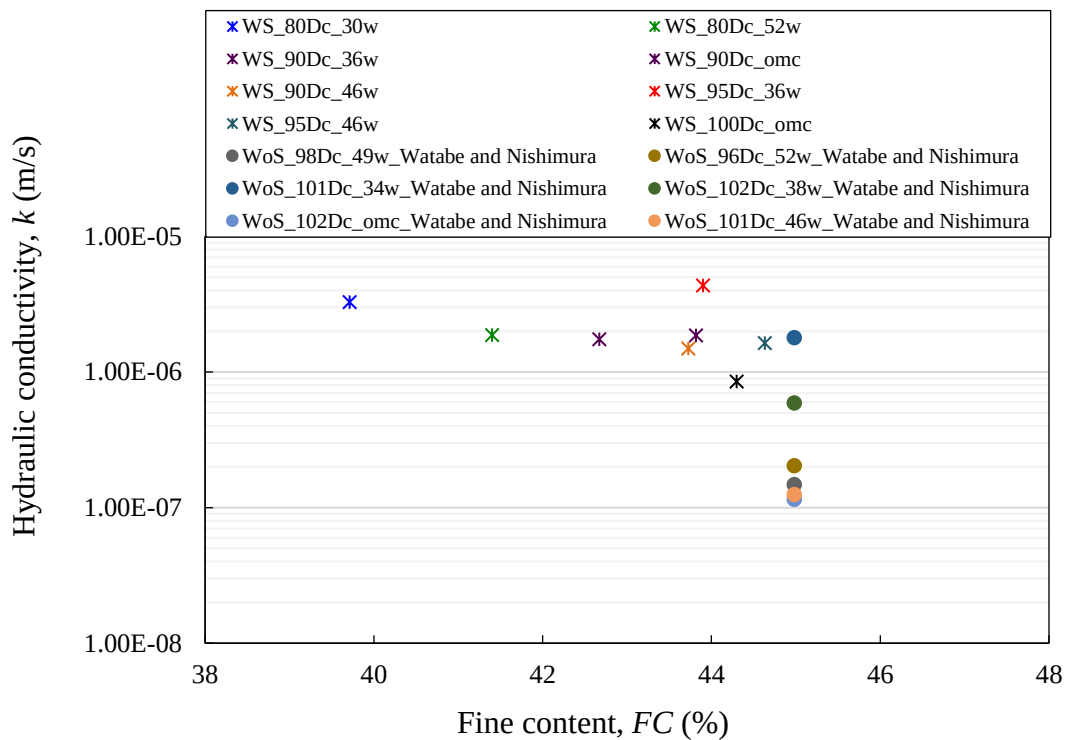


Fig. 6.11. Relationship between hydraulic conductivity and fines content at different degrees of compaction.

With the migration of fines during suffusion, the particle size ratio (Wiacek and Stasiak, 2018) of the soil decreases due to the comparatively large size and high angularity of the coarse sand particles (Fig. 3.7), which causes the subsequent increase in the void ratio of the compacted soil (Yilmaz et al., 2023). In addition, the higher value of hydraulic conductivity for the specimens with suffusion and a lower fines content, in comparison to the value of hydraulic conductivity for the specimens without suffusion and no change in the fines content (Watabe and Nishimura, 2020), also signifies an increase in pore volume (Fig. 6.11) due to suffusion. Tsomokos and Georgiannou (2010) stated that grain shape and angularity have a positive effect on the undrained shear behavior of sand by increasing the undrained shear strength and exhibiting stable behavior. Therefore, the strength enhancement of suffusion-induced densely compacted pumice sand can be attributed to the alteration of the soil microstructure in the form of the omission of less angular fines, a change in pore volume, and the rearrangement of the highly angular grains brought about by the combined action of hydraulic force and effective stress. This resulted in a decrease in the ratio of rolling to sliding between particles, and a simultaneous improvement in the interlocking of the angular coarse grained particles, thus ensuring a more efficient stress transfer through the soil skeleton made up of coarse grains and greater shear resistance (Mair et al., 2002), as shown in Fig. 6.12. The process of change in soil microstructure during the complete experiment is summerized as a flow chart in Fig. 6.13. This change in the soil microstructure correspondingly caused a change in the shearing resistance angle at $q_{\max}$ ($\phi_{q\max}$), with the maximum shear resistance angle ($\phi_{\max}$) finally influencing the stress-strain response and stress-dilatancy relation (Yilmaz et al., 2023) which concur with the observations made in this study.

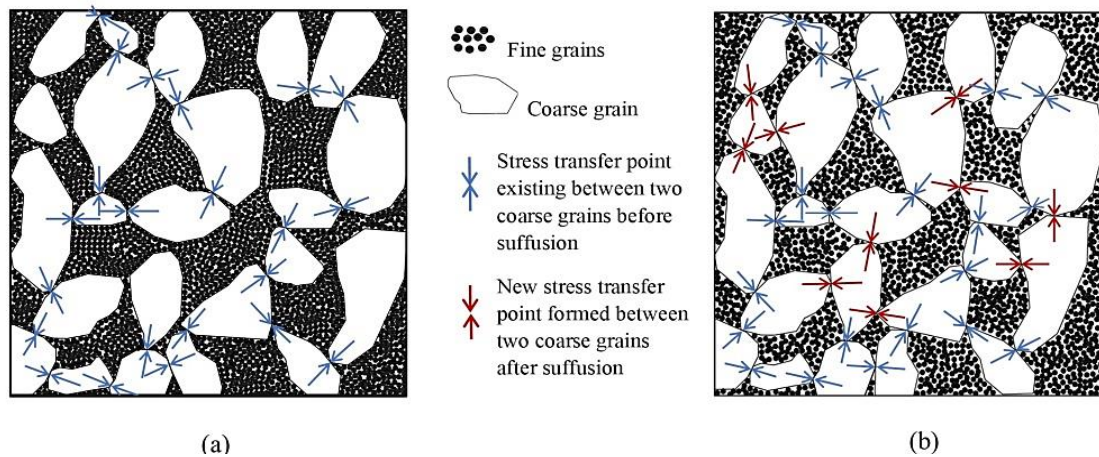


Fig. 6.12. Changes in soil microstructure and stress transfer mechanism (a) before suffusion, and (b) after suffusion.

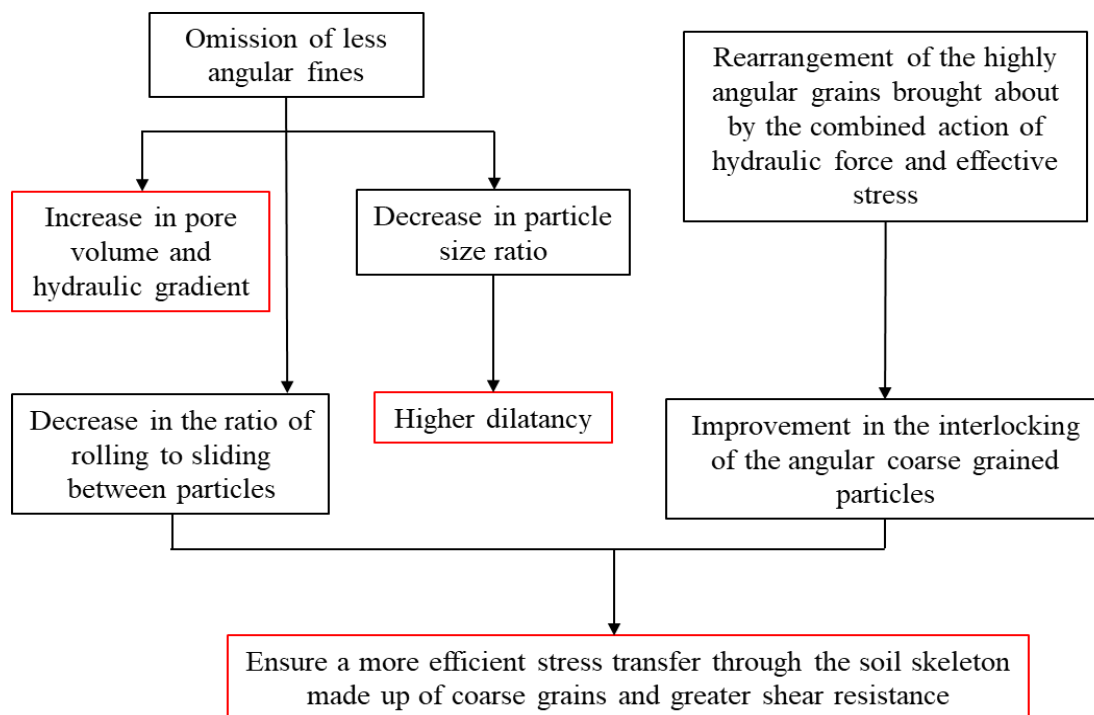


Fig. 6.13. Process of change in soil microstructure.

Chapter 7

CONCLUSIONS AND PERSPECTIVE

7.1 Conclusions

Using a modified triaxial setup equipped with the capability to induce suffusion, an extensive series of consolidated undrained triaxial shear tests ($\overline{CU}$ tests) were performed on well-graded densely compacted pumice sand. This volcanic sand especially contains a significant proportion of non-plastic fines (45%). The comprehensive analysis of the ($\overline{CU}$ tests) has led to several noteworthy inferences as mentioned below regarding the interplay between suffusion and the mechanical properties of the Satozuka pumice sand. These findings contribute to the broader understanding of how suffusion, in the context of well-graded and densely compacted pumice sand with a notable presence of non-plastic fines, shapes the material's response to external stresses. The conclusions are:

1. Assessment of suffusion susceptibility after the characterization of pumice sand using US Army criteria, Instomina criteria, Kenney and Lau's method and Burenkova's criteria reflected its internal instability in spite of the sand's well-graded nature.
2. The extent of suffusion increases with a decrease in the degree of compaction D_c during the specimen preparation due to changes in the constriction size and inter-particle grain to grain contact. Furthermore, for equally compact specimens, the degree of saturation S_r (or moisture content) during the specimen preparation adversely affects the extent of suffusion due to the

formation of an inter-pore network called macro-porosity in specimens with low saturation.

3. Suffusion increases the hydraulic conductivity, k , of pumice sand by enlarging the flow path (pore diameter of the bottleneck) due to the removal of fines.
4. The increase in residual shear strength under undrained shearing for specimens with suffusion indicates that there is no significant negative impact brought about by suffusion on the deterioration of densely compacted earth fill constructed with pumice sand having a high fraction (approximately 45%) of non-plastic fines. This tendency is consistent for a variety of D_c from 80% to 100%. It is noteworthy that the initial D_c has a significant impact on the absolute value of the maximum deviator stress as the increase in density is accompanied by an exponential increase in normalized maximum deviator stress. The normalized maximum deviator stress at the very high density of $D_c > 95\%$ becomes highly sensitive to a minute variation in density as a consequence of this exponential increase and high strength. Densely packed specimens with $D_c > 90\%$ show negligible brittleness irrespective of the suffusion condition. Thus, suffusion exhibits no pronounced influence on the brittleness index.
5. The slope of the ESP at $q_{\max}$, $M_{q_{\max}}$, and the shear resistance angle at $q_{\max}$, $\phi_{q_{\max}}$, exhibit higher values for specimens with suffusion up to $D_c \leq 90\%$ and a marginal reduction or no change at all to these values for specimens with suffusion at $D_c \geq 95\%$. In addition, the slopes of the ESP and the shear resistance angles show increments with a rise in density for the specimens both with and without suffusion.

6. The dilation tendency of densely compacted pumice sand is enhanced by suffusion. During undrained shear, the specimens undergoing suffusion reach the phase transformation point at a lower axial strain ($\epsilon_{a,PTP}$) than the specimens without suffusion, indicating that the change in volume from contraction to dilation (or phase transformation) occurs earlier in specimens with suffusion.
7. The omission of less angular fines due to suffusion resulted in rearrangement of highly angular soil grains brought about by the combined action of hydraulic force and effective stress. This resulted in a higher hydraulic gradient; decrease in the ratio of rolling to sliding between particles, and a simultaneous improvement in the interlocking of the angular coarse-grained particles, thus ensuring a more efficient stress transfer through the soil skeleton made up of coarse grains and a greater shear resistance. Moreover, the changes in particle size ratio due to omission of fines have resulted in increase in dilatancy.

7.2 Recommendations

This comprehensive study digs into the contemporary and a practical problem of suffusion which can be harmful to earthen constructions. Through rigorous experimentation and detailed analysis, it sheds light on the significant implications of suffusion on pumice sand; which is a type of well graded soil of volcanic origin and contains a high fraction of fines. The findings and observations from this research serve as a crucial contribution to the broader understanding of the influence of suffusion on densely compacted pumice sand, offering valuable insights to researchers and engineers regarding its mechanical behavior for further exploration and practical

applications in design and construction of earthen structures on volcanic soil. This study will also assist engineers in estimating the strength of densely compacted grounds made of pumice sand, whether with or without suffusion. This area has not been extensively studied to date.

In this study, it is evident that well-graded pumice sand with a high fine content is prone to suffusion, even when compacted to a dense state. However, this susceptibility does not adversely affect its mechanical behavior. Therefore, the recommended compaction condition, with $D_c > 95\%$ as per current industrial practice standards, proves suitable for this soil type in spite of its suffusion potential.

Moreover, observations indicate that soil specimens at the optimum moisture content (OMC) or on the wet side of OMC exhibit enhanced resistance to suffusion and possess higher shear strength. Hence, it is advisable to compact this soil type at OMC or on the wet side of OMC to optimize its performance and achieve maximum benefits.

The fine content of volcanic soils in the Hokkaido region varies across different locations. When soil with a low fine content is compacted to an extremely high density, its fine content tends to increase due to particle crushing from repeated compaction. This phenomenon can result in unexpected mechanical behavior of pumice sand with initially low fine content. Therefore, maintaining a higher fine content is ideal to prevent particle crushing in pumice sand and avoid unforeseen changes in its mechanical characteristics.

Additionally, presence of well graded volcanic soil similar to pumice sand is not limited to the Satozuka district of Sapporo city only. This type of pyroclastic flow deposit (Spfl) is available in the other countries such as Italy, Indonesia etc. as well;

thus, making this study an example of a likely global problem applicable to other parts of the world as well.

7.3 Scopes for Future Study

The suggestions for future work, which can help improve further understanding of the mechanical behavior of suffusion induced pumice sand are as follows:

1. Liquefaction occurred in the area under study after the Eastern Iwate earthquake in 2011. Also, the soil at low density displayed static liquefaction behavior under monotonic loading triaxial test. Therefore, the understanding of the cyclic loading behavior of the soil under consideration can provide fruitful insight regarding influence of suffusion on liquefaction resistance and post cyclic shear strength of volcanic soil.
2. Studying the influence of suffusion on mechanical properties of volcanic soil with varying fine contents can offer more insights into this topic and assess the suitability of such soils under controlled loose and dense packing.
3. Detailed study on the influence of grain morphology of pumice sand can aid in better interpretation of the role of fine content in changes in soil microstructure due to suffusion.
4. Modification of the experimental set up is recommended to measure the instantaneous volume change during suffusion.
5. The test procedure can be modified to determine the hydraulic conductivity of the specimens instantaneously under the action of effective stress.

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LIST OF PUBLICATIONS

This dissertation is a modified and revised form of the following original journal article and conference proceeding-

1. Sarmah, R., Watabe, Y., 2023. Suffusion in densely compacted Satozuka pumice sand and its impact on static loading undrained shear strength and dilation behaviour. Soils and Foundations, 63(6), 101397.
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2. Sarmah, R., Kubota, S., Watabe, Y., 2021. Influence of suffusion on shear strength and dilatancy characteristics of pumice sand. Proceedings of the 10th International Conference on Scour and Erosion (ICSE-10) by ASCE, Arlington, Virginia, USA, 1034-1043.
https://www.issmge.org/uploads/publications/108/109/Influence_of_Suffusion_on_Shear_Strength_and_Dilatancy_Characteristics_of_Pumice_Sand.pdf