Soil deformation due to suffusion and its consequences on undrained behavior under various confining pressures

Lin Kei, Mao Ouyang, Kazuki Horikoshi and Akihiro Takahashi

i) Postdoctoral Researcher, Dept. of Civil Engineering, Tokyo Institute of Technology, 2-12-1, O-okayama, Tokyo 152-8552, Japan.
ii) Ph.D Student, Dept. of Civil Engineering, Tokyo Institute of Technology, 2-12-1, O-okayama, Tokyo 152-8552, Japan.
iii) Ph.D Student, Dept. of Civil Engineering, Tokyo Institute of Technology, 2-12-1, O-okayama, Tokyo 152-8552, Japan.
iv) Professor, Dept. of Civil Engineering, Tokyo Institute of Technology, 2-12-1, O-okayama, Tokyo 152-8552, Japan.

ABSTRACT

Suffusion, defined as the phenomenon whereby the fines gradually migrate through the voids of coarse fractions in a soil, has been widely detected in natural deposits and artificial earth structures. The occurrence of suffusion may chronically loosen the soil structure, increasing the vulnerability against large deformation and soil failure. In this paper, experimental studies on volume change of saturated gap-graded cohesionless soil during suffusion and its mechanical influence on undrained behavior are presented. Test results reveal that because of the loss of large amounts of fines during suffusion, volume of tested soil decreases and void ratio increases. Correspondingly, a distinctive undrained behavior of suffusional soil is noted from that of the reference soil without suffusion.

Keywords: suffusion, undrained behavior, volumetric strain

1 INTRODUCTION

The internal extraction of soils accompanied with seepage flow, commonly known as internal erosion, is frequently detected at natural soil deposits and earth structures, especially dams and embankments. Two fundamental types of internal erosion could be further identified regarding how the soil particles are dislodged: piping and suffusion. “Piping” refers to the phenomenon that underground water flows along a continuous opening, such as crack, and simultaneous dislodgement of soils occurs along the wall of the “pipe”, whereas “suffusion” describes the migration of a portion of the finer fraction of a soil through its coarser fraction. This paper concentrates on the latter. Hicher (2013) noted that the probability of failure of a dam induced by internal erosion is approximately more than 10 times that of failure by sliding. The occurrence of such soil failure might be increasing frequently in the ageing earth structures. Moreover, with the transportation of fines, suffusion may render a much looser soil structure and the strength of suffusional soil may reduce consequently. A telling example involves the flow slide of embankments constructed on catchment topography, such as swamps and valleys which usually are accompanied by a large volume of fresh water, during Noto Peninsula Earthquake of Japan in 2007 (Sugita et al., 2008). It is inferred that years of suffusion may chronically loosen the soil and consequently, the earth structure becomes vulnerable to seismic shaking.

Catastrophic though the consequences of suffusion are, unfortunately, the mechanical behavior of suffusional soil has been insufficiently understood. Chang and Zhang (2011) noted that the drained strength of soil decreased after suffusion, whereas Xiao and Shiwiyhat (2012) showed that the post-suffusion undrained peak strength became larger. Furthermore, Chang and Meidani (2012) indicated that the mechanical behavior of suffusional soil seemed dependent on the confining pressure when suffusion occurred. Inheriting the existing studies, this paper introduces the influence of effective confining pressures on the volumetric strain during the progress of suffusion and the associated post-suffusion mechanical behavior of cohesionless soil by experimental investigations in a revised triaxial permeameter. Efforts are devoted to demonstrating the changes of undrained behavior after suffusion under various effective confining pressures.

2 EXPERIMENTAL INVESTIGATIONS

The experimental investigations are conducted by a modified triaxial permeameter, which is capable of directly investigating not only the mechanism of...
suffusion but also the corresponding mechanical behavior of suffusional soil. The apparatus mainly consists of a constant-flow-rate control unit, an automated triaxial system, and an eroded soil-collection unit. The whole system realizes independent control of the hydraulic conditions and the stress state of tested specimens. To keep the fully saturated condition of tested soil, back pressure is maintained during suffusion.

It is recognized that gap-graded soils, like sandy gravels or silty sands, are especially vulnerable to suffusion due to its deficiency in certain particle size. In this study, the tested specimens are the binary mixtures of Silica sand No.3 and No.8. Sub-rounded to sub-angular Silica sand No.3 ($D_{50}=1.83\text{mm}$, $C_{u}=1.47$, $G_{s}=2.65$) constitutes the soil skeleton while the fine Silica No.8 ($D_{50}=0.12\text{mm}$, $C_{u}=1.72$, $G_{s}=2.65$) is the erodible fines, where $D_{50}$ is median particle size (mm), $C_{u}$ refers to uniformity coefficient, and $G_{s}$ indicates specific gravity. A fines content (mass ratio of Silica No.8 to the binary mixture) of 35% is adopted so that the binary mixture is internally unstable ($(D_{15c}/d_{50})_{35\%}=7.9>4$, where $D_{15c}$ is the particle size at 15% by passing of Silica No.3 (mm) and $d_{50}$ refers to the particle size at 85% by passing of Silica No.8 (mm)) assessed by Kezdi’s method (Kezdi, 1979) and vulnerable to suffusion if seepage flow is assigned. The particle size distribution is shown in Fig. 1.

A summary of the test specimens is shown in Table 1. Overall, the test program includes soil preparation, vacuum saturation, isotropic consolidation, seepage test and compression test. The tested specimens are prepared by the moist tamping method and the tamping on each specimen is in a systematic manner to guarantee an identical input energy. The mean effective stress considered in the isotropic consolidation is 50kPa, 100kPa and 200kPa, which approximately corresponds to the earth pressure of 5m, 10m and 20m in depth, respectively. In average, the relative density of the tested specimens after consolidation is about 47%.

Some of the specimens will first experience suffusion during seepage tests to create the suffusional specimens for further compression while the rest specimens without suffusion are the reference soil for the comparison purpose. Seepage tests are performed at the stress state the same as that of the specimen after isotropic consolidation. To demonstrate the mechanical effects of suffusion systematically, the imposed inflow rate for each specimen keeps constant as $5.17 \times 10^{-6}\text{m}^3/\text{s}$. The seepage tests are usually terminated after 3 hours. At most circumstances, the post-suffusion B-value is larger than 0.93 because of the maintenance of back pressure. After the seepage test, a series of monotonic compression test is performed on the suffusional soil without changing the cell pressure and the back pressure to investigate the mechanical consequences of suffusion. The compression test is displacement controlled with an axial strain rate of 0.1%/min to allow pore pressure to reach equilibrium. Further information regarding the permeameter and test procedures would be referred to Ke and Takahashi (2014).

### 3 VOLUMETRIC DEFORMATION OF SOIL DURING SUFFUSION

During suffusion, large amounts of fines would be dislodged away and corresponding volumetric strain occurs. Change of void ratio is caused by the fines loss and possible particle re-arrangement, which hypothetically follows two steps: (1) as soon as suffusion initiates, no deformation occurs due to the dislodgement of fines and the volume of eroded fines is replaced by water on condition that soil is fully saturated. $\varepsilon'$ indicates the void ratio after erosion of fines without soil deformation, which can be given by:

$$\varepsilon' = (\varepsilon_0 + \Delta FC) / (1 - \Delta FC)$$  \hspace{1cm} (1)

where $\Delta FC$ refers to fines loss during suffusion (%);

(2) After the loss of large amounts of fines, a metastable structure might be formed and re-arrangement of remaining particles may occur, causing volumetric deformation and changes in void ratio. Then, the post-suffusion void ratio ($\varepsilon_v$) is:

$$\varepsilon_v = \varepsilon' - \varepsilon_0 (1 + \varepsilon') = (1 - \varepsilon_0) (\varepsilon_0 + \Delta FC) / (1 - \Delta FC) - \varepsilon_0$$  \hspace{1cm} (2)

where $\varepsilon_v$ indicates volumetric strain during suffusion in percentage.

The relation above indicates the dependence of post-suffusion void ratio on the volumetric strain during suffusion. Under the circumstance where the volume of soil expands, the largest void ratio would be obtained, whereas a contractive deformation may inhibit the tendency of the increasing of void ratio even diminish the void ratio after suffusion. As for confining pressure, it may influence the amounts of fines loss, thereby affecting the volumetric strain and the post-suffusion void ratio. In this study, the influence of the initial effective confining pressure on the cumulative eroded
soil loss during suffusion is noted in Fig. 2, indicating a less fines loss under the larger initial effective confining pressure. It can be explained that at the larger confining pressure, the fines are expected to be packed among coarse particles densely and the interstitial spaces may be narrower. Thus, the seepage flow might transport fewer fines. The corresponding volumetric strain and post-suffusion void ratio under different confining pressures are summarized in Fig. 3 and Table 1. The tested specimens show the tendency of decreasing in volume during suffusion and less volumetric strain occurs under the larger initial effective confining pressure. Correspondingly, the void ratios of tested specimens significantly increase after suffusion, suggesting that suffusion causes the loss of large amounts of fines with limited changes in the void structure of the coarse fractions. Under larger initial effective confining pressure, less increments in void ratio is noted. Because of the increasing of void ratio and the declining of fines content, the mechanical behavior of suffusional soil may be distinctive from the reference soil.

### 4 UNDRAINED RESPONSES OF SUFFUSIONAL SOIL

The post-suffusion intergranular void ratios ($e_s$) are similar (Table 1), which is considered as the comparison base for interpreting the data. The “intergranular void ratio” refers to the voids between coarse particles by regarding the volume of fines as a part of the voids. The differences in the undrained responses of suffusional specimens are mainly caused by the differences in the confining pressure during suffusion and the packing of soil particles formed after suffusion. Figure 4 presents the undrained responses of the suffusional specimens in terms of stress ~ strain curves and effective stress paths, respectively. Generally, the deviator stress of the suffusional specimens reaches a marked peak at low axial strain, approximately 1%, followed by the temporary strain softening and then dilating at the subsequent compression. Figure 5 plots the mobilized effective friction angle at peak against the initial effective confining pressure, indicating a smaller mobilized friction angle observed under the smaller initial effective confining pressure. It may be explained that because of the larger void ratio after suffusion under the smaller initial effective confining pressure, the particle interlocking of suffusional soil becomes weaker and therefore the soil exhibits less friction.

![Diagram](image1)

![Diagram](image2)

![Diagram](image3)

Table 1. A summary of the properties of tested specimens.

| Specimens | $FC_0$ (%) | $e_0$ | $p'_0$ (kPa) | $e_c$ | $FC_e$ (%) | $e_e$ | Type of compression |
|-----------|------------|-------|--------------|-------|------------|-------|--------------------|
| 35E-50(1) | 35.0       | 0.59  | 50           | 0.56  | 13.1       | 0.99  | 1.3                | Undrained |
| 35E-100   | 35.0       | 0.60  | 100          | 0.55  | 15.9       | 0.92  | 1.3                | Undrained |
| 35E-200   | 35.0       | 0.59  | 200          | 0.55  | 22.0       | 0.80  | 1.3                | Undrained |
| 35N-50(1) | 35.0       | 0.60  | 50           | 0.56  | --         | --    | 1.4                | Undrained |
| 35N-100   | 35.0       | 0.60  | 100          | 0.54  | --         | --    | 1.4                | Undrained |
| 35N-200   | 35.0       | 0.59  | 200          | 0.54  | --         | --    | 1.4                | Undrained |

Note: (1) Those specimens named with “E” mean suffusional soil while those named with “N” means reference soil without suffusion; (2) $FC_0$ ($FC_e$): fines content before (after) suffusion (%); (3) $e_0$: initial void ratio prior to consolidation; (4) $p'_0$: initial effective confining pressure (kPa); (5) $e_c$ ($e_e$): void ratio before (after) suffusion; (6) Intergranular void ratio $e_s=(e_e+FC_e/100)/(1-FC_e/100)$ (suffusional soils) or $(e_c+FC_0/100)/(1-FC_0/100)$ (reference soils).
The state at which the undrained deviator stress reaches a local minimum is named as quasi-steady state. To evaluate the liquefaction potential of the suffusional specimens with different initial effective confining pressures quantitatively, a plot of $q_{ss}/q_{peak}$ against initial effective confining pressure is also shown in Fig. 5 ($q_{ss}$ ($q_{peak}$): deviator stress at quasi-steady state (peak state) (kPa)). A $q_{ss}/q_{peak}$ of zero indicates complete drop of deviator stress to zero at the quasi-steady state, whereas that of unity stands for a stable soil behavior. It is seen that the larger initial effective confining pressure results in the larger shear stress ratio, indicating greater stability. For comparison, Fig. 6 shows the undrained response of reference soil, i.e. uneroded soil. The reference specimens reach a peak at low axial strain, approximately 1% ~ 1.5%, followed by strain softening. The after-peak deviator stress maintains constantly without the sign of further dilatancy.

The expected mechanical consequences of suffusion include the potential changes of undrained behavior which commonly consists of peak state, quasi-steady state and steady state for granular soils. Because of the dislodgement of fines and re-arrangement of remaining particles during suffusion, the initial fabric of suffusional specimens might be fundamentally altered. Wan and Guo (2001) concluded that different initial fabrics may influence the undrained response of sand at least up to the quasi-steady state. Thus, the mechanical influence of suffusion could be reflected by the changes of soil responses at peak and quasi-steady state. The relation of mobilized effective friction angle at peak and initial effective confining pressure is indicated in Fig. 7. Both suffusional and reference specimens exhibit a larger mobilized effective friction angle at peak under a larger initial effective confining pressure. A postulated tendency line (the dashed line) for the reference specimens with $e_o$ of 1.3, which might be somehow in parallel to that of the reference specimens with $e_o$=1.4, is drawn in order to compare the responses at peak between the suffusional and reference specimens. As is shown, the suffusional specimens ($e_o$=1.3) show a relatively smaller difference in friction angle at peak for $p_0'=50$ kPa, but a larger difference for $p_0'=200$ kPa. Since the peak state (within 1% axial strain) is expected to be affected by the initial packing of soil, the changes in friction angle under different initial effective confining pressures comparing to that of reference soil should be associated with the change of the packing of soil particles after suffusion. For $p_0'=200$ kPa (specimen 35E-200) before suffusion, the fines near the contacts of coarse particles may probably participate in force chains and suffusion may not cause much movement of those active fines, whereas the rest fines without engagement of the force chains would probably be dislodged away. Consequently, void ratio of soil increases after suffusion and hence a larger difference in friction angle at peak is noted. In terms of
particles, thereby strengthening the soil. Its effect might surpass the contractive tendency induced by the increase in the void ratio after suffusion and smaller differences in friction angle at peak occur.

For fines-containing soil, Ishihara (1996) noted it may develop amounts of deformation while keeping the magnitude of deviator stress at the lowest level at quasi-steady state under undrained conditions and that deviator stress is regarded as the representative soil strength, named as residual strength \( \tau \)

\[
\tau = \frac{q' \cos \phi_v}{2} \tag{3}
\]

where \( \phi_v \) refers to the gradient of phase transformation line at quasi-steady state (°).

The reference specimens do not exhibit the quasi-steady state but keep at a constant deviator stress with further straining. At this circumstance, this constant value is utilized to determine the residual strength. A plot of the normalized residual strength (normalized with \( p_0' \)) against initial effective confining pressure is plotted in Fig. 8. Within the test range, the residual strength of suffusional specimens is larger than that of the reference specimens. An interesting point is the nonzero intercept in the vertical axis for the suffusional specimens. Ishihara (1996) noted that the normalized residual strength was proportional to the initial effective confining pressure for the reconstituted sands, as the reference specimens shown in the plot. However, the suffusional specimens appear to gain the strength even within the domain of very small confining pressure (i.e., nonzero intercept in the vertical axis), similar to “cohesion”, which may again suggest the existence of a reinforced soil packing after suffusion. Due to reinforced soil packing, the undrained mechanical behaviors of suffusional specimens are distinct from those of reference specimens.

![Figure 6](image6.png)
**(a) Relations of deviator stress and axial strain**

![Figure 7](image7.png)
**(b) Effective stress paths in \( p' - q \) space**

![Figure 8](image8.png)
**(c) Residual strength against initial effective confining pressure**

**5 CONCLUSIONS**

This paper experimentally investigates the
volumetric deformation of cohesionless soil during suffusion and its associated mechanical influences under different initial effective confining pressures.

Accompanied with the dislodgement of large amounts of fines, suffusion causes a decrease in the volume and an increase in the void ratio. A smaller initial effective confining pressure would raise the amounts of eroded fines leading to greater volumetric strain and void ratio. Suffusional soil behaves distinctively from the reference soil without suffusion. The suffusional soil exhibits a dilative tendency after passing through the quasi-steady state and the larger initial effective confining pressure results in the larger shear stress ratio ($q_{st}/q_{peak}$), indicating a greater stability. Comparatively, the change in frictional angle at peak is smaller under the smaller initial confining pressure, whereas it becomes larger under the larger initial confining pressure. The residual strength of the tested specimens increases after suffusion. The results that the suffusional specimens gain the strength even within the domain of very small confining pressure may be due to the existence of reinforced soil packing after suffusion.

REFERENCES

1) Chang, C.S. and Meidani, M. (2012): Deformation and failure behavior of soils under erosion, Poster of NSF CMMI Engineering Research and Innovation Conference, 0928433.

2) Chang, D.S. and Zhang, L.M. (2011): A stress-controlled erosion apparatus for studying internal erosion in soils, Geotech. Test. J., 34 (6), 579-589.

3) Hicher, P.-Y. (2013): Modelling the impact of particle removal on granular material behaviour, Géotechnique, 63(2), 118-128.

4) Ishihara, K. (1996): Soil behavior in earthquake geotechnics, Clarendon press, Oxford, UK.

5) Ke, L. and Takahashi, A. (2014): Triaxial erosion test for evaluation of mechanical consequences of internal erosion, Geotech. Test. J., 37 (2), 347-364.

6) Kezdi, A. (1979): Soil Physics: Selected Topics, Elsevier Sci. Ltd.

7) Sugita, H., Sasaki, T. and Nakajima, S. (2008): Damage investigation of road embankment caused by the 2007 Noto Peninsula, Japan Earthquake, PWRI Report.

8) Wan, R.G and Guo, P.J. (2001): Effect of microstructure on undrained behaviour of sands, Can. Geotech. J., 38 (1), 16-28.

9) Xiao, M. and Shwiyhat, N. (2012): Experiment investigation of the effects of suffusion on physical and geomechanic characteristics of sandy soils, Geotech. Test. J., 53(6), 1-11.