Canadian Geotechnical Journal
Revue canadienne de géotechnique

Flume-Scale Experiments on Suffusion at the Bottom of Cutoff Wall in Sandy Gravel Alluvium

Journal:	<i>Canadian Geotechnical Journal</i>
Manuscript ID	cgj-2016-0248.R3
Manuscript Type:	Article
Date Submitted by the Author:	13-Mar-2017
Complete List of Authors:	Luo, Yulong; Hohai University, College of Water Conservancy and Hydropower Engineering Nie, Min; Hohai University, College of Water Conservancy and Hydropower Engineering Xiao, Ming; The Pennsylvania State University, Civil and Environmental Engineering
Keyword:	suffusion, cutoff wall, deep sandy gravel alluvium, multi-phase and multi-field coupling

SCHOLARONE™
Manuscripts

Flume-Scale Experiments on Suffusion at the Bottom of Cutoff Wall in Sandy Gravel**Alluvium**

Yulong Luo, PhD¹, Min Nie², and Ming Xiao, PhD, PE³

¹Associate Professor, College of Water Conservancy and Hydropower Engineering, Hohai University, Nanjing 210098, China; and Visiting Scholar, the Pennsylvania State University, Department of Civil and Environmental Engineering, University Park, PA, USA. E-mail: lyl8766@hhu.edu.cn.

²Graduate student, College of Water Conservancy and Hydropower Engineering, Hohai University, Nanjing 210098, China. E-mail: nm920221@163.com.

³Associate Professor, Department of Civil & Environmental Engineering, the Pennsylvania State University, University Park, PA 16802. E-mail: mxiao@engr.psu.edu.

Corresponding author: Yulong Luo (e-mail: lyl8766@hhu.edu.cn)

Abstract: This paper presents a series of flume-scale experiments to investigate suffusion at the bottom of a cutoff wall in an internally unstable sandy gravel alluvium. The initiation, progression, and potential failure of suffusion and the interactive effects of geomechanical and hydraulic conditions with the evolution of suffusion were investigated in this particular application. Temporal and spatial development of pore pressure, earth pressure, and settlement demonstrated suffusion was a multiphase (involving pore water, fine and coarse fractions) and multi-field (involving seepage, seepage-induced fine fraction variation, and stress-deformation) coupling phenomenon. Suffusion initiated at the downstream side of the tip of the cutoff wall and then generally progressed backward to the upstream side. The monitored earth pressure provided an evidence of the heterogeneous stress distribution in internally unstable soil. Two linear empirical formulas for average hydraulic gradients at the initiation of suffusion and at blowout were derived based on the flume-scale model experiments.

Key words: suffusion; cutoff wall; deep sandy gravel alluvium; multi-phase and multi-field coupling.

Introduction

Suffusion, a type of subsurface or internal erosion, refers to the migration of fine particles in a coarser soil matrix; it can occur in the presence of widely graded or gap graded soils. With the loss of fine particles, local cavities and deformation (Fannin and Slangen 2014) can occur. Deep sandy gravel alluvium is the Quaternary unconsolidated sediment that has been accumulated in valleys and can be more than 30 m thick. It has loose structure, lithologic discontinuity, complicated genetic types, non-uniform physical and mechanical properties, and high permeability. Deep alluvium with sandy gravel can be an internally unstable soil, and dams built on such alluvium may be susceptible to suffusion. For example, the Tarbela Dam in Pakistan was constructed in a deep alluvium up to 213 m in depth. When the reservoir was first filled, 362 sinkholes were observed in the upstream impervious blanket, and the largest sinkhole had a diameter of 12.2 m and the deepest depth was 4.0 m. The sinkholes increased seepage discharge significantly, and about two hundred relief wells at the downstream toe of the dam discharged more than $0.028 \text{ m}^3/\text{s}$ per well (Ul Haq 1996). The sinkholes were considered serious threats to dam safety and resulted in an emergency drawdown of the reservoir. A geotechnical engineering panel concluded that suffusion was the main cause of the sinkholes (Ul Haq 1996). A large number of high earthen and rockfill dams have been constructed in deep alluvium, such as the Serre-Poncon dam in France, the Aswan dam in Egypt, the Mattmark dam in Switzerland, the Manic III dam in Canada, and the Yele dam in China (Dang and Fang 2011). Cutoff walls are widely used to control under-seepage of dams (Rice and Duncan 2010). Due to geometric conditions and mass

balance, the seepage velocity at the bottom of a cutoff wall may be the greatest, and it may affect the potential for suffusion. The thirty case histories of dams assembled by Rice and Duncan (2010) suggested that cutoff walls often drastically increase hydraulic gradients around the boundaries of the walls and significantly give rise to the potential of suffusion. Suffusion-induced deformation, which is termed as “suffosion” by Fannin and Slangen (2014), and sinkholes in an embankment are specific concerns of dam safety. Understanding the mechanism of suffusion in deep sandy gravel alluvium, especially around cutoff walls, is of great importance for dam safety and is the subject of this study.

The mobilization and migration of fine particles in the evolution of suffusion are affected by hydraulic and in-situ stress (Garner and Fannin 2010). The tests performed by Skempton and Brogan (1994) found that the critical hydraulic gradient that initiated suffusion in internally unstable sandy gravel was far lower than the critical gradient given by Terzaghi (1925). They inferred that a major portion of the overburden load was carried by gravel particles, leaving the sand under relatively small pressure. Sterpi (2003) and Cividini and Gioda (2004) investigated the influence of hydraulic gradient on suffusion and established empirical relationships expressing the quantity of eroded particles with hydraulic gradient and time. Bendahmane et al. (2008) designed a triaxial suffusion apparatus and performed a series of experiments to investigate the effects of hydraulic gradient, clay content, and confining pressure on the evolution of suffusion. In order to simulate the real stress state of soil, Richards and Reddy (2010, 2012) developed a true triaxial apparatus for suffusion experiments, and they found that hydraulic gradient was a less reliable indicator than exit

velocity for initiation of internal erosion of non-cohesive soils. The spatial and temporal progressions of internal instability were studied by Moffat et al. (2011), and a novel concept of a hydro-mechanical path in the stress-gradient space was proposed (Moffat and Fannin 2011). The effect of stress state on critical hydraulic gradient for suffusion was investigated by Chang and Zhang (2011, 2013a), and three critical gradients termed as initiation, skeleton-deformation, and failure hydraulic gradients were defined. Li and Fannin (2012) found that suffusion was related to hydraulic gradient and effective stress in the finer fraction of a grain size distribution. The above studies indicate that stress state and hydraulic condition are two important factors for suffusion. Meanwhile, due to the scale constraint of triaxial suffusion experiments, the spatial variations of pore pressure and effective stress in the soil and their interactions with the evolution of suffusion have not been thoroughly investigated.

The erosion and migration of fine particles by suffusion may also change the mechanical properties of soil. The stress-strain behavior in the evolution of suffusion was studied by Chang and Zhang (2011), and it was found that the stress-strain behavior under drained compression tests changed from dilation to contraction after a significant loss of fine particles. Xiao and Shwiyhat (2012) observed that gap graded soils tended to produce more pronounced physical and geo-mechanical changes than poorly graded soils when subjected to seepage, and suffusion resulted in changes in permeability, compressive strength, and volume of soils. The variation of shear strength due to suffusion was investigated by Ke and Takahashi (2012), and they found that there was a significant reduction in shear strength after

a loss of 3–4% of the fine particles. A number of monotonic and cyclic compression tests on post-suffusion soil specimens were also conducted by Ke and Takahashi (2014a, 2014b) and they concluded that suffusion may reduce drained compressive strength. Several numerical models have also been developed (e.g., Muir Wood et al. 2010; Scholtès et al. 2010; Hicher 2013), and the numerical results showed that shear strength decreased with the removal of fine particles. These previous studies have provided improved understanding of the mutual effect of suffusion and the geotechnical properties of bench-scale soil specimens. In the application of cutoff walls in deep alluvium of sandy gravel that may be susceptible to suffusion, the relationships among seepage field, soil matrix, and geomechanical properties in the evolution of suffusion remain a knowledge gap.

This paper presents experimental research on suffusion at the bottom of a cutoff wall in an internally unstable alluvium consisting of sandy gravel. The research is to quantify the interactive correlations of pore pressure, effective stress, and settlement with the evolution of suffusion.

Methodology

Characteristics of deep alluvium of sandy gravel

Figure 1 depicts the grain size distribution (GSD) of the deep alluvium of sandy gravel used in this research. It was an internally unstable soil according to several geometric criteria

(Kenney and Lau 1985; Wan and Fell 2008; Li and Fannin 2008; Chang and Zhang 2013b). The assessment of the soil's internal stability using these criteria is shown in Table 1. Wan and Fell (2004) found that the diameter of the largest fine particle of broadly graded soils could be found at the point on the GSD with $(H/F)_{\min}$, which was defined by Kenney and Lau (1985). Using these criteria and based on the GSD in Figure 1, for the sandy gravel used in this study, the largest fine particle was 0.7 mm, and the corresponding percentage of fine particles was about 19 %, which was lower than the critical fine content (29 % for loose state or 24 % for dense state) postulated by Kenney and Lau (1985), and the particles greater than 0.7 mm constitute primary fabric, which mainly supports loads and transfers stresses (Kenney and Lau 1985). This open graded sandy gravel had large internal voids that fine particles could enter.

The uniformity coefficient and curvature coefficient were 83.3 and 3.3, respectively. The minimum and maximum dry densities determined by ASTM D4253-00 and D4254-00 were 1.83 g/cm^3 and 2.54 g/cm^3 , respectively. Based on ASTM D2434-68, the permeability of the soil without suffusion ranged from 1.6×10^{-3} to $4.5 \times 10^{-3} \text{ cm/s}$. The GSD of the sandy gravel in this paper was in the range of the GSDs of the similar alluvium soils studied in the relevant literature (Kenney and Lau 1985; Skempton and Brogan 1994; Wan and Fell 2008; Moffat et al. 2011).

Flume-scale suffusion apparatus

This paper is mainly focused on the evolution of suffusion at the tip of a cutoff wall in a deep alluvium as shown Figure 2, instead of along the entire cutoff wall. In the field condition, a concrete slab is typically used beneath the clay core to prevent the clay from penetrating into the alluvium. To simulate this situation, a flume-scale suffusion apparatus was designed and constructed. As depicted in Figure 3, the apparatus consisted of an axial loading system, a soil container, a seepage pressure system, a funnel-shaped drainage system and a data acquisition system.

The axial loading system was used to simulate the overburden pressure acting on a deep alluvium, such as from a high earthen and rockfill dam. The loading system consisted of a reaction frame, a jack and a steel loading plate. It should be noted that the steel loading plate was used to simulate the concrete slab in Figure 2. Although air bladder can apply uniform pressure on the soil specimen, it does not allow the measurement of vertical soil settlement that may be induced by suffusion; it also does not simulate the actual stress distribution under a rigid concrete slab in the field. The loading system can provide a maximum pressure of approximate 3.3 MPa.

The seepage pressure system, as shown in Figure 3, provided the driving mechanism for fine particle migration. It consisted of a pressurized water source, a constant-head water tank with adjustable height, an inlet pipe, an upstream catchment basin, an upstream porous plate with 5 mm pore opening size, a control gauge for outlet pressure, and an outlet pipe. It had mechanisms of applying two ranges of hydraulic head: a high hydraulic head mode (3

m<h<50 m) and a low hydraulic head mode ($h<3$ m). The former was realized by the pressurized water source, and the latter was controlled by raising the water tank.

The funnel-shaped drainage system was comprised of a downstream perforated plate with 5 mm pore opening size, a downstream catchment basin and a funnel-shaped outlet. The eroded particles can pass through the downstream perforated plate into the funnel-shaped outlet freely and were collected in the catchment basin. Pore opening size of the downstream perforated plate may have significant influence on suffusion. In previous studies (such as Moffat et al. 2011; Chang and Zhang 2013a; Ke and Takahashi 2014b), the ratio of the pore opening size to the largest fine particle, R_p , ranged from 6.7 to 28.5. In this study, the largest fine particle was 0.7 mm by adopting the criteria by Wan and Fell (2004); therefore, $R_p=7.1$ and it can be concluded that the adopted pore opening size of the downstream perforated plate was suitable.

The data acquisition system consisted of a vertical displacement transducer to measure the specimen deformation during suffusion, a pressure transducer to record the overburden pressure, five earth pressure transducers with diameter of 28 mm embedded in the specimen to measure the vertical earth pressure (see Figure 4, denoted by “T”), and pore pressure measurements at 24 locations in the specimen (see Figure 4, denoted by “C”). Generally a large earth pressure transducer relative to the largest particle (60 mm) of the soil in this research can help reduce the scale effect of measurement, yet such large transducer may induce contact erosion around it, which may significantly influence experimental results. Li

and Fannin (2012) found that suffusion was related to the effective stress in the finer fraction of a grain size distribution. Therefore, compared with a large transducer, a small one is more likely to measure the stress transferred by fine fraction rather than that transferred by coarse fraction. The 24 pore pressure probes and plastic tubes were installed in the soil specimen and connected to the pore pressure transducers located outside of the specimen. The system can monitor the variations of displacement, pore pressure, and earth pressure during the entire process of an experiment. Flow rate was determined by measuring the volume of effluent with time, the measure precision was 0.1 ml/s.

Specimen preparation and instrumentation configuration

Figure 4 shows the vertical section view of the test specimen and the instrumentation configuration. The test setup was specifically designed to simulate the seepage condition at the bottom of a cutoff wall and to monitor the complex variations of pore pressure and earth pressure in the same region.

A clay layer was placed along the upstream and downstream sidewalls, respectively, to create impermeable barriers, so that seepage flowed over the top of the upstream clay barrier, passed under the cutoff wall, and exited over the top of the downstream clay barrier. This simulated a flow condition around the cutoff wall in the field. The clay barriers were 300 mm long, 340 mm high, and 50 mm thick. In addition, geotextiles were placed at the interfaces between the clay layers and the sandy gravel to prevent contact erosion. The sandy gravel was compacted

in the apparatus in layers (with 5 cm thickness per layer) at a water content of 5 % to reach a dry density of 2.36 g/cm^3 , which was consistent with the values in Wan and Fell (2004). After the compacted soil specimen reached the bottom elevation of the cutoff wall, the cutoff wall was slid into the specimen container along two rubber-lined grooves on the sidewalls in order to minimize side-wall leakage around the cutoff wall during a suffusion test. The cutoff wall was simulated by a steel plate that was 10 mm thick, 300 mm long, and 400 mm deep. Compaction of the soil continued with the cutoff wall in place until the top of specimen reached the top of the cutoff wall. Care was taken to ensure the same compaction around the cutoff wall as in the rest of the specimen.

In order to track the migration of fine particles, a green sand zone, which was 300 mm long, 100 mm wide, and 100 mm high, was placed at the bottom section of the cutoff wall, as depicted in Figure 4. The green sand zone had the same gradation as the rest of the specimen before the experiment. Particles finer than 10 mm in the green sand zone were replaced by green sand, while particles greater than 10 mm in the green sand zone were still the original gravels. The green sand had the same specific gravity and similar grain shape (approximately rounded) with the replaced original sand. Due to the same gradation and compaction in the sand substitution, it was expected that the geo-mechanical properties of the green sand zone were the same as the rest of the specimen. Figure 5 shows the embedment of the pore pressure cells and earth pressure cells on both sides of the cutoff wall.

Experimental procedure

After specimen preparation and instrument installation, the specimen container was covered by the loading plate, and the top of the cutoff wall was connected into a rubber-lined groove in the loading plate and secured by bolts. Before application of overburden pressure, all the readings of the five earth pressure transducers were set to zero to filter out the irregular and fluctuant earth pressure induced by compaction. Overburden pressure was applied onto the loading plate by the loading system. An inflated rubber tire encapsulated the perimeter of the loading plate, which tightly fitted into the inner walls of the specimen container. A thin layer of Vaseline was applied to the inner walls. Such configuration of the top loading plate provided a sealed and free-moving top boundary, so that any suffusion-induced settlement would not affect the application of overburden pressure. Four tests with overburden pressures of 0.2 MPa, 0.8 MPa, 1.6 MPa, and 2.4 MPa, respectively, were performed. The applied pressures simulated the overburden pressures caused by an earthen dam in a sandy gravel alluvium with combined dam and subsurface depth of 10 m to 120 m.

After the application of overburden pressure, specimen saturation process started. In general, flow and consequent particle migration depend on the degree of saturation, and the presence of air has profound implications on seepage tests for particle migration. In this test program, only effluent volume was recorded, so it could not quantify the degree of saturation. Yet considering the relatively high permeability of the sandy gravel (1.6×10^{-3} to 4.5×10^{-3} cm/s), CO₂ was not necessary to facilitate the saturation. The soil specimen was continuously flushed with water at low gradient (to avoid suffusion) for an extended period. For example,

in the experiment under overburden pressure of 0.8 MPa, saturation started under a hydraulic head difference of 16.9 cm. Effluent was constantly monitored and no fines were visually observed in the effluent during saturation. Pore pressure transducer tubes and air escape valves on the top of the loading plate were open to expel air until no escaped air bubble was observed for five minutes. Therefore, saturation was assumed to complete after 5.6 hours.

Seepage was then introduced into the specimen under different step-increase hydraulic head differences. For the experiment under overburden pressure of 0.8 MPa, seven step-increase hydraulic head differences were applied: 101 cm, 295 cm, 385 cm, 546 cm, 759 cm, 1192 cm and 1750 cm. Migration of fine particles at the bottom of the cutoff wall was closely monitored through the front transparent Plexiglas wall of the apparatus. Under each hydraulic head difference, flow rate, settlement, and earth pressures were recorded in every 10-minute interval. When the measured flow rate was stable, the pore pressures were recorded as the stable pore pressure values of this step.

Definition of “suffusion initiation” and “blowout”

The evolution of suffusion can be judged by measured and derived parameters, and some observed phenomena in the experiments. The measured parameters in this study were pore pressure, earth pressure, settlement, and flow rate. The derived parameters were average hydraulic gradient derived from pore pressures, and average rate of settlement, v_s , which was the ratio of settlement to the corresponding time. The observed phenomena were the

migration of fine particle and the turbidity of effluent. In this study, suffusion at the tip of the cutoff wall was a localized phenomenon, and the measured pore pressures and the derived hydraulic gradient at the tip of the cutoff wall were also localized physical parameters, so these two parameters were used as main indications of suffusion. As flow rate, settlement, and average rate of settlement were not localized physical parameters, and they may not reflect the evolution of suffusion comprehensively and instantaneously, so these measured and derived parameters and observed phenomena were secondary indications of suffusion.

With the increase of total hydraulic head difference between the upstream and downstream sides, a sudden decrease in pore pressure or hydraulic gradient at the bottom of the cutoff wall indicates the initiation of suffusion. The suffusion experiments performed by Moffat et al. (2011) also found that the sudden decrease in local hydraulic gradient was an indication of suffusion initiation. Simultaneously, some secondary indications also could be used to check the initiation of suffusion. For example, earth pressure at the bottom of the cutoff wall started to increase, flow rate started to increase significantly, noticeable fine particle migration was observed at the bottom of the cutoff wall, and effluent became turbid.

In this study, the failure induced by suffusion was termed as blowout (Sibille et al. 2015), which was characterized by an increased particle migration within a short time period (blowout of fine particles) and produced rapid, large settlement of specimen. In the later stage of the suffusion experiment, further increase of the total hydraulic head difference eventually induced blowout. At this time, significant decrease in pore pressure or hydraulic gradient at

the downstream side of the cutoff wall was observed again. It indicates blowout occurred. In addition, some secondary indications also had significant changes at blowout: the earth pressure at the bottom of the cutoff wall started to increase significantly, settlement or average rate of settlement increased significantly, flow rate started to increase, some coarse particles that served as soil primary fabric were observed to migrate at the bottom of the cutoff wall, and effluent was turbid.

Results and Discussion of the Test under Overburden Pressure of 0.8 MPa

It is noted that only the experimental results under overburden pressure of 0.8 MPa were described comprehensively in this section. Similar results and evolution trends in terms of seepage field, hydraulic gradient, earth pressure, and settlement also appeared in the other three experiments under different overburden pressures.

Determination of suffusion initiation and blowout

For the experiment under overburden pressure of 0.8 MPa, when the total hydraulic head difference between the upstream and downstream sides (ΔH) increased from 295 cm to 385 cm, the total head at C21 (at the downstream side of the tip of the cutoff wall) started to decrease from 249 cm to 238 cm. Meanwhile, the average hydraulic gradient between C21 and C22 (i_{21-22}), which was calculated using the total head difference between the two survey points and their distance, decreased from 13.3 at $\Delta H = 295$ cm to 12.1 at $\Delta H = 385$ cm;

similarly, i_{22-23} also decreased from 10.3 at $\Delta H = 295$ cm to 9.6 at $\Delta H = 385$ cm. According to the main suffusion indications, which are the sudden decreases in pore pressure and hydraulic gradient at the bottom of the cutoff wall with the increase of ΔH , it can be judged that suffusion initiated at the downstream side of the tip of cutoff wall at $\Delta H = 295$ cm. In addition, at $\Delta H = 295$ cm, noticeable fine particle migration that lasted about 30 minutes was observed in the green sand zone through the transparent wall. The effluent became slightly turbid. The secondary indications also indicate that suffusion initiated at the downstream side of the tip of the cutoff wall at $\Delta H = 295$ cm.

When ΔH increased from 546 cm to 759 cm, the total head at C21 started to decrease from 247 cm at $\Delta H = 546$ cm to 235 cm at $\Delta H = 759$ cm. Meanwhile, significant decreases in hydraulic gradient at the downstream side of the cutoff wall occurred. i_{21-22} decreased from 11.6 at $\Delta H = 546$ cm to 2.7 at $\Delta H = 759$ cm, and i_{22-23} also decreased from 10.9 at $\Delta H = 546$ cm to 3.4 at $\Delta H = 759$ cm. In addition, the primary fabric with diameter approximately from 1 mm to 2 mm at the bottom of the cutoff wall was observed to migrate. The effluent was still slightly turbid. According to the main and secondary indications of suffusion, it can be judged that blowout occurred at $\Delta H = 759$ cm.

Effect of suffusion on seepage field

Figure 6 shows the temporal and spatial evolution of equipotential lines under three total head differences with a constant overburden pressure of 0.8 MPa. The equipotential lines were

obtained based on: (a) the measured total head (including measured pore pressure head and corresponding elevation head) at every survey point (C1~C24), (b) measured total heads at the upstream and downstream boundaries, and (c) the total heads along the loading plate and the cutoff wall. For simplicity, it is postulated that the total head on the flow line along the cutoff wall varied linearly, and then the total head on this line was obtained based on linear interpolation between the total heads at the upstream and downstream boundaries.

The seepage field clearly reflected the change in seepage during the progression of suffusion. Fine particle migration and clogging in the evolution of suffusion induced the heterogeneous variation of permeability in the specimen and eventually affected the distribution of equipotential lines. After the saturation process, there was no fine particle migration in the specimen, and the permeability was approximately homogenous in the whole specimen, so the seepage field was generally symmetrical with respect to the cutoff wall, and the equipotential lines concentrated at the tip of the cutoff wall, as shown in Figure 6(a). It indicates that the hydraulic head loss mainly occurred at the tip of the cutoff wall. With the increase of ΔH , fine particle migration was visually observed at the tip of the cutoff wall, then the downstream side of the tip of the cutoff wall may be gradually clogged by fine particles, and the permeability of this region may be far lower than the other regions in the specimen. Consequently, the equipotential lines became non-symmetrical and they mainly clustered at the downstream side of the cutoff wall. It indicates that the hydraulic head loss mainly occurred at the downstream side of the cutoff wall. At $t = 632.3$ min and $\Delta H = 295$ cm, the downstream side of the cutoff wall started to have less ability to cause hydraulic head loss,

and suffusion initiated in this region, as shown in Figure 6(b). At $\Delta H = 385$ cm and 546 cm, the upstream side and the tip of the cutoff wall started to be clogged by the eroded fine particles, and the permeability of these two regions may become far lower than the other regions in the specimen. So the equipotential lines started to cluster at the upstream side and the tip of the cutoff wall. It indicated that suffusion started to expand from the downstream side of the cutoff wall to the upstream side. At $t = 846.3$ min and $\Delta H = 759$ cm, the hydraulic head drops became increasingly concentrated on the upstream side of the cutoff wall. At $t = 937$ min and $\Delta H = 1750$ cm, as shown in Figure 6(c), the equipotential lines clustered at the upstream side of the cutoff wall, and the corresponding total head drop in this region was 65 % of ΔH ; it indicated that the hydraulic head loss mainly occurred in the upstream side of the cutoff wall at this time, and suffusion had completely expanded from the downstream side of the cutoff wall to the upstream side. The similar trend of the seepage field evolution as described above was also observed in the experiments with overburden pressures of 0.2 MPa, 1.6 MPa, and 2.4 MPa.

Figure 7 depicts the time evolution of flow rate in the same experiment with overburden pressure of 0.8 MPa. It can be seen that the variation of flow rate in the evolution of suffusion was erratic. It indicated that the permeability of the entire heterogeneous specimen was erratic in the whole process of suffusion, but overall it reduced. In addition, it might suggest that this overall flow rate was not a reliable indication of suffusion.

Figure 8 shows the evolution of average hydraulic gradient around the cutoff wall with the

suffusion progression under overburden pressure of 0.8 MPa. The legend “high”, “medium” and “low” in Figure 8 represents comparatively high, medium and low hydraulic gradients, respectively. The average hydraulic gradients between pore pressure measurements were calculated using the total head difference between the two measurements and their distance. It should be noted that the values in Figure 8 were not the local hydraulic gradient calculated exactly on the same flow lines; rather, they showed approximate variation of hydraulic gradient around the cutoff wall at various stages of suffusion development. The evolution of average hydraulic gradient at various ΔH in Figure 8 intuitively and quantitatively supported the explanation of the seepage field variation during the suffusion progression that is shown in Figure 6. The similar trend of the average hydraulic gradient evolution as described above was also observed in the experiments with overburden pressures of 0.2 MPa, 1.6 MPa, and 2.4 MPa. In addition, it is noted that the high local hydraulic gradients in this paper (12.1 to 64.7 at various head differences) may be representative of field conditions. Rice (2007) listed the maximum calculated hydraulic gradients across seepage barriers in seven dams, and they ranged from 8 to 47. The maximum and minimum hydraulic gradients of suffusion initiation in the experiments carried out by Moffat and Fannin (2011) were 57 and 7, respectively.

Effect of suffusion on grain size distribution

The grain size distribution of the specimens subjected to suffusion also varied. After each experiment, the grain size distributions of the eroded soil, the green sand zone and Zone A (see Figure 4) were determined by sieve analysis according to ASTM D6913-04. Figure 9

shows the grain size distributions before and after the experiment under overburden pressure of 0.8 MPa. The total eroded mass collected in the outlet was 63.8 g, and the percentage of particles finer than 0.7 mm (the largest fine particle) was 87.5 %. The grain size of the green sand zone significantly changed: the mass percentage of the particles with diameters ranging from 1 mm to 2 mm increased from 2 % to 7.5 %, while the mass percentage of the particles with diameters ranging from 0.5 mm to 1 mm decreased from 11 % to 4.0 %. Compared with the green sand zone, the gradation of Zone A changed slightly, it might suggest that the fine particles that migrated into Zone A from the upstream side were basically equal to those that migrated out of Zone A.

Variations of earth pressures

Figure 10 shows the variation of earth pressures in the experiment under overburden pressure of 0.8 MPa. It can be seen that the monitored earth pressures showed two unusual characteristics: the earth pressure at T3 was greater than the applied overburden pressure; the earth pressures at T1 and T4 were far lower than the overburden pressure. The unusual earth pressures might be due to the relatively small size of the pressure transducers and the significant heterogeneity of stress transfer caused by internally unstable soil, which are further explained as follows.

The heterogeneity of stress transfer in internally unstable soil was studied. Kenney and Lau (1985) inferred that fine particles only transferred a little or no load when fine content was

lower than a critical content. Skempton and Brogan (1994) proposed a stress reduction factor, α , which was the ratio of the effective stress transferred by fine particles to the overall effective stress. For the internally unstable samples A and B in their paper, α was approximately 0.18 and 0.33, respectively. Based on a DEM simulation, Shire et al. (2014) found that α ranged from 0.02 to 1.19, which indicates that the effective stress transferred by fine particles might be greater than the overall effective stress in some cases.

The DEM analyses performed by Voivret al. (2009) and Langroudi et al. (2015) showed that contact force chains in internally unstable soil were significantly heterogeneous. Major force chains preferentially passed through coarse (fabric) particles, while a large number of fine particles were excluded from the force chain network and transferred little stress. In this study, it can be inferred that if an earth pressure transducer (such as T1 or T4) was installed in the region that was not on major force chains, the monitored earth pressure might be much lower than the external load. If a transducer (such as T3) was on major force chains, the measured earth pressure might be larger than the external pressure. The monitored earth pressure here provided an evidence of the heterogeneous stress distribution in internally unstable soil. In addition, the rigid loading plate on the cutoff wall was also shown to affect the stress transfer in the soil. The earth pressures were not the same on the same elevation, due to the existence of the loading plate.

It also can be seen from Figure 10(a) that the variation of earth pressures includes three test stages: 1) application of overburden pressure, 2) saturation, and 3) suffusion. It should be

noted that the earth pressure cells (as pictured in Figure 5) were placed on fine sand and measured the pressures taken by the fine sand. The earth pressures increased rapidly during the application of overburden pressure and gradually decreased in the saturation stage. During the suffusion stage, the earth pressures at T1, T3, and T4, which were located far away from the region where suffusion occurred, changed slowly; while the earth pressures at T2 and T5, which were close to the region where suffusion occurred, increased by 24 % and 23 % relative to the earth pressures at the end of saturation, respectively. The earth pressure at T5 significantly increased from 850 kPa to 940 kPa at the initiation of suffusion; while at blowout, it increased from 1007kPa to 1057 kPa. Similarly, the earth pressure at T2 significantly increased from 612 kPa to 696 kPa at blowout. The similar trend of the earth pressure variation was also found in the other three experiments under different overburden pressures.

Based on the heterogeneity of stress transfer in internally unstable soil, it can be inferred that the increase in earth pressures at T2 and T5 at the initiation of suffusion and at blowout may be attributed to the migration of fine particle. Figure 11 shows a conceptual diagram to demonstrate the rearrangement of the primary fabric and fine particles on an earth pressure transducer in the evolution of suffusion. Before the initiation of suffusion, the stresses at T2 and T5 were transferred by fine particles and primary fabric together. At the initiation of suffusion or at blowout, the fine particles on T2 and T5 were eroded gradually, and then the stresses at T2 and T5 were mainly transferred by the primary fabric. Consequently, the earth pressures at T2 and T5 increased when suffusion initiated or blowout occurred.

Variation of settlement

With the loss of fine particles, especially at blowout, the specimen might experience a sudden settlement. Figure 12 shows the settlement and average rate of settlement (v_s) in the evolution of suffusion under overburden pressure of 0.8 MPa. In this experiment, $v_s < 0.002$ mm/min was observed before the initiation of suffusion. At the initiation of suffusion, $v_s = 0.004$ mm/min. At blowout, v_s significantly increased to 0.01 mm/min. The total suffusion-induced settlement was on the order of 0.32 mm or 0.064 % of the total height of the specimen (50 cm). Similarly, in the experiment under overburden pressure of 2.4 MPa, at the initiation of suffusion, $v_s = 0.002$ mm/min was observed. At blowout, v_s increased tenfold to 0.02 mm/min. The sudden increase in settlement at blowout was also observed by Tomlinson and Vaid (2000) and Moffat et al. (2011). In addition, comparison with Figures 10(b) and 12 shows that the settlement and the average rate of settlement at the initiation of suffusion and at blowout slightly lagged behind the changes in earth pressures.

Hicher (2013) interpreted the mechanism of suffusion-induced settlement. The solid fraction removal progressively increased the void ratio, decreased the sliding resistance of inter-particle contact, and eventually led to macroscopic deformations of the soil specimen. In this paper, when blowout occurred, some primary fabric particles with diameter from 1 mm to 2 mm were observed to migrate. Consequently, microstructure and major force chains inside the soil might be destroyed, causing a sudden settlement at blowout.

In addition, it should be clarified that the measured settlement in this study was the settlement of the loading plate and an average settlement of the entire soil surface. In the field, settlement could be much greater in localized areas that experienced blowout.

Influence of overburden pressure on suffusion

Figure 13 shows the relationships between overburden pressure and the average hydraulic gradients at the initiation of suffusion and at blowout. In the figure, σ is overburden pressure (MPa), ΔH_{cr} and ΔH_b are total head differences (cm) between the upstream and downstream sides at the initiation of suffusion and at blowout, respectively, i_{cr} and i_b are average hydraulic gradients at the initiation of suffusion and at blowout, respectively, and they are the ratios of corresponding ΔH_{cr} and ΔH_b to the seepage path length (180 cm) along the loading plate and cutoff wall. It can be seen that both i_{cr} and i_b linearly increased with the increase in σ , which was consistent with the observation by Moffat and Fannin (2011). According to Skempton and Brogan (1994), the hydraulic gradient that initiated suffusion for sandy gravel under zero overburden pressure was significantly low and only about 0.16~0.2. In this study, a conservative assumption was made that i_{cr} under zero overburden pressure was set to zero. Consequently, two linear relationships were derived and shown in Equations (1) and (2); the coefficients of determination (R^2) were both greater than 0.9.

Average hydraulic gradient at the initiation of suffusion:

$$i_{cr} = 3.27\sigma \quad (\text{Equation 1})$$

Average hydraulic gradient at blowout:

$$i_b = 3.66\sigma + 2.01 \quad (\text{Equation 2})$$

It should be noted the above two equations were derived based on the flume-scale experiments, and their applicability depended on the soil and test conditions. Application of these equations in the field scale was not verified in this study.

In addition, it should be noted that hydro-fracture did not occur in this study. Based on Lirer et al. (2011), if sandy gravel was in dense state as in this study, the at-rest earth pressure coefficient was generally greater than 0.4. Consequently, the minimum principal stress under overburden pressures of 0.2, 0.8, 1.6, and 2.4 MPa was about 0.08, 0.32, 0.64, and 0.96 MPa, respectively. The pore water pressures at blowout were all less than the corresponding minimum principle stress, so hydro-fracture due to tension did not occur in these four experiments.

Multi-phase and multi-field coupling effect of suffusion

Based on the above experimental results, it can be concluded that suffusion is a multi-phase

(involving pore water, fine fraction and coarse fraction of solids) and multi-field (involving seepage, seepage-induced variation of fine content, and stress-deformation) coupling phenomenon, as shown in Figure 14. The detailed interactions can be summarized as follows.

(1) Interactions between pore water seepage and fine fraction variation. Increase in pore water seepage induced the migration of fine particles, and consequently changed the fine fraction in the soil matrix, as depicted in Figure 9. Conversely, fine particle migration in the evolution of suffusion induced heterogeneous variation of permeability, and eventually affected pore water seepage. Before the initiation of suffusion, the downstream side of the tip of the cutoff wall may be clogged by fine particle migration, and the permeability of this region may be far lower than the other regions. In the progression of suffusion, the upstream side and the tip of the cutoff wall started to be clogged gradually, and the permeability of these two regions may become far lower than the other regions, as depicted in Figures 6 and 8.

(2) Interactions between pore water seepage and coarse fraction stress-deformation. Under the same external load, decrease in pore water pressure increased the effective stress of coarse fraction according to Terzaghi's principle of effective stress, and then led to macroscopic deformation of coarse fraction, as shown in Figure 12. Conversely, increase in effective stress of coarse fraction induced an additional deformation, and then affected pore structure and decreased the permeability of the soil. The heterogeneous variation of permeability in the specimen eventually led to changes in pore water seepage, as depicted

in Figures 6 and 8.

(3) Interactions between coarse fraction stress-deformation and fine fraction variation.

Changes in stress and deformation of coarse fraction induced changes in constriction size between coarse fractions, which is the geometry criterion of suffusion (Wan and Fell 2004), and eventually affected fine fraction variation. Conversely, with the increase of fine particle migration, contact force chains inside soil may rearrange, and the effective stress transferred by coarse fraction may start to redistribute, as the increases in earth pressures at T2 and T5 in the evolution of suffusion depicted in Figure 10; Meanwhile, changes in fine fraction also induced a significant reduction in shear strength of soil (Ke and Takahashi 2012; Muir Wood et al. 2010; Scholtès et al. 2010; Hicher 2013). Although the applied overburden pressure was kept constant in the whole process of the experiment, it still led to an additional deformation of the soil specimen owing to the decrease in shear strength, as shown in Figure 12.

Conclusions

This paper presented the initiation and progression of suffusion, eventually leading to blowout at higher hydraulic gradients around the tip of a cutoff wall in a flume-scale model of sandy gravel alluvium. The results indicate that suffusion is a multi-phase (involving pore water, fine and coarse fractions) and multi-field (involving seepage, seepage-induced variation of fine fraction content, and stress-deformation) coupling phenomenon. Temporal

and spatial developments of seepage field and average hydraulic gradient around the tip of the cutoff wall intuitively depicted the evolution of suffusion. Suffusion initiated at the downstream side of the tip of the cutoff wall, and then generally progressed backward to the upstream side, until coarse fraction started to erode, eventually leading to blowout at higher hydraulic gradients. There were sudden decreases in local pore pressure and hydraulic gradient, and sudden increases in local earth pressure and settlement at blowout. The recorded earth pressure provided an experimental evidence of the heterogeneous stress distribution in internally unstable soil. Based on four flume-scale suffusion tests under different overburden pressures, two linear empirical formulas for the initiation of suffusion and blowout were derived.

Acknowledgements

The supports of Natural Science Foundation of China under project No. 51679070, 51009053, 51579078, 51474204, Hohai University Excellent Innovative Talents Support Program, A Project Funded by the Priority Academic Program Development of Jiangsu Higher Education Institutions, and the Pennsylvania State University are gratefully acknowledged.

References

- ASTM D4253-00. 2000. Standard test methods for maximum index density and unit weight of soils using a vibratory table. Annual book of ASTM Standards. ASTM international, West Conshohocken, PA.
- ASTM D4254-00. 2000. Standard test methods for minimum index density and unit weight of soils and calculation of relative density. Annual book of ASTM Standards. ASTM international, West Conshohocken, PA.
- ASTM D2434-68. 2006. Standard test method for permeability of granular soils (Constant Head). Annual book of ASTM Standards. ASTM international, West Conshohocken, PA.
- ASTM D6913-04. 2009. Standard test methods for particle-size distribution (gradation) of soils using sieve analysis. Annual book of ASTM standards. ASTM international, West Conshohocken, PA.
- Bendahmane, F., Marot, D., and Alexis, A. 2008. Experimental parametric study of suffusion and backward erosion. *Journal of Geotechnical and Geoenvironmental Engineering*, 134(1): 57-67.
- Chang, D.S., and Zhang, L.M. 2011. A stress-controlled erosion apparatus for studying internal erosion in soils. *Geotechnical Testing Journal*, 34(6): 579-589.
- Chang, D.S., and Zhang, L.M. 2013a. Critical hydraulic gradients of internal erosion under complex stress states. *Journal of Geotechnical and Geoenvironmental Engineering*, 139(9): 1454-1467.

- Chang, D.S., and Zhang, L.M. 2013b. Extended internal stability criteria for soils under seepage. *Soils and Foundations*, 53(4): 569-583.
- Cividini, A., and Gioda, G. 2004. Finite element approach to the erosion and transport of fine particles in granular soils. *International Journal of Geomechanics*, 4(3): 191-198.
- Dang, L.C., and Fang, G.D. 2011. Major technical problems of constructing dam on deep overburden. *Water Power*, 37(2): 24-28+45.
- Fannin, R.J., and Slangen, P. 2014. On the distinct phenomena of suffusion and suffosion. *Géotechnique Letters*, 4, 289-294.
- Garner, S.J., and Fannin, R.J. 2010. Understanding internal erosion: a Decade of research following a sinkhole event. *International Journal on Hydropower and Dams*, 17(3): 93-98.
- Hicher, P.Y. 2013. Modeling the impact of particle removal on granular material behaviour. *Géotechnique*, 63(2): 118-128.
- Ke, L., and Takahashi, A. 2012. Strength reduction of cohesionless soil due to internal erosion induced by one-dimensional upward seepage flow. *Soils and Foundations*, 52(4): 698-711.
- Ke, L., and Takahashi, A. 2014a. Triaxial erosion test for evaluation of mechanical consequences of internal erosion. *Geotechnical Testing Journal*, 37(2): 347-364.
- Ke, L., and Takahashi, A. 2014b. Experimental investigations on suffusion characteristics and its mechanical consequences on saturated cohesionless soil. *Soils and Foundations*, 54(4): 713-730.
- Kenney, T.C., and Lau, D. 1985. Internal stability of granular filters. *Canadian Geotechnical Journal*, 22(2): 215-225.

- Langroudi, M.F., Soroush, A., and Shourijeh, P.T. 2015. A comparison of micromechanical assessments with internal stability/instability criteria for soils. *Powder Technology*, 276: 66-79.
- Li, M., and Fannin, R.J. 2008. Comparison of two criteria for internal stability of granular soil. *Canadian Geotechnical Journal*, 45(9): 1303-1309.
- Li, M., and Fannin, R.J. 2012. A theoretical envelope for internal instability of cohesionless soil. *Géotechnique*, 62(1): 77-80.
- Lirer, S., Flora, A., and Nicotera, M.V. 2011. Some remarks on the coefficient of earth pressure at rest in compacted sandy gravel. *Acta Geotechnica*, 6(1): 1-12.
- Moffat, R.M., and Fannin, R.J. 2011. A hydromechanical relation governing the internal stability of cohesionless soil. *Canadian Geotechnical Journal*, 48(3): 413-424.
- Moffat, R.M., Fannin, R.J., and Garner, S.J. 2011. Spatial and temporal progression of internal erosion in cohesionless soil. *Canadian Geotechnical Journal*, 48(3): 399-412.
- Muir Wood, D., Maeda, K., and Nukudani, E. 2010. Modelling mechanical consequences of erosion. *Géotechnique*, 60(6): 447-457.
- Rice, J.D. 2007. A study on the long-term performance of seepage barriers in dams. Ph. D. thesis, Virginia Polytechnic Institute and State University, Blacksburg, Virginia.
- Rice, J.D., and Duncan, J.M. 2010. Findings of case histories on the long-term performance of seepage barriers in dams. *Journal of Geotechnical and Geoenvironmental Engineering*, 136(1): 2-15.
- Richards, K.S., and Reddy, K.R. 2010. True triaxial piping test apparatus for evaluation of piping potential in earth structures. *Geotechnical Testing Journal*, 33(1): 83-95.

- Richards, K.S., and Reddy, K.R. 2012. Experimental investigation of initiation of backward erosion piping in soils. *Géotechnique*, 62(10): 933-942.
- Scholtès, L., Hicher, P.Y., and Sibille, L. 2010. Multiscale approaches to describe mechanical responses induced by particle removal in granular materials. *Comptes Rendus Mécanique*, 338(10-11): 627-638.
- Shire, T., O'Sullivan, C., Hanley, K., and Fannin, R.J. 2014. Fabric and effective stress distribution in internally unstable soils. *Journal of Geotechnical and Geoenvironmental Engineering*, 140(12): 04014072.
- Sibille, L., Marot, D., and Sail, Y. 2015. A description of internal erosion by suffusion and induced settlements on cohesionless granular matter, *Acta Geotechnica*, 10(6): 735-748.
- Skempton, A.W., and Brogan, J.M. 1994. Experiments on piping in sandy gravels. *Géotechnique*, 44(3): 449-460.
- Sterpi, D. 2003. Effect of the erosion and transport of fine particles due to seepage flow. *International Journal of Geomechanics*, 3(1): 111-122.
- Terzaghi, K. 1925. *Erdbaumechanik*. Vienna: Deuticke.
- Tomlinson, S.S., and Vaid, Y.P. 2000. Seepage forces and confining pressure effects on piping erosion. *Canadian Geotechnical Journal*, 37(1): 1-13.
- Ul Haq, I. 1996. Tarbela Dam: resolution of seepage. *Proceedings of the Institution of Civil Engineers-Geotechnical Engineering*, 119(1): 49-56.
- Voivret, C., Radjai, F., Delenne, J.Y., and El Youssoufi, M.S. 2009. Multiscale force networks in highly polydisperse granular media. *Physical Review Letters*, 102(17): 17801.

- Wan, C.F., and Fell, R. 2004. Experimental investigation of internal instability of soils in embankment dams and their foundations. UNICIV Rep. R429, The Univ. of New South Wales, Sydney, Australia.
- Wan, C.F., and Fell, R. 2008. Assessing the potential of internal instability and suffusion in embankment dams and their foundations. *Journal of Geotechnical and Geoenvironmental Engineering*, 134(3): 401-407.
- Xiao, M., and Shwiyhat, N. 2012. Experimental investigation of the effects of suffusion on physical and geomechanic characteristics of sandy soils. *Geotechnical Testing Journal*, 35(6): 890-900.

Draft

Table captions

Table 1. Evaluation of internal instability of the soil

Figure captions

Figure 1. Grain size distribution of the deep alluvium of sandy gravel

Figure 2. Illustration of suffusion around the tip of a cutoff wall in a deep alluvium of sandy gravel

Figure 3. Photo of the suffusion apparatus

Figure 4. Schematic of specimen and instrumentation configuration (vertical section view)

Figure 5. Embedment of the transducers (top view)

Figure 6. Temporal and spatial evolution of equipotential lines under different total head differences, unit: cm (overburden pressure of 0.8 MPa): (a) At $t=354.3$ min and $\Delta H=16.9$ cm, saturation; (b) At $t=632.3$ min and $\Delta H=295$ cm, initiation of suffusion; (c) At $t=937$ min and $\Delta H=1750$ cm, after blowout.

Figure 7. Time evolution of flow rate (overburden pressure of 0.8 MPa)

Figure 8. Temporal and spatial development of the hydraulic gradient around the cutoff wall in the evolution of suffusion (overburden pressure of 0.8 MPa): (a) At $t=632.3$ min and $\Delta H=295$ cm, initiation of suffusion; (b) At $t=716.3$ min and $\Delta H=385$ cm, development of suffusion; (c) At $t=846.3$ min and $\Delta H=759$ cm, blowout; (d) At $t=937$ min and $\Delta H=1750$ cm, after blowout.

Figure 9. Grain size distributions before and after the experiment under overburden pressure of 0.8 MPa

Figure 10. Variation of earth pressures in the experiment under overburden pressure of 0.8 MPa: (a) Variation of earth pressures in the entire experiment; (b) Variation of earth pressures at T2 and T5 in the evolution of suffusion.

Figure 11. Conceptual diagram to demonstrate the rearrangement of the primary fabric and fine particles on an earth pressure transducer in the evolution of suffusion: (a) Before the initiation of suffusion; (b) At the initiation of suffusion or at blowout

Figure 12. Settlement and average rate of settlement in the evolution of suffusion under overburden pressures of 0.8 MPa

Figure 13. Relationships between overburden pressure and average hydraulic gradient at the initiation of suffusion and blowout

Figure 14. Illustration of multi-phase and multi-field coupling mechanism of suffusion

Table 1. Evaluation of internal instability of the soil

Criteria	Material description	The soil is internally stable if	Internal instability of the soil in this paper
Kenney and Lau (1985)	Granular soils	$(H/F)_{\min} > 1$, For broadly graded soil, $0 < F < 20\%$; For narrowly graded soil $0 < F < 30\%$.	$(H/F)_{\min} = 0.45$ internally unstable
Wan and Fell (2008)	Broadly graded and gap-graded soils	$P = e^Z / [1 + e^Z]$ $Z = 3.875 \log(h'') - 3.591 h' + 2.436$ for sand-gravel mixtures	$P = 94\%$ internally unstable
Li and Fannin (2008)	Granular soils	For $F < 15\%$, $(H/F)_{\min} \geq 1.0$; For $F > 15\%$, $H \geq 15\%$	For $F < 15\%$, $(H/F)_{\min} = 0.93 < 1.0$; internally unstable For $F > 15\%$, $H = 10\%$ or $8.7\% < 15\%$; internally unstable
Chang and Zhang (2013b)	Broadly graded soils	$P' < 5$, $(H/F)_{\min} > 1.0$; $5 \leq P' \leq 20$, $(H/F)_{\min} > -(1/15)P' + 4/3$; $P' > 20$, Stable	$P' < 5$, $(H/F)_{\min} = 0.45 < 1.0$ internally unstable

Note: F = mass fraction at any grain size d ; H =mass fraction between grain size d and $4d$; P = probability of internal instability; $h' = d_{90}/d_{60}$; $h'' = d_{90}/d_{15}$; d_{90} , d_{60} , and d_{15} = diameters of 90 %, 60 %, and 15 % mass passing, respectively; P' =fines content (<0.063 mm).

Figure 1. Grain size distribution of the deep alluvium of sandy gravel

Figure 2. Illustration of suffusion around the tip of a cutoff wall in a deep alluvium of sandy gravel

Figure 3. Photo of the suffusion apparatus

C1~C24 are pore pressure measurements; T1~T5 are earth pressure transducers.

Figure 4. Schematic of specimen and instrumentation configuration (vertical section view)

Figure 5. Embedment of the transducers (top view)

Figure 6. Temporal and spatial evolution of equipotential lines under different total head differences, unit: cm (overburden pressure of 0.8 MPa): (a) At $t=354.3$ min and $\Delta H=16.9$ cm, saturation; (b) At $t=632.3$ min and $\Delta H=295$ cm, initiation of suffusion; (c) At $t=937$ min and $\Delta H=1750$ cm, after blowout.

Figure 7. Time evolution of flow rate (overburden pressure of 0.8 MPa)

Figure 8. Temporal and spatial development of the hydraulic gradient around the cutoff wall in the evolution of suffusion (overburden pressure of 0.8 MPa): (a) At $t=632.3$ min and $\Delta H=295$ cm, initiation of suffusion; (b) At $t=716.3$ min and $\Delta H=385$ cm, development of suffusion; (c) At $t=846.3$ min and $\Delta H=759$ cm, blowout; (d) At $t=937$ min and $\Delta H=1750$ cm, after blowout.

Figure 9. Grain size distributions before and after the experiment under overburden pressure of 0.8 MPa

Figure 10. Variation of earth pressures in the experiment under overburden pressure of 0.8 MPa: (a) Variation of earth pressures in the entire experiment; (b) Variation of earth pressures at T2 and T5 in the evolution of suffusion.

Figure 11. Conceptual diagram to demonstrate the rearrangement of the primary fabric and fine particles on an earth pressure transducer in the evolution of suffusion: (a) Before the initiation of suffusion; (b) At the initiation of suffusion or at blowout

Figure 12. Settlement and average rate of settlement in the evolution of suffusion under overburden pressures of 0.8 MPa

Figure 13. Relationships between overburden pressure and average hydraulic gradient at the initiation of suffusion and blowout

Figure 14. Illustration of multi-phase and multi-field coupling mechanism of suffusion