Analysis on water-inrush the process of deep excavation in karst area caused by soil internal erosion

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Abstract. Non-negligible internal erosion threatens deep foundation pit excavation safety and stability. A water-inrush incident in Guilin city, a typical karst area, is thoroughly analysed theoretically after revealing groundwater seepage effects on hydraulic conductivity, internal erosion degree and water-flooding probability. The distribution characteristics of the main granular soils at the site are examined. Internal erosion by suffusion of readily erodible gap-graded soil, widely distributed in situ, is specifically studied. The hydraulic conductivity of soil can be determined by the internal erosion extent, S (%), directly calculated by the effective particle size. Comparing the influences of different parameters on permeability, it is found that the formula with uniform coefficient $C_u$ is more practical, because it is more applicable in gap-graded soil with a wide range of hydraulic conductivity, such as the permeability coefficient range of $10^{-6}$ to $10^1$ cm/s. The permeability coefficient-based relationship between seepage velocity and hydraulic gradient is determined. By establishing and implementing a numerical model, the pore-pressure history in each excavation phase reveals that the pore pressure notably increases during excavation, especially in areas adjacent to ancient collapse bodies. Seepage flow driven by excess pore pressure potentially generates a penetrating flow upward through the overlying soil, possibly triggering a water-inrush incident at a maximum seepage velocity of $8.0 \times 10^{-1}$ cm/s. Several suggestions are proposed to prevent water-inrush incidents during deep foundation pit excavation in similar geological setting areas.

1. Introduction
Instability and failure of deep foundation pits have always been difficult problems to be solved in construction projects [1-3], especially in karst areas. In particular, it is very challenging for engineers to excavate deep foundation pits in karst areas, such as Guilin city. Because of karstification over geological time, complex geological conditions, such as karst caves, karrens, and karst collapses, were formed, substantially causing many excavation engineering difficulties, such as during deep foundation pit excavation.

First, the effectiveness of factors triggering disasters is difficult to verify and test in advance [4-5]. The damage caused by different factors is mentioned in the relevant specifications [6-7]. Among many factors, internal erosion by suffusion on the interior of the foundation pit has been fully studied.

Second, models for stability analysis can not reflect the complex geological conditions. Therefore, many commercial software programs are combined and applied by the majority of scholars [8].

This paper takes the example of a water-inrush [9-10] incident that occurred during the construction and excavation of Guilin city from March to June 2016. The course of water-inrush during excavation of deep foundation pit in karst area is explored.
2. Overview of the study site

2.1. Geological setting
As shown in Figure 1, the proposed building is located at the intersection of Wumei road, Jiaotong road and Lingui road, adjacent to several buildings.

![Figure 1. Sketch of the excavation area, ancient collapse area range and labelled areas as a consequence of construction.](image)

The building site belongs to the geomorphic unit of the first terrace of the Lijiang River, Guilin city, where the ground soil is mainly composed of soil overburden and bedrock with strong karst development. According to borehole data, the soil from top to bottom consists of (1) a thin artificial earth fill and silty clay ($Q^m_{4ml+al}$) layer and (2) an alluvial formation ($Q^a_{4al}$). The first aquifer contains sandy and gravelly soil with good roundness and sorting properties, followed by (3) an alluvial-diluvial formation ($Q^m_{3al+pr}$), consisting of clayey sand and gravelly soil connecting the first and second aquifers, especially through the ancient collapse area. Finally, (4) Devonian Rongxian Formation limestone ($D^s_3$) occurs, which is the second aquifer with a complicated network of karst fissures, fractures and pipes.

Large discrepancies exist in stratum consistency, soil compactness and grain-size distribution, and even the spatial distribution of strata is discontinuous and intermittent. The ancient collapse area is filled with silt and clay soil, referred to as unconsolidated collapse materials, surrounded by silty sand, clayey sand and gravel soil ($Q^m_{4col}$).

2.2. Groundwater condition
The excavation area is shown in Figure 1. The earth retaining support system (ERSS) is used to protect the foundation pit in here.

The first groundwater aquifer in the study area is locally supplied by adjacent surface water bodies, such as Guihu Lake and Ronghu Lake (Figure 1). There are annual seasonal variations in the water level ranging from 2.0~3.0 m. The groundwater type is pore groundwater mainly stored in the sand and gravel soil layer. In the dry season, the initial water table depth ranges from 6.30~7.47 m, and the static water table depth ranges from 5.17~6.23 m, i.e., 145.96~147.30 m above sea level.

3. Analysis of PSD curves for the internal erosion potential in gap-graded soil

3.1. Particle-size distribution characteristics
The sieving experiment results based on field sampling are shown in Figure 2 (PSD). The clayey sands and gravel soil in the third layer are widely distributed in Guilin and constitute an inhomogeneous soil
with complex hydraulic properties. The gradation characteristics of this soil result in a bimodal pattern of its grain-size distribution curve, in which coarse and fine particles account for a large proportion with no intermediate particles. Figure 2 reveals a platform in the PSD, which indicates that the soil has a low content of intermediate particles with sizes ranging from 0.25 to 20.0 mm, only 2.5%–6.3%. In addition, the coarse and fine particles account for more than 50% and 20%, respectively, of all particles.

Figure 2. Gradations of the three in situ soil types. The red solid curve represents gap-graded soil, i.e., clayey sands and gravel soil. The dashed curve is poorly graded soil, namely, fine sand. The dashed dotted curve is well-graded gravel.

From the grain-size distribution curve, \(d_{10}, d_{30}, d_{60} \) and \(d_{70} \) are determined as 0.015, 0.11, 17.1 and 21.6 mm, respectively. The uniformity coefficient of this soil reaches 1141.3. Furthermore, the content of fine particles \(P_x \), which can be determined from the PSD curve (Figure 2), corresponding to a \(d_q \) value of 0.57 mm, is approximately 36.4%.

3.2. Evaluation of the likelihood of internal erosion
Because the uniformity coefficient \(C_u \gg 5 \), the permeability of this bimodal soil is dominated and controlled by its fine particles. Therefore, it is necessary to evaluate the likelihood of internal erosion based on the fine-grain content, which is assumed to be the result of consecutive gradation. Moreover, it is postulated that the volume originally occupied by large and coarse grains, with a diameter larger than \(d_q = 0.57 \) mm in this study, would be completely filled with fine grains. This is referred to as the optimal fine-grain content, and the equation is given by [11]:

\[
P_{op} = \frac{0.30 - n + 3n^2}{1 - n}
\]

(1)

where \(n \) is the porosity of the original soil (%) and \(P_{op} \) is the optimal fine-grain content of the soil (%).

The criteria for evaluating the likelihood of internal erosion based on \(P_{op} \) are as follows:
(1) \(P_x < 0.9P_{op} \), the potential for soil-piping erosion exists;
(2) \(P_x > 1.1P_{op} \), the potential for soil-flowing erosion exists; and
(3) \(P_x = (0.9~1.1)P_{op} \), transition between the above erosion types.

The porosity \(n \) ranges from 48%–30%, and the calculated \(P_{op} \) value ranges from 1.0 to 0.39. Based on the above criteria and \(P_x < 36.4\%\), as previously determined, it can be concluded that the clayey sands and gravel soil are likely to experience soil-piping erosion.
4. Analysis of graded erosion and the water inrush phase by suffusion

4.1. Critical hydraulic gradient for the gradual internal erosion potential

At present, there are a dozen calculation methods for the critical hydraulic gradient of the seepage force considering the effect of potential erosion. Terzaghi and Peck [12] derived the earliest theoretical equation suitable for soil-flowing erosion resulting from upward seepage flow.

Other approaches considering different influencing factors have been proposed in past decades [13-14]. Among them, an equation was recommended to determine the critical hydraulic gradient based on graded soil erosion applicable to a wide grain-size range and bimodal gravel soil. It generates a series of updated PSD curves corresponding to different internal erosion levels $S$ (%).

Ojha et al. [15] derived a model for critical head based on the Bernoulli’s equation and a critical dragged stress condition. The model is verified to support Bligh’s empirical findings by piping test. Additionally, another model based on critical velocity is also developed to estimate the critical head. The grain-size gradation of soil can be changed by the internal erosion extent through altering the porosity as the Figure 3 shows.

4.2. Permeability calculation based on varying soil-geometric parameters

Indraratna and Radampola [16], based on Poiseuille’s equation, derived the following equation:

$$
k_h = n \cdot \left( \frac{\gamma_w}{\mu_w} \right) \frac{d_0^2}{32}
$$  \hspace{1cm} (2)

where $k_h$ is the hydraulic conductivity ($\text{cm/s}$); $\mu_w$ is the dynamic viscosity of water ($\text{N} \cdot \text{s/m}^2$); and $d_0$ is the updated minimum equivalent pore channel diameter ($\text{mm}$) based on the right-shifting grain-size distribution curves.

Compared with the abovementioned equation, the permeability equation proposed by Amer and Award [17] was based on solid matrix properties with a notable influence, such as the uniformity coefficient $C_u$. Especially in the excavation area in this study, the soil in the ancient collapse areas attains a large uniformity coefficient $C_u$ of 1141.3, which should not be neglected. The following equation can be considered as a correction of the previous equation:

$$
k_h = 3.5 \times 10^{-6} \cdot \left( \frac{\gamma_w}{1 + e} \cdot \frac{d_{10}^{2.326}}{C_u} \right)
$$  \hspace{1cm} (3)

where $d_{10}$ is the particle size at which 10% of all particles by weight pass through the filter ($\text{mm}$); $C_u$ is the uniformity coefficient; and $e$ is the void ratio, which can be calculated from the soil porosity, $e = n / (1 - n)$.

The variation in the hydraulic conductivity $k_h$ with the internal erosion level $S$ (%). Eq. 3, applicable to a wide range of permeability coefficients, is more representative of clayey sand and gravel soil, which is the unique in situ soil in Guilin city.

4.3. Analysis of the hydraulic behaviour due to seepage on the basis of varying the hydraulic gradient

The hydraulic conductivity controls the extent of internal erosion by suffusion. The hydraulic gradient can be calculated by the critical hydraulic gradient for the different grain-size grades being eroded, which indirectly represents the internal erosion level by suffusion. The typical behaviour of the seepage velocity as a function of the hydraulic gradient and hydraulic conductivity. As expected, the seepage velocity of a unit area of soil increases with the hydraulic gradient.

Four stages of internal erosion by suffusion can be identified according to the range of the hydraulic gradient $i$: (1) Percolation, for $i$ ranging from $3.8 \times 10^{-3}$ to $9.5 \times 10^{-3}$; (2) fine-grain erosion, for $i$ ranging from $9.5 \times 10^{-3}$ to $1.3 \times 10^{-1}$; (3) coarse-grain erosion, for $i$ ranging from $1.3 \times 10^{-1}$ to $5.0 \times 10^{-1}$; and (4) water flooding, for $i$ ranging from $5.0 \times 10^{-1}$ to $8.9 \times 10^{-1}$. 
5. Numerical simulation

To more intuitively reveal the process during the water-inrush incident, a numerical model was adopted [18]. Numerical fitting shows that the model is realistic and effective. The parameters obtained from the soil tests and model calibration and verification are summarized in Table 1.

Table 1. Soil material parameters used for the numerical simulation.

| Properties                      | Silty clay \((Q_{s\text{ml+al}}^2)\) | Sandy and gravel soil \((Q_{s\text{al}}^2)\) | Clayey sand and gravel soil \((Q_{s\text{al+pr}}^2)\) | Limestone \((D_{sr})\) | Soil in the collapse area \((Q_{s\text{col}}^2)\) |
|---------------------------------|-------------------------------------|----------------------------------------------|-------------------------------------------------|----------------------|-----------------------------------------------|
| Hydra. conductivity (cm/s)     | \(6 \times 10^{-6}\)              | \(5 \times 10^{-1}\)                      | \(5 \times 10^{-4}\)                          | \(6 \times 10^{-11}\) | \(5 \times 10^{-3}\)                          |
| Bulk modulus (MPa)             | 5.89                               | 4.24                                         | 0.69                                           | 14.49                | 0.05                                          |
| Shear modulus (MPa)            | 2.72                               | 2.3                                          | 0.17                                           | 7.87                 | 0.02                                          |
| Porosity (%)                   | 0.5                                | 0.3                                          | 0.3                                            | 0.2                  | 0.5                                          |
| Density (kN/cm^3)              | 19                                 | 19                                           | 19.5                                           | 25                   | 17.8                                         |
| Cohesion (kPa)                 | 0.4                                | 0.16                                         | 0.15                                           | 300                  | 0.10                                         |
| Friction (°)                   | 10.3                               | 18.8                                         | 25                                              | 33                   | 10                                           |

According to the simulation, seepage occurred into the ancient collapse area, which has a low pore pressure, in the initial phase. The average specific discharge is approximately \(0.15\) cm/s near the ancient collapse area, and the corresponding stage of internal erosion by suffusion is the second stage, i.e., fine-grain erosion.

As the excavation proceeds, the positions where the maximum seepage velocity occurs migrate to the excavation area and approach the collapse area. Although the value decreased from 0.92 to 0.57 cm/s in the model, the observed maximum seepage velocity of 0.47 cm/s, occurring at the edge of the excavation area, is more consistent with the simulation value of 0.57 cm/s. In the simulation, some seepage flow gushes out of the ground, and the corresponding stage of internal erosion by suffusion is likely the third stage, i.e., coarse-grain erosion extent, approximately reaching 34%.

According to the difference in pore pressure, the estimated inflow and outflow velocity values in the soil third layer are approximately \(8.0 \times 10^{-1}\) and \(7.0 \times 10^{-2}\) cm/s, respectively. Particularly near the collapse body, a steep increase in seepage velocity occurs. The simulation results provide a better explanation why water-inrush incidents more readily occur in the neighbourhood of ancient collapse areas.

The spatial positions of the monitoring points are shown in Figure 4. Figure 5 demonstrates that the pore pressures present a general upward trend. This implies that an excess pore-pressure zone is generated as excavation progresses. The excess pore pressure zone will drive the flow up and create an upward erosive force. The fluctuation in the piezometric level is relatively smaller than that in the pore pressure, where the former attains a value of 0.45±0.03 MPa, while the latter varies from 0.20 to 0.47 MPa.

When the pore pressure exceeds the piezometric level, the likelihood of a water-inrush incident occurring greatly increases. Clearly, when the excavation activities enter the phase of subarea C, the pore pressure will fluctuate within the range of the piezometric level, which means that it becomes extremely likely that intermittent water-inrush incidents can occur.
6. Conclusion
(1) By analysing the PSD curves obtained by sieving tests, gap-graded soil is susceptible to internal erosion by suffusion, mainly soil-piping erosion. Considering the internal erosion extent $S$ (%) as an influencing factor, a series of shifting PSD curves under the effect of gradual internal erosion are derived.

(2) By adopting the equation considering gradual erosion, a series of critical hydraulic gradients aimed at different particle gradations are determined based on the shifting PSD curves. By comparing the three permeability equations, adaptation of the hydraulic conductivity equation under the effect of specific parameters, such as the effective grain size, porosity and uniformity coefficient, is examined. The equation accounting for the uniformity coefficient $C_u$ as an influencing factor is more consistent with the hydraulic characteristics of the soil in the study area and applies to a wide range.

(3) The simulation results reveal that the seepage velocity and pore pressure near the ancient collapse body are greatly affected by the excavation process. The change in seepage velocity at the site may lead to the development of internal erosion from the second stage to the third stage, i.e. coarse-grain erosion, approximately reaching 34%. In addition, local upward seepage occurs driven by the excess pore pressure generated beneath the ERSS. The likelihood of water gushing into the foundation pit is extremely high and is most likely to occur during the excavation phase of subarea C.

(4) The water-inrush incident occurred because the structure consisting of cement mixing piles failed to reach the limestone bedrock to act as a waterproof curtain. In similar karst areas, it is suggested that the pile structure should rest on the bedrock as much as possible or local in situ grouting should be applied.

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