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# The Stability Analysis of Foundation Pit Under Seepage State Based on Plaxis Software

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## Abstract

The stability of excavation engineering is closely related to groundwater, so it is important to study the impact of seepage flow on the stability of foundation pit. The work is based on the percolation theory and principles of strength reduction. The computation were done with use of the Plaxis software. There were studied simulations which included the seepage state and simulations which didn't include this effect. In order to study the influence of seepage on the stability of foundation pit, there was computed the stability coefficient by using the strength. The results show that, when the seepage stability was not considered, the coefficient is 30% larger than when considering the seepage. Therefore, when designing and calculating the excavation, the seepage should be considered when checking stability if there is groundwater.

**Keywords:** foundation pit, seepage, Plaxis, stability, strength reduction

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## 1. Introduction

For excavation, especially the excavation in high water table areas, there are head differences inside and outside the pit, besides, under that case, groundwater seepage will occur at the pit head difference. Pore pressure inside and outside the hole and the change of the effective stress caused by groundwater seepage are serious threats to the stability of the excavation engineering (Sheng, 2008). Research shows that 60% of pit accidents are directly or indirectly related to groundwater (Qiang et al., 2007). Thus, the analysis of excavation stability must be attached with great importance to the groundwater and groundwater seepage.

Seepage problems of excavation are mainly related to the role of groundwater flow in the rock excavation of soil pores or cracks and other media. Domestic researchers have done a lot of research on the impact of water and explained different aspects of seepage stability on the excavation and other related computing problems, and they have achieved certain project benefits (Zhuanzheng et al., 2012; Cheng et al., 2009; Chang et al., 2002; Huangchun and Xiaonan, 2001). Seen from the engineering point of view, it has greater applicability when handling finite element seepage problems. However, different finite element software and soil constitutive model would result in different calculated results. Based on this point, in order to explore the impact of groundwater seepage on pit excavation engineering further, the application of geotechnical engineering software Plaxis specific engineering examples and comparative analysis should both consider with seepage and without it, and apply strength reduction method to calculate the stability factor for these two different states.

## 2. Strength reduction

Strength reduction was first proposed by J. M. Duncan; he pointed out that the safety factor can be defined as the extent of soil shear strength reduction when the slope has just reached the critical failure state (Quan et al., 2008). By gradually reducing the shear strength index, the  $c$  value is divided by a corresponding reduction coefficient at the same time, and a new set of strength index is obtained; calculate this way for several times, until the slope reaches a critical failure state, and  $F_{ST}$  is used at this time, which is the safety factor of the slope (Quan et al., 2008):

$$F_{sr} = \frac{c}{c'} = \frac{\tan(\phi)}{\tan(\phi')} \quad (1)$$

In the formula, the original  $c$  and  $\phi$  are cohesion and friction angle of slope. The reduced values are labelled  $c'$  and  $\phi'$ . They are used when the slope reaches a critical damage at the level of Poe safety factor. It can be seen from the basic principles of strength reduction that the safety factor is obtained by the method clearly, and the method is simple enough to be applied into practical engineering.

The determination of instability criterion has also been a focus of discussion on the overall stability analysis of foundation. There are five parts proposed about the slope failure criterion currently (Quan et al., 2008): the convergence criterion; plastic zone generalized shear strain criterion or generalized plastic strain criterion; criterion dynamics criterion; displacement or displacement criterion; and mutation rate criterion.

Each criterion has its own scope of application here the fifth criterion is chosen as the criterion of slope failure, namely, the displacement of the feature point mutations can occur suddenly or displacement will increase quickly when the slope is close to its destruction. Because the displacement of the mutation will mean the beginning of part instability, displacement mutation selection criterion or displacement mutation rate criterion has the physical advantages, and the only problem lies in the selection of feature point. Theoretically speaking, only points

within slip surface can serve as the feature points in terms of selecting feature points, starting from the inside retaining wall engineering dot located near the surface of the soil excavation is more suited to the convergence point discriminating judgment.

### 3. Plaxis software and percolation theory

#### 3.1. Plaxis software

Plaxis geotechnical engineering finite element analysis software is a software that is used to solve the problems of geotechnical engineering, such as deformation, stability, as well as groundwater seepage, and it has already become the world-renowned geotechnical engineering finite element analysis software. Compared with other similar types of geotechnical engineering software, Plaxis software has certain advantages on soil stability calculation or seepage calculation (Wei et al, 2011).

In the Plaxis software, groundwater percolation theory is mainly based on percolation theory of finite elements. Flow in porous media can be described by Darcy's law. Considering the vertical flow within the  $x$ - $y$  plane:

$$q_x = -k_x \frac{\partial \phi}{\partial x}; q_y = -k_y \frac{\partial \phi}{\partial y} \quad (2)$$

In the formula,  $q$  is treated as the flow rate ratio, which is calculated by the permeability and groundwater head gradient obtained. Head is defined as:

$$\phi = y - \frac{p}{r_w} \quad (3)$$

In the formula,  $y$  is in a vertical position and represents the pore water pressure (negative pressure) and severe water. For steady-state flow, the continuous application conditions are:

$$\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} = 0 \quad (4)$$

Formula (4) represents the total amount of water flowing into the unit body total water per unit time and is equal to the outflow. After the simulation of the entire discrete objects, corresponding groundwater head in any place within the cell can be used to represent the node cell values:

$$\phi(\xi, \eta) = N \varphi^e \quad (5)$$

In the formula,  $N$  is the shape function, and  $\xi$  and  $\eta$  are the local coordinate units. In accordance with Eq. (2) that the flow rate is based on the gradient of the groundwater head, the gradient matrix can be determined. It is the spatial derivative of interpolating function. To describe saturated soil (less saturation line) and nonsaturated soil (phreatic line), reduction function is introduced in the Darcy's theorem:

$$q_x = -K^r k_x \frac{\partial \phi}{\partial x}; q_y = -k_y \frac{\partial \phi}{\partial y} \quad (6)$$

The value of reduction function below the saturation line is 1 (positive pore pressure), and the value above the phreatic line is less than 1 (negative pore pressure). In the transition zone above the phreatic line, the function value is reduced to  $10^{-4}$ . In the transition zone, function logarithmic is in linear relationship:

$$\lg K^r = -\frac{4h}{h_k} \quad (7)$$

In the formula,  $h$  represents the head pressure and  $h_k$  represents the head pressure where the reduction function reduces to the head pressure of  $10^{-4}$ . In Plaxis, the default is 0.7 m (with the selected unit of length has nothing to do). In the numerical analysis, the ratio of the flow is written as:

$$q = -K^r R B \phi^e \quad (8)$$

among them:

$$q = \begin{bmatrix} q_x \\ q_y \end{bmatrix}; R = \begin{bmatrix} k_x & 0 \\ 0 & k_y \end{bmatrix} \quad (9)$$

Flow from the node ratio can be obtained by integrating the node traffic:

$$Q^e = -\int B^T q dV \quad (10)$$

In the formula,  $B^T$  is the transposed matrix. The following equation is applied in the unit level:

$$Q^e = K^e \phi^e; K^e = \int K^r B^T R B q dV_0 \quad (11)$$

On the global level, all units of contributions are superimposed, and boundary conditions are applied (groundwater flow and head loss), which is formed in  $n$  unknown quantities of  $n$  equations:

$$Q = K \phi \quad (12)$$

In the formula,  $K$  is a global traffic matrix and  $Q$  includes boundary conditions specified as flow losses. When the saturation line is unknown (unconfined water issues), the pickup (Picard) iterative method is used to solve the balance of the system. At this point the problem can be solved by the iterative process which can be written as:

$$K^{j-1} \delta \phi^j = Q - K^{j-1} \phi^{j-1}; \phi^j = \phi^{j-1} + \delta \phi^j \quad (13)$$

In the formula,  $j$  is the iteration number, which is an unbalanced vector. In each iteration of the groundwater heads, nodes of unbalanced flow should be computed, and effective head

should be added. New distribution of the groundwater head should be computed again according to formula (8) recalculation of the traffic, and integrated into the node traffic. This process continues to standard imbalance vector, namely, the error node traffic is smaller than the allowable error.

### 3.2. Soil model

Constitutive model of soil is the premise of stability calculation. At present, constitutive model of soil can be roughly divided into three categories: Elastic class model (elastic model, Duncan-Chang (DC) model), elastic-ideal plastic class model (Mohr-Coulomb (MC) model, Drucker-Prager (DP) model), and strain hardening elastoplastic model class (Modified Cambridge (MCC) model, Plaxis hardening soil (HS) model). MC model is the most widely used, but MCC model and the HS model have greater applicability in the simulation of the nature of the soil (Zhonghua and Weidong, 2010; Feng and Po, 2011).

HS model is put forward by Schanz (Duncan, 1996), which is an isotropic hardening elastoplastic model. HS model can consider both the shear and compression hardening, and the use of Mohr-Coulomb failure criterion. The basic idea is to assume partial HS model and vertical drained triaxial stress test should remain in hyperbolic relationship. For HS elastoplastic model to express this relationship, HS model considers soil dilatancy and neutral loading. Ideal elastoplastic model is different, and HS model in stress space yield surface is not fixed and it varies with plastic strain and expansion. HS models can adapt to describe a variety of damage and deformation behavior of soil types and is suitable for various geotechnical engineering applications, such as embankment filling, foundation bearing capacity, slope stability analysis and excavation, and so on. The following numerical simulation of the following will adopt HS model.

## 4. Numerical simulation

### 4.1. Project overview

Specific examples of excavation adopt the foundation of Sheng (2008). Excavation width is 20 m, depth 10 m, with two 15 m deep and 0.35 m thick concrete diaphragm walls and two rows of anchors as shoring structure, where the first row of anchor length is 14.5 m with 33.7° inclination, and the second row bolt length is 10 m, with an angle of 45°. Considering the surrounding load factors, a load of 10 and 2.5 kN/m<sup>2</sup> is added around the pit. Related soils are filling (0–3 m), sand (3–15 m), and sand and mud (>15 m), and the underground water level in the initial state lies in 3 m below the surface.

Combing with the case background, a geometric mode with 80 m width and 20 m height is established by the Plaxis software, and the generated geometric model and network are shown in **Figure 1** and **Figure 2**. Parameters related to soil properties and structures are shown in **Table 1** and **Table 2**, by taking the default value of the software, which is no longer listed in **Table 1**. The excavation pit is divided into three stages, namely, the first excavation of the subsurface 3 m, then reexcavation of 4 m, and the last remaining excavation 3 m. Plaxis is divided into six steps of excavation.



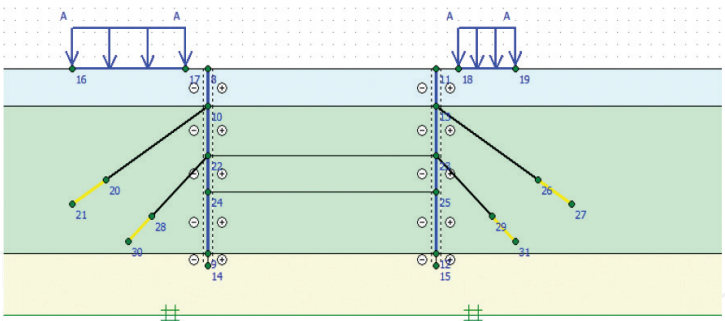


Figure 1. The geometric model of foundation.

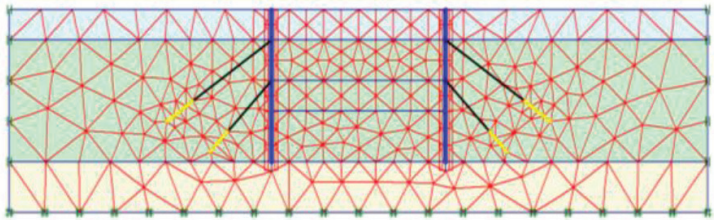


Figure 2. The grid division of foundation.

Parameter	Name	Filling	Sand	Sand and mud
Severe natural (kN/m <sup>3</sup> )	$\gamma_{\text{unsat}}$	16.00	17.00	17.00
Severe saturation (kN/m <sup>3</sup> )	$\gamma_{\text{sat}}$	20.00	20.00	19.00
Horizontal permeation coefficient (m/day)	$k_x$	1.000	0.500	0.100
Vertical permeability coefficient (m/day)	$k_y$	1.000	0.500	0.100
Test secant stiffness of triaxial test (kN/m <sup>2</sup> )	$E_{50}^{\text{ref}}$	22,000	40,000	20,000
The main tangent stiffness in the loading consolidation apparatus (kN/m <sup>2</sup> )	$E_{\text{oed}}^{\text{ref}}$	22,000	40,000	20,000
Unloading/reloading stiffness (kN/m <sup>2</sup> )	$E_{\text{ur}}^{\text{ref}}$	66,000	40,000	20,000
Power exponential function	$m$	0.50	0.50	0.60
Group cohesiveness (kN/m <sup>2</sup> )	$c$	1.00	1.00	8.00
Friction angle (angle)	$\phi$	30.00	34.00	29.00
Dilation angle (degree)	$\psi$	0.00	4.00	0.00
Interface reduction factor	$R_{\text{inter}}$	0.65	0.70	1.00

Table 1. The soil parameters of foundation.

4.2. Simulation results

The simulation is divided into two cases: one is that considers the seepage, and the other is that does not consider the seepage and displacement. They are shown in Figure 3.

	Parameter	Name	Numerical value
Diaphragm wall panel trench	Axial rigidity (kN)	EA	$1.2 \times 10^7$
	bending rigidity (kN. m)	EI	$1.2 \times 10^3$
	Equivalent thickness (m)	$d$	0.346
	Severe (kN/m <sup>3</sup> )	$w$	8.3
	Poisson ratio	$v$	0.15
Bolting	Axial stiffness (kN)	EA	$2 \times 10^5$
	horizontal spacing (m)	LS	2.5
Grout	Axial rigidity (kN)	EA	$2 \times 10^5$

Table 2. Structural parameters.

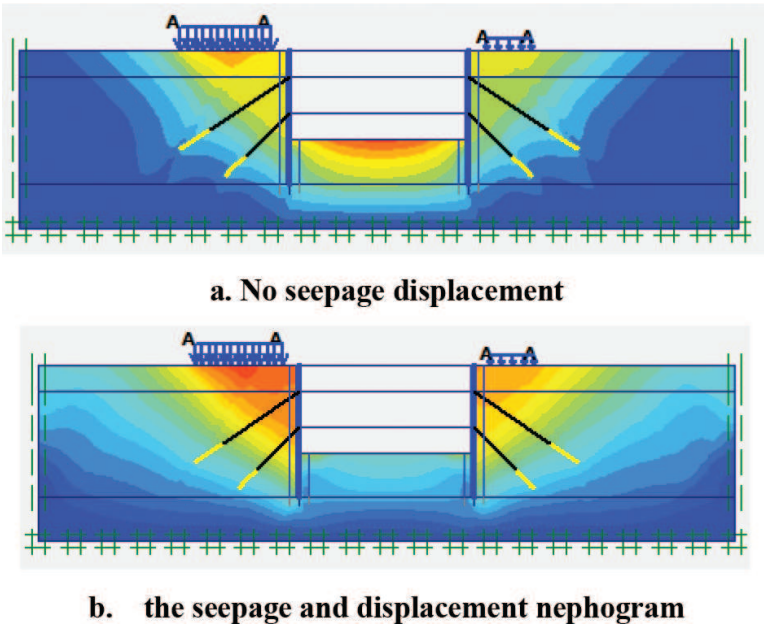


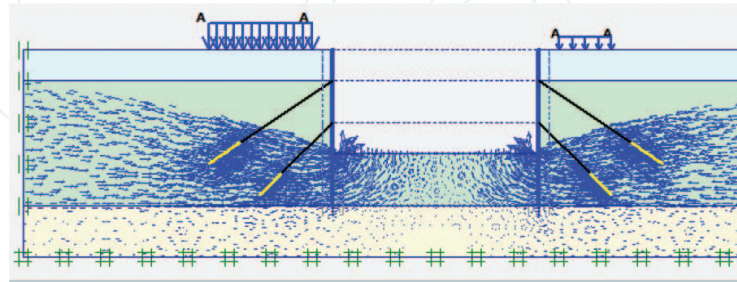
Figure 3. Yaxi Expressway. At the end of excavation displacement contours. (a) No seepage displacement and (b) the seepage and displacement nephogram.

From the numerical simulation computed by finite element, it can be seen that when the seepage is not considered, the maximum displacement is 22 mm, which occurs in pit bottom. When considering seepage, the maximum displacement is 47 mm, which occurs in the soil layer with a load of 10 kN/m<sup>2</sup>. By comparing the differences between the two, it can be seen that in most regions of the pit, soil displacement is greater than the case that the seepage is considered, it is not displaced when considering the seepage. So when seepage is not considered, foundation displacement calculation is a little dangerous.

When considering seepage (Figure 4), it can be seen that the boundary seepage pit is slightly arc-shaped, grout, and anchor near the foot of the slope location seepage velocity, and the maximum value will reach  $387.73 \times 10^{-3}$  m/day. By contrast, considering the seepage and



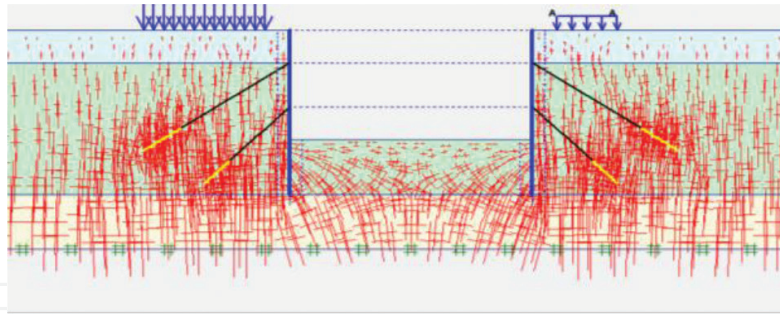
displacement map and when not considering seepage, seepage velocity larger field position, displacement difference reaches 20 mm, which is slightly large, indicating the presence of seepage field, in terms of the pit, it will increase the displacement of its territories. If the flow is not considered, it is unreasonable to value soil excavation pit stabilization with the calculated displacement.



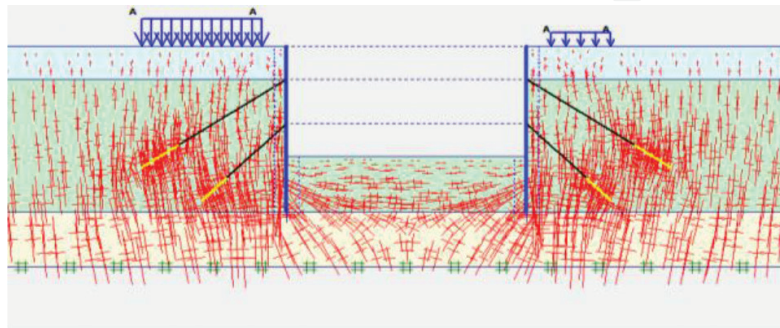
**Figure 4.** Seepage field with seepage.

When comparing the difference between displacement, it can be seen that there exists great difference between these two cases that with and without considering the seepage flow. Differences between the soil stress can be analyzed through **Figure 5** and **Figure 6**. **Figure 5** shows the effective stress diagram in which flow is not considered, because in the case without considering the water, effective stress and total stress is always equal, and the total stress diagram is no longer listed specially. **Figure 6** shows effective stress diagram and the total stress that considers seepage. It can be seen from **Figure 5**, the maximum effective stress occurs near the body part of bolting and grouting, the maximum effective stress is  $-363.91 \text{ kN/m}^2$ . From **Figure 6**, it can be seen when considering the seepage, there are similarities between pit effective stress exhibited seepage pit and without considering the effective stress distribution. And all, the maximum occurs in the vicinity of bolting and grouting body soil. There are also great differences. When considering the seepage, the maximum effective stress reaches  $-407.11 \text{ kN/m}^2$ . In terms of the entire distribution, when considering the seepage pit, the distribution of effective stress is not as intensive as shown in **Figure 5**, which is so concentrated in the vicinity of the distribution of grout; therefore, when considering effective stress of seepage pit, most of the soil area is larger than that does not consider effective stress of seepage time. By analyzing the total stress diagram when considering the seepage, it can be seen that the total maximum stress occurs when there is seepage near the foot of the slope anchor grouting and excavation position, which has certain pertinence with the maximum speed occurring at the seepage field. By analyzing **Figure 4** together with **Figure 6**, it can be found that the increase of the total stress mainly manifests in the area where there is seepage, and the greater speed the seepage becomes, the more obvious the seepage increases.

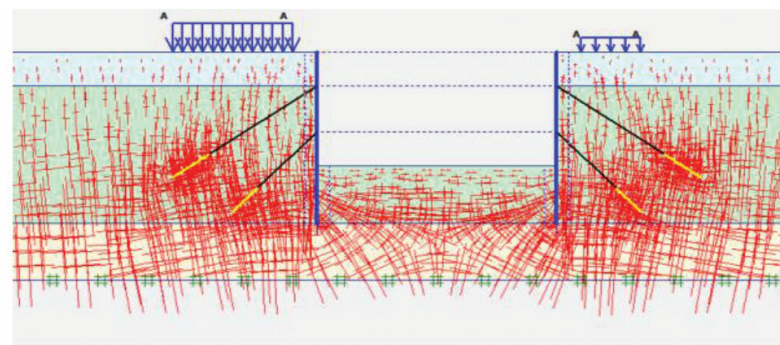
By comparing and considering the two cases, it can be found that when seepage is not taken into account, both the soil displacement and stress are small, so in this case, the calculated conditions will reduce the accuracy of numerical simulation.



**Figure 5.** The absence of effective stress of seepage pit.



**a. The effective stress seepage of foundation pit**



**b. The total stress seepage of foundation pit**

**Figure 6.** The stress of foundation with seepage pit.

#### 4.3. Finite element strength reduction

To further study the impact of seepage pit stability in the original calculation step by considering the seepage and without considering, add a new step 7, reset displacement to zero, and conduct strength reduction operation. Select mutations displacement for instability criterion, select point A as the displacement point, as shown in **Figure 7**. Point A has a distance of 28 m to the left edge, and 7 m to the upper boundary. The displacement corresponding to A under different reduction coefficient is shown in **Figure 8**.

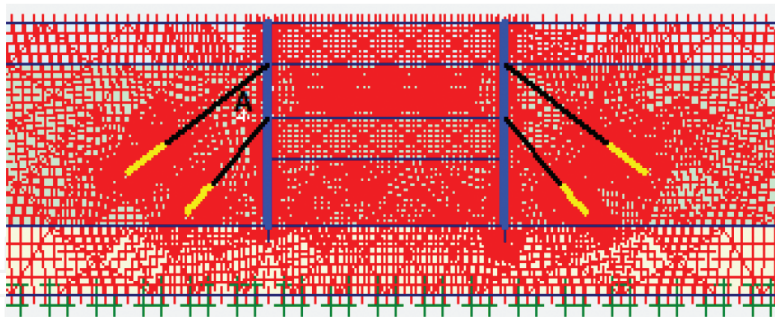


Figure 7. The position of displacement point A.

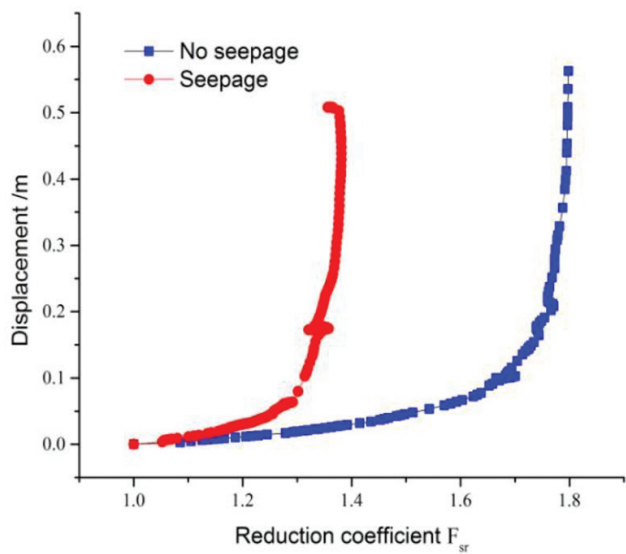


Figure 8. The corresponding values for the various displacement reduction factor.

By analyzing **Figure 8**, it can be seen that for the case when there is no seepage, point mutation displacement of A occurs between 1.6 and 1.7 reduction factor, and by combining the specific data, the stability factor of 1.67 is determined. For seepage cases, it can be seen that point A displacement occurs between 1.2 and 1.3 mutation, and the steady flow coefficient was 1.28 by combining with the specific data. By comparing these two cases, we find that the gap between large and stable coefficient is a little great, in which when seepage is not considered, the stability coefficient is 30% larger than the contrary case with more errors. The strength reduction operation further indicates that the results without considering seepage are rather dangerous. And the stability of the excavation cannot be assessed reasonably.

## 5. Conclusion

In this chapter, the process of numerical simulation program is excavated by applying Plaxis and combining the example. It analyzed the stability under seepage pit. The conclusions are as follows:

- (1) By finite element numerical simulation, it can be seen that in most regions of the pit, soil displacement with considering seepage is larger than when not considered. And in greater flow velocity field position, displacement values are greater, too.
- (2) The differences between the soil stresses can be analyzed through **Figure 5** and **Figure 6**. In effective stress when considering the seepage, there is distribution similarity in pit effective stress. With the contrary case, it has also shown great differences. When considering the seepage, most of the effective stress soil excavation area is greater than the effective stress without considering the seepage. By analyzing the total stress diagram, it can be seen that when considering the seepage, the total stress distribution has a certain relevance with flow field velocity distribution. And the increase of the total stress mainly manifests in the area with seepage, and the greater speed of the seepage is, the more obvious the increase is.
- (3) Comparing these two cases through strength reduction method, it can be seen that the stability factor is larger, when seepage is not considered, and the stability coefficient is 30% larger than the contrary case with more errors.
- (4) Through the overall analysis, calculated seepage is unreasonable without considering the case of seepage, which will reduce the practical significance of engineering. The impact of seepage should be taken into account when analyzing foundation stability.

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