International journal of advances in engineering and management (IJAEM)
Volume 3, issue 6 June 2021, pp: 1484-1492 www.ijaem.net ISSN: 2395-5252

Basic Theory of Seepage Consolidation and Analysis of Soil Settlement after Application of Water-proof Layer

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Submitted: 01-06-2021 Revised: 14-06-2021 Accepted: 16-06-2021

ABSTRACT:RongjiangRiverTunnel is a key control project on GanzhouCity Expressway with a total length of 1745mm. In this paper, numerical simulation of the stability of the face of Rongjiang Tunnel under partial infiltration is carried out. The variation laws of surface displacement, axial force, pore water pressure and safety factor of the face before and after reinforcement are revealed, which can provide technical reference for safe entry of the tunnel.

Key words:Seepage consolidation; Water barrier; Soil settlement; Analysis

I. INTRODUCTION

The construction of shallow excavation tunnel needs to be carried out in the anhydrous environment, and the dewatering construction of rock and soil mass in the scope of tunnel construction should be carried out before the tunnel excavation. Precipitation would not only lead to the seepage consolidation settlement of this part of soil layer, but also bring surface vegetation to death due to water loss.

II. BASIC THEORY OF SEEPAGE CONSOLIDATION

2.1 Seepage theory

1) Effects of seepage on rock and soil

(1) Chemical potential erosion

(2) Physical weakening reaction

(3) Mechanical action

2)Basic concept of seepage

(1) Waterhead function

Head is a basic physical quantity to characterize seepage field. It is a function of spatial coordinate and time t, which is a scalar. General head H_d at any point in the seepage field can be expressed as:

$$H_d = z + \frac{p}{\gamma_w} + \frac{v^2}{2g} \tag{1}$$

In Formula:

p —— pore water pressure, $\frac{p}{\gamma_w}$ called pressure head;

z —— position head;

v —— seepage velocity, $\frac{v^2}{2g}$ called velocity head.

Because the velocity of groundwater is small in the seepage process in rock and soil, the seepage velocity head is usually ignored, so the water head H in the seepage field can be expressed as follows:

$$H = Z + \frac{P}{\gamma_w} \tag{2}$$

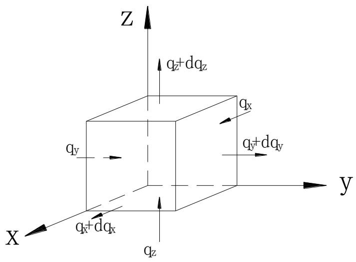

Figure 1 Unit in the seepage region

(2) Permeability velocity

Permeability velocity v refers to the average velocity of seepage flow perpendicular to the cross section. It is the basic physical quantity to characterize the seepage field. It is a function of spatial coordinate and time t, which is a vector.

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$$v = nu$$
（3）

In Formula:

n —— porosity of rock and soil;

u —— the true speed of water flow in rock and soil.

(3) Permeability flow

The seepage velocity of rock and soil can be obtained by seepage analysis, and the seepage flow Q of corresponding area can be obtained by selecting the appropriate seepage control section S :

$$Q = \iint_S v_n dS \quad （4）$$

(4) Hydraulic gradient

Hydraulic gradient i , is the head loss per unit seepage length, the same as the ratio of head loss to seepage length, indicating the change law of water head in seepage flow:

$$i = -\frac{dH}{dL}$$
（5）

In Formula:

dH —— head loss between two points;

dL —— seepage length.

The negative sign is because the water head decreases gradually along the seepage direction. So the increment of water head is negative in the seepage direction, while the hydraulic gradient is positive.

3)The basic law of seepage —— Darcy's law

Darcy (H.Darcy) carried out a large number of experiments on uniform sand, and obtained the law of seepage motion in sand under laminar flow condition, namely, Darcy's law (D racy's law).

Through the experimental study, Darcy found that the seepage flow Q is proportional to the Hydraulic gradient i and cross section S , related to the soil permeability:

$$Q \propto S\frac{dH}{dL}$$
(3)

The same equation as $Q = -kS\frac{dH}{dL}$ (7)

Or $v = \frac{Q}{S} = -k\frac{dH}{dL} = ki$ (8)

In Formula: k —— permeability coefficient, reflecting the permeability of soil.

This formula is Darcy's law.

Permeability coefficient k is the coefficient that reflets the permeability of soil, which is related to many factors, such as soil type, particle size distribution, void ratio, mineral composition, structure and saturation.

Different types of soil have different permeability. The permeability coefficient of cohesive soil is smaller than that of non cohesive soil. The following table shows the average permeability coefficient of common soil, as shown in Table 1.

Table 1 Permeability coefficients of various soils

Types of soil	k permeability coefficient m/s	Types of soil	k permeability coefficient m/s
Clay clay	<6×10^{-8}	Fine sand	1×10^{-5}~6×10^{-5}
Clay loam	6×10^{-8}~1×10^{-6}	Medium sand	6×10^{-5}~2×10^{-4}
Light clay loam	1×10^{-6}~6×10^{-6}	coarse sand	2×10^{-4}~6×10^{-4}
Loess Plateau	3×10^{-6}~6×10^{-6}	Round gravel	6×10^{-4}~1×10^{-3}
Sand	6×10^{-6}~1×10^{-5}	Pebble	1×10^{-3}~6×10^{-3}

The properties of water also have an effect on k , mainly due to the different temperature, the different viscosity degree of liquid and the different density of liquid. The permeability coefficient is inversely proportional to viscosity and directly proportional to liquid density.

4) General Darcy's Law

In the three-dimensional seepage problem, the head h of each point in the seepage field is a function of its position coordinates (x, y, z), and the components of the hydraulic gradient in x,y,z three directions is i_x, i_y, i_z ,which can be expressed as:

$$\begin{cases} i_x = -\dfrac{dH}{dx} \\ i_y = -\dfrac{dH}{dy} \\ i_z = -\dfrac{dH}{dz} \end{cases}$$
(9)

$$i = \sqrt{i_x^2 + i_y^2 + i_z^2} \quad (10)$$

As a result, the seepage velocity in x ,y,z three directions can be expressed as follows:

$$\begin{bmatrix} v_x \\ v_y \\ v_z \end{bmatrix} = -\begin{bmatrix} k_x & k_{xy} & k_{xz} \\ k_{yx} & k_y & k_{yz} \\ k_{zx} & k_{zy} & k_z \end{bmatrix} \begin{bmatrix} i_x \\ i_y \\ i_z \end{bmatrix} \quad (11)$$

or abbreviated as:

$$[v] = -[k][i] \quad (12)$$

k is the matrix of permeability coefficient, which is a symmetric matrix, the same as $k_{xy} = k_{yx}$. And the permeability of any point in the rock and soil mass is its inherent property, which is not affected by the selection of different coordinate systems. Therefore, the permeability coefficient matrix k satisfies the coordinate transformation rule, and the direction corresponding to $k_{xy} = k_{yx} = 0$ is called the seepage principal axis direction.

5）Saturated seepage field of porous media

In the continuity equation of seepage, according to the law of conservation of mass, the mass volume of fluid in the process of seepage will not change. As a result, the increase or decrease of water content in a unit is the difference of water quantity which flows in and out of the unit. A microelement soil with integral product as $dx \cdot dy \cdot dz$ in the stable seepage field is selected. If the seepage velocity of soil in three directions is vx、vy、vz, the volume of water flowing through the left side of soil per unit time is $Q_x = v_x \rho_w dydz$, and the volume of water flowing out from the right side of soil is $Q_x + dQ_x = (v_x + \dfrac{\partial v_x}{\partial x})\rho_w dydz$. Then the difference of the left and right surface flow of the microelement soil is. In the same way, the difference between the front and back face of the microelement soil and the upper and lower surface flow can be calculated, and the difference of the flow rate in the three directions is superimposed. As a result, the expression of the total water quantity change of the microelement in unit time can be obtained:

$$dQ = -\left(\frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} + \frac{\partial v_z}{\partial z}\right)\rho_w dxdydz \quad (13)$$

If the porosity of soil medium is n , the volume of water in saturated soil is ndxdydz, its mass is $M_w = n\rho_w dxdydz$, and the change rate of M with time is:

$$\frac{\partial M}{\partial t} = \frac{\partial (n\rho_w dxdydz)}{\partial t} \quad (14)$$

For saturated soil, the change of soil itself can be ignored, so that the change of soil volume is equal to the change of fluid volume in soil void. If the compression growth of aquifer is positive, so:

$$\frac{\partial (n\rho_w dxdydz)}{\partial t} = \frac{\partial (\rho_w \varepsilon_v)}{\partial t} dxdydz \quad (15)$$

in the formul, ε_v is the volume strain of soil.

According to the law of conservation of mass, the change of total water quantity per unit time of microelement body is equal to its mass change, so:

$$\left(\frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} + \frac{\partial v_z}{\partial z}\right)\rho_w dxdydz = \frac{\partial (\rho_w \varepsilon_v)}{\partial t} dxdydz$$
(16)

The upper formula is the continuity equation of seepage field obtained from the law of conservation of mass.

By substituting Darcy's law formula into the above formula, the basic equations that must be satisfied in the solution region in the seepage field can be obtained:

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x} + k_{xy}\frac{\partial H}{\partial y} + k_{xz}\frac{\partial H}{\partial z}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial H}{\partial yx} + k_y \frac{\partial H}{\partial y} + k_{yz}\frac{\partial H}{\partial z}\right) + \frac{\partial}{\partial z}\left(k_{zx}\frac{\partial H}{\partial x} + kz_{xy}\frac{\partial H}{\partial y} + k_z \frac{\partial H}{\partial z}\right) = -\frac{\partial \varepsilon_v}{\partial t}$$
(17)

When the main direction of seepage flow is consistent with the direction of coordinate spindle, the formula above can be simplified as follows:

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial H}{\partial y}\right) + \frac{\partial}{\partial z}\left(k_z \frac{\partial H}{\partial z}\right) = -\frac{\partial \varepsilon_v}{\partial t}$$
(18)

In homogeneous media, $k_x = k_y = k_z$, so:

$$\frac{k}{\gamma_w}\nabla^2 u = -\frac{\partial \varepsilon_v}{\partial t} \quad (19)$$

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III. ANALYTIC SOLUTION OF SETTLEMENT OF OVERLYING SOIL LAYER CAUSED BY PRECIPITATION AFTER APPLICATION OF AQUICLUDE

After the project is constructed as aquifuge, there is no direct hydraulic connection between the groundwater in the upper layer of aquifuge and the groundwater in the lower layer of aquifuge, and the groundwater in the lower layer of aquifuge is converted from phreatic water to confined water.Confined water is the gravity water between two impervious layers or aquitards. Next, the settlement of overlying soil layer caused by confined water precipitation will be calculated.

By using the Mindlin displacement method, the settlement of overlying soil layer caused by precipitation under vegetation is calculated.

3.1 Calculation of seepage flow

Based on Darcy's law, the French hydraulic scientist Dupuit put forward the formula of steady seepage of groundwater into the well plane， called Dupuit formula.

Dupuit put forward the following assumptions for the precipitation of confined water wells:

I.The confined water layer is homogeneous, horizontal equal thickness and isotropic;

II.The groundwater is laminar flow. When pumping, the groundwater movement is stable, which conforms to Darcy's law;

III.The dewatering well has a fixed head recharge boundary.

IV.The roof and floor of confined water are completely water-resisting;

$$Q = \frac{2\pi skM}{\ln \frac{R}{r}} \quad (19)$$

Q —— dewatering well flow;

s —— depth of confined water precipitation;

k —— permeability coefficient of confined water layer;

M —— the thickness of the confined water layer;

R —— radius of influence;

r —— the radius of the measuring point.

Then the depth of confined precipitation is:

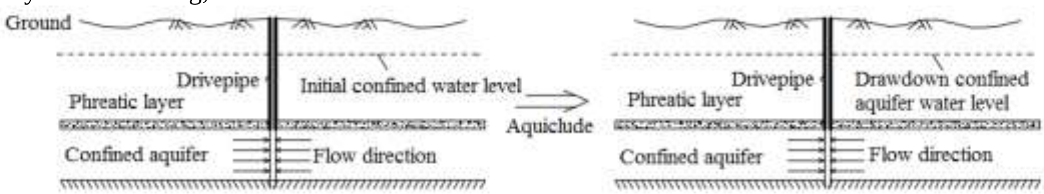
$$s = \frac{Q}{2\pi kM} ln \frac{R}{r} \quad (20)$$

The influence radius R determined by Siechardt formula:

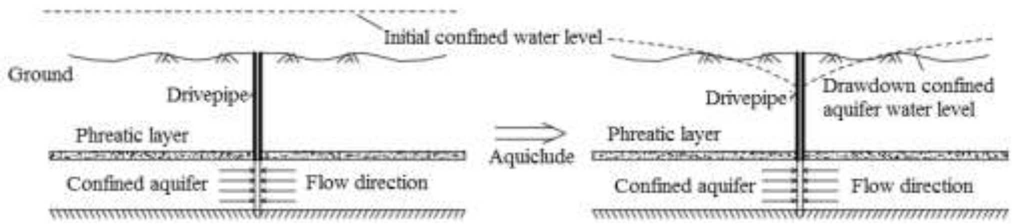
$$R = 10S_w\sqrt{k} \quad (21)$$

3.2 Additional force of confined water precipitation on overlying soil of aquiclude

When the dewatering well is used to depressurize, the confined water roof is regarded as a complete water-resisting layer, and there is no hydraulic connection between the phreatic layer and the confined water layer. n addition, a casing is installed in the upper part of the aquiclude to prevent the water in the soil above the aquiclude from being pumped away. As a result, the head of the phreatic layer remains stable and would not decrease with the decrease of the head of the confined layer. The water head curve after dewatering is shown in Figure2.



a)Change of Submersible Head



b)Change of Confined Head

Figure 2 Water head curve after precipitation

Before dewatering, the confined water has a certain supporting effect on the upper soil, which is proportional to the confined water head. After dewatering, the supporting force of confined water on the upper soil decreases, and the decrease is proportional to the decrease of water head. It is equivalent to adding a downward additional force to the upper soil. Because the magnitude of the force is proportional to the value of the confined water head, the pressure head of the dewatering well falls in the funnel shape with the center of the dewatering well, so the distribution of the additional force is also funnel shape, and the distribution diagram is shown in figure 3.

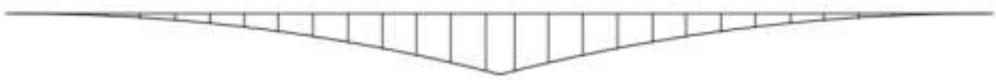


Fig .3 Distribution of Additional Force in Upper Soil Cover

Gong Xiaonan calculates the value of the additional force F in《the analysis of the settlement of overlying soil layer caused by pressure drop of confined water》.

$$F = \gamma_w \frac{Q}{2\pi T} ln \frac{R}{r} \left(r_w \le r \le R \right) \quad (22)$$

r_w—— the radius of the dewatering well.

This paper holds that there is no hydraulic connection between containing water in the upper soil and confined water in the lower soil, and when the additional force acts on the upper soil, the soil will produce Excess pore water pressure. The pore water pressure coefficient is proposed by Skapton combined with axisymmetric triaxial compression test. When pore water is not allowed to flow in and out of soil, the ratio of excess pore water pressure caused by additional stress increment to additional stress increment is called pore water pressure coefficient. The formula for calculating pore water pressure in Skipton is:

$$\Delta u_2 = B \cdot A \Delta \left(\sigma_1 - \sigma_3 \right) \quad (23)$$

$$B = \frac{1}{1 + n \dfrac{C_f}{C_{sk}}} \quad (24)$$

Δu_2 —— excess pore water pressure increment;

C_f —— pore fluid volume compression coefficient;

C_{sk}—— the volume compression coefficient of soil skeleton;

n—— porosity of soil;

$\Delta\left(\sigma_1 - \sigma_3\right)$—— additional stress.

This paper holds that excess pore water pressure caused by additional stress of overlying soil due to dewatering of confined water under aquiclude is as follows:

$$\Delta u_2 = B \cdot A (\gamma_w \frac{Q}{2\pi T} ln \frac{R}{r}) (r_w \le r \le R)$$

(25)

The effective force generated by the additional force is:

$$F' = \left(1 - B \cdot A\right) \gamma_w \frac{Q}{2\pi T} \ln \frac{R}{r} \left(r_w \le r \le R \right)$$

(26)

3.3 Calculation of settlement of overlying soil layer caused by tunnel excavation after application of aquifuge

The phreatic water level of the overlying soil layer does not change before and after the dewatering of confined water, so the consolidation settlement caused by phreatic seepage is not considered. In this paper, Mindlin displacement solution is used to calculate the ground settlement caused by additional force. Because the Mindlin displacement solution is aimed at homogeneous semi-infinite space, it meets the requirements of this paper. As shown in figure 4, in the Mindlin displacement solution, if the semi-infinite body is subjected to vertical concentrated force F at depth h, so the settlement at $M\left(x, y, z\right)$ is:

v ——Poisson's ratio of soil;

h ——Depth of action point of concentrated force;

$R_1 = \sqrt{x^2 + y^2 + (z-h)^2}$,

$R_2 = \sqrt{x^2 + y^2 + (z+h)^2}$

The additional force of confined water on the overlying soil layer after dewatering is axisymmetrically distributed at the bottom of the overlying soil layer. h is the thickness of the overlying soil layer. When the above formula is transformed into polar coordinates, the settlement at any point of the surface caused by the additional force s_x is as follows:

$$S_x = \iint_M s_x' r dr d\theta = \int_0^{2\pi} d\theta \int_0^R s_x' r dr \quad (26)$$

s_x' —— The settlement of f at the distance x from the center of the surface;

As shown in Figure 5, shaft shift is adopted. The displacement is produced by the concentrated force $f(a,b,h)$ at $(0,0,0)$. By using shaft shift, The displacement is produced by the concentrated force $f(0,0,h)$ at $D(r,0,0)$, so the s' is as follows:

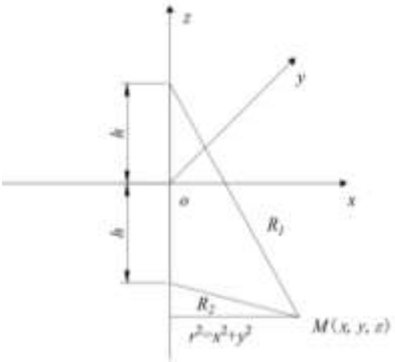


Figure 4 Mindlin Displacement

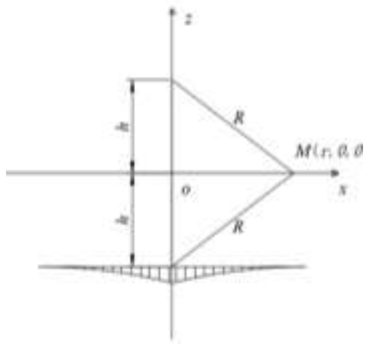


Figure 5 Coordinate Transformation Diagram

$$s_x' = \frac{f(1+v)}{2\pi E}\left[\frac{2(1-v)}{((r\cos\theta - x)^2 + (r\sin\theta)^2 + h^2)^{\frac{1}{2}}} + \frac{h^2}{((r\cos\theta - x)^2 + (r\sin\theta)^2 + h^2)^{\frac{3}{2}}}\right] \quad (27)$$

Combined formula (27) can be obtained by substituting non-uniform load $F = f'$ into integral

$$s_x = \gamma_w \frac{(1-B\cdot A)(1+v)Q}{4\pi^2 EkM}\int_0^{2\pi}\int_0^R ln\frac{R}{r}\left[\frac{2(1-v)}{((r\cos\theta - x)^2 + (r\sin\theta)^2 + h^2)^{\frac{1}{2}}} + \frac{h^2}{((r\cos\theta - x)^2 + (r\sin\theta)^2 + h^2)^{\frac{3}{2}}}\right] r dr d\theta \quad (28)$$

As a result of the axial symmetry, the settlement at the center point is the largest. So the displacement at the center point is

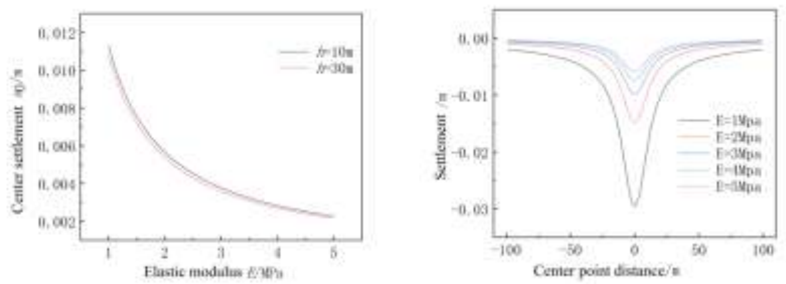
$$s_x = \gamma_w \frac{(1-B\cdot A)(1+v)Q}{2\pi EkM}\int_0^R ln\frac{R}{r}\left[\frac{2(1-v)}{((r\cos\theta - x)^2 + (r\sin\theta)^2 + h^2)^{\frac{1}{2}}} + \frac{h^2}{((r\cos\theta - x)^2 + (r\sin\theta)^2 + h^2)^{\frac{3}{2}}}\right] r dr \quad (29)$$

According to formula (29), the surface settlement displacement is related to the thickness and the elastic modulus of the overlying soil layer, the permeability coefficient and precipitation of confined water and so on.

3.4 Analysis of the influence of parameters on the settlement of overlying soil

In this paper, the control variable method is used to analyze the influence of various parameters on the overlying soil settlement.

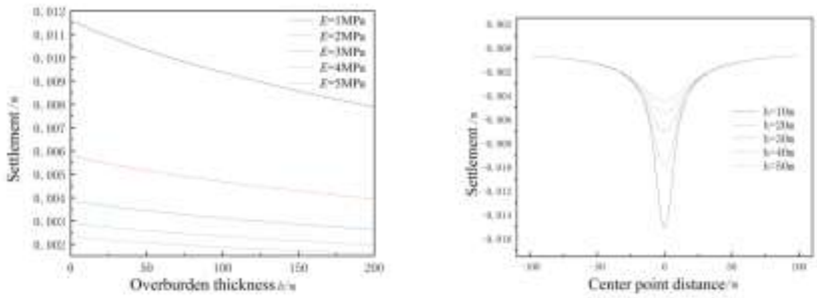
① elastic modulus of the overlying soil layer



(a) Settlement Of Central Point (b) Settlement Of Transverse Surface

Fig .6 Relationship between settlement s and elastic modulus E

Figure 6 shows the relationship between the settlement of the center point and the elastic modulus E when (a) $h=10m, h=30m$ and (b) $E=1-5MPa$. Figure 8(a) shows that, s_0 ,the settlement of the center point decreases with the increase of elastic modulus. When E rises from $1MPa$ to $2MPa$, s_0 is reduced by 20% less, which is a large drop. As the E continues to grow, the range of changes of s tends to be flat. According to formula (29), the elastic modulus of overlying soil is independent of the integral variants, and s is inversely proportional to E . When other parameters are fixed, the greater the elastic modulus of the overlying soil are, the smaller the central settlement is. Figure 6(b) shows that with the elastic modulus of the overlying soil decreases, the influence range of precipitation has also been expanded.



(a) Settlement Of Central Point (b) Settlement Of Transverse Surface

Fig .7 Relationship curve between s , the settlement of central point, and h , the thickness of overlying soil layer

Figure 7 shows the relation curve between s_0, the settlement of the central point, and h , the thickness of the overlying soil layer when (a) $E=1-5MPa$ and (b) $h=10-50m$ .According to figure7, with the increase of the thickness of the overlying soil layer, the settlement of the central point gradually decreases, indicating that the deeper the confined water aquifer is, the smaller the impact that the additional stress generated after precipitation on the surface settlement is. The larger the elastic modulus of the overlying soil is, the greater the variation range of the settlement of the central point with the thickness of the overlying soil is. The figure(b) shows that although the thickness of H has great influence on the settlement, it has little influence on the range of influence, and the settlement mainly

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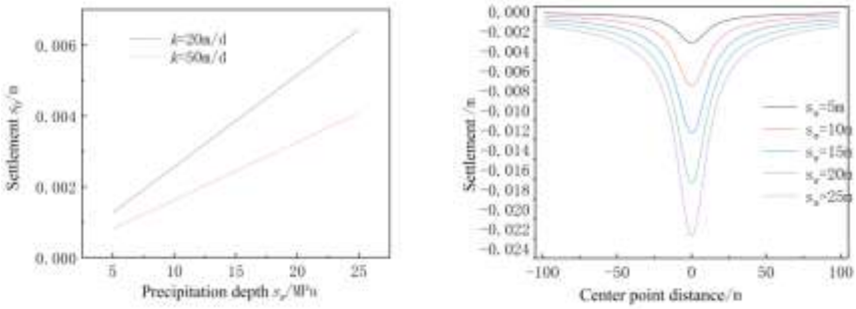
Volume 3, issue 6 June 2021, pp: 1484-1492 www.ijaem.net ISSN: 2395-5252

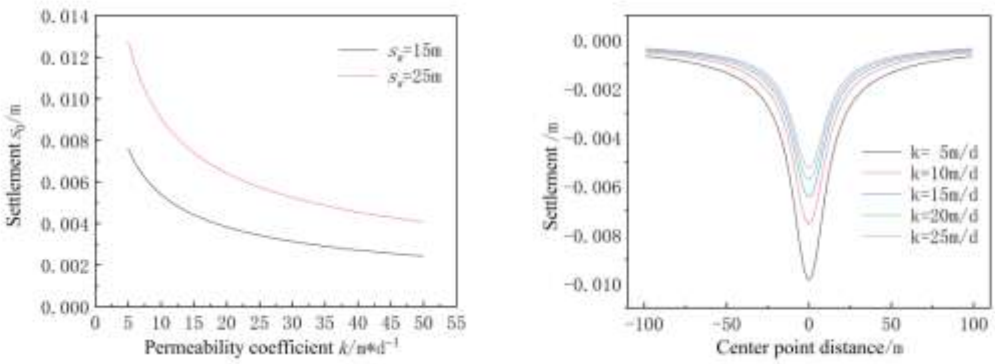
occurs at 25m near the central point.



(a) Settlement Of Central Point (b) Settlement Of Transverse Surface

Fig .8 Relationship curve between s ,central point settlement, and dewatering depth of confined water

Figure 8 shows the relation curve between s_0 , the settlement of the central point, and the dewatering depth of confined water under the conditions of (a) $s_w = 15m$, $s_w = 25m$ and (b) $s_w = 5-25m$. According to the figure, the settlement of the central point increases linearly with the increase of the dewatering depth of the confined water, and the increase rate is related to the permeability coefficient of the confined water. The smaller the permeability coefficient is, the smaller the influence of precipitation depth on the settlement is. When the precipitation depth is fixed, the smaller the permeability coefficient is, the smaller the settlement is.



(a) Settlement Of Central Point (b) Settlement Of Transverse Surface

Fig .9 Relationship curve between s ,central settlement, and k ,pore water pressure coefficient of confined aquifer

Figure 9 shows the relation curve between s , central settlement, and k , the pore water pressure coefficient of confined aquifer under the conditions of (a) $k = 20m/d, k = 50m/d$ and (b) $k = 5-25m/d$. According to the figure, the settlement is inversely proportional to the pore water pressure coefficient of confined aquifer. And the permeability coefficient of confined water layer has a certain influence on the increase rate of central point settlement with the dewatering depth of confined water. It's different that the dewatering depth of confined water does not affect the inverse coefficient of settlement and permeability coefficient of confined water.Figure (b) shows that the precipitation depth has a great influence on the range of surface subsidence.

In this paper, the integral of equation (27) is calculated by MATLAB software. The parameters of soil layer are as follows: thickness of overlying soil layer, $h = 5m$, elastic modulus of overlying soil $E = 3MPa$, Poisson's ratio of overlying soil layer $\upsilon = 0.28$, water bearing thickness of confined water $M = 20m$, permeability coefficient $k = 20m/d$, pumping flow of single well

$Q = 3600\, m^3 / d$, dewatering depth of confined water $s_w = 15m$. The result of settlement of center point calculated by Matlab software is $0.0028m$.

①The lower boundary of the aquiclude is regarded as aquiclude without thickness, and the part above the lower boundary of the aquiclude and the fill above the aquiclude are regarded as the overlying soil layer of the aquiclude with homogeneous soil. Because the elastic modulus of the aquiclude is greater than that of the fill, the increase of the thickness of the aquiclude will increase the elastic modulus of the overlying soil layer, which will help to reduce the settlement of the overlying soil layer. Therefore, the thickness of aquiclude should be increased within the allowable range.

②In the same way, increasing the elastic modulus of aquiclude will reduce the settlement of overlying soil.

③The settlement of overlying soil decreases with the increase of overlying soil thickness. Increasing the depth of aquifuge, same as increasing the thickness of overlying soil layer, will reduce the settlement of overlying soil layer.

④The settlement of overlying soil increases with the increase of confined water precipitation. Therefore, reducing the height and amount of groundwater precipitation under the aquiclude can meet the requirements of tunnel construction.

IV. CONCLUSION

(1) The basic theory of seepage is introduced;

(2) The settlement of the overlying soil layer caused by precipitation under the aquiclude is calculated, the influencing parameters of the settlement are analyzed, and the calculation of the center point settlement is extended to the calculation of the horizontal surface settlement.

V. ACKNOWLEDGEMENT

This paper is Sponsored by project of Research on key technology of safety construction of super shallow buried small clear distance large section river crossing tunnel in soft interbed (020518C1105002502000201800002Z004), here thank them.

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