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Design of excavation and tunnelling in soft ground in China

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1 INTRODUCTION

The total length of Shanghai Metro. Line No.1 Project is 14.57km., with one ground station and eleven underground stations. The diameter of shield driven tunnel is 6 meters. Reversed Construction method was used to construct the stations under Huaihai Road. Side walls of the stations are diaphragm walls with multi-storied structures between them. The cross-sections of Xingzha Road Station and the running tunnel are shown in Fig.1-1.

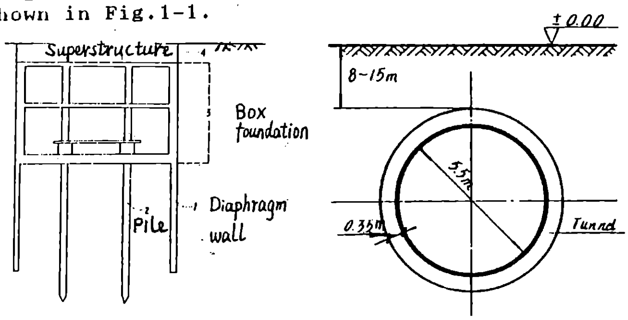


Fig. 1-1 Cross-section of Xingzha Road Station and tunnel

The first and second line of Beijing Metro. were constructed by Cut and Cover Method, with sheet pile or soldiers wall, the third line is being constructed by mining method, such as NATM.
High-rise buildings were constructed in larg numbers since the 70s in China. New Century

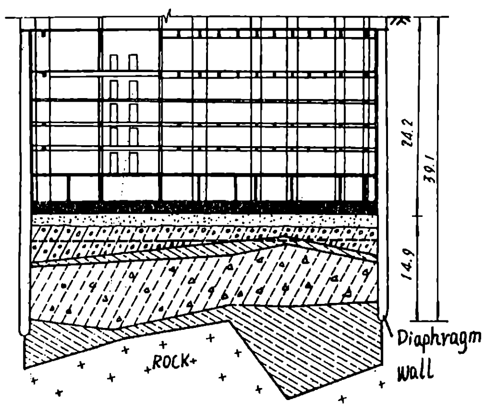


Fig. 1-2 Six-storey basement of New Ceatury Mansion

Mansion in Fuzhou is being built with 45 stories above ground and 6 stories under ground, which is one of the deepeest in Asia.
Most of thr above mentioned projects are constructed in soft ground which is widely distributed in east coastal cities. There exists soft soils deposited in salt-watered fresh water in Bohai Gulf, Tanggu, Yangtse River Delta, Zhejiang Province, Zhujing River Delta and coastal region of Fujian Province. The soft soils are mixtures of organism and mineralnism, with characteristics of soft, big void ratioed, high compressibilied and low strengthed and thickness of several meters to tens of meters. But in the same region, the thickness changes little and strata are distributed in stripe.
Shallow soils in Shanghai region are soft Quatenary deposits whose geological age is not long and the degree of consolidation is low. Free water surface is very shallow, the depth of water level is only 50 to 70 centimeters and the soil distribution has some degree of regularity. The effective internnal friction angle of mucky soil in Shanghai is 26° to 30° whereas the degree of consolidation is 100 percent. The internal friction angle determined from consolidated quick test is 8° to 12° whereas the degree of consolidation is about 25 to 40 percent. The latter one is similar to the shear strength determined in vane shear test. Plastic Index of most soils in China is lower than 35, so it's suitable to determine shear strength through vane test.
Braced walls include structure wall and soil improvement wall, the former one uses the structural wall to resist external forces, the latter one makes full use of the strength of consolidated soil.

1. The types of wall include sheet pile wall, diaphragm wall, steel sheet pile and so on.
2. The types of consolidation include soil mixing, high presure jet grouting pile, grouting, root piles and so on.

Bracing system includes braced type, tie back anchor type and reverse construction method, Soil anchor has been fully studied and widely used in some soft soil projects of China.
Tunnelling includes NATM and shield driven method. In soft soil, when the ground can't bear its own weight, shield has to be adopted. Since the 50Ss, tunnels with diameters of 2 to 11 meters have been completed in China. For example, the total length of the subaqeous tunnel of Yanan Road East in Shanghai is 1476 meters. It passes through mucky stratum. The smallest thickness of covering in bottom is 5.8 meters. Exteral diameter of the tunnel is 11 meters. It consists of eight concrete segmemt

with width of 1 meter and thickness of 0.55 meter.
The flexible lining is no longer a structural arch but a membrane. The low bending rigidity and high axial rigidity can transfer passive resistance of strata. The shot concrete of NATM has been commonly recognized and widely used. The flexible lining of shield tunnel has the same function by means of reducing both thickness and joint rigidity and increasing joint numbers.

2 Excavation

2.1 Water and earth pressure

Active pressure should be calculated as lateral pressure, however, to long term retaining structure and those tightly surported in time, at rest pressure could be used. To cohesive soil, cohesion C leads to low permeability, therefore the lateral pressure of water and soil should be calculated together,namely to multiply the vertical soil pressure and water pressure by lateral coefficient. To sandy soil, high permeability resulting from ignorable cohesion C makes the function of water pressure more obvious. Hence lateral pressure of water and soil should be calculated seperately, namely to multiply the vertical soil pressure by lateral coeficient, and then add the value of water pressure.

2.2 In conventional design formulae with $\phi=0$ or C=0 is available. On the basis of our practice and reseach, the formulae considering C and o simultaneously has been suggested and written into Shanghai Foundation Code.

1. C and ϕ method

The two factors namely c and ϕ are involved in shear strength simultaneously in the case of common cohesive soil. According to Parandtl's or Terzaghi's formula of subsoil bearing capacity, the ultimate bearing capacity slip curve is given in Fig. 2-1.

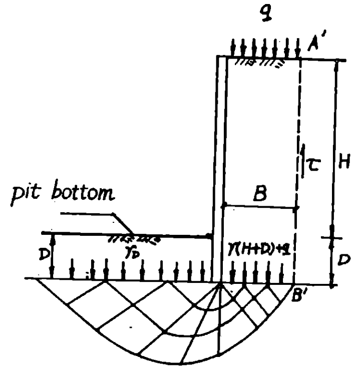


Fig. 2-1 Stip curve considering C and ϕ simultaneously

Safety factor against upheaving is suggested as:

$$K_s = \frac{\gamma D N_q + C N_c}{\gamma(H+D)+q} \qquad (2\text{-}1)$$

Where: D is the embedment depth, H is excavation depth, r is the unit weight of soil, C indicates cohesion of soil under the pit, q is surcharge on surface , and N_q, N_c are coefficients of subsoil bearing capacity.
In Prandtl's formular:

$$N_{qP} = tg^2(45°+\phi/2)e^{\pi tg\phi}$$
$$N_{cP} = (N_{qP}-1)/tg\phi \qquad (2\text{-}2)$$

In Terzaghi's formular:

$$N_{qT} = \frac{1}{2}\left[\frac{e^{(\frac{3}{4}\pi-\frac{\phi}{2})tg\phi}}{\cos(45°+\phi/2)}\right]^2 \qquad (2\text{-}3)$$
$$N_{cT} = (N_{qT}-1)/tg\phi$$

Where: K is bearing resistance coefficient. In Eq. (2-2), K $\geqslant$ 1.1 to 1.2; in (2-3), K $\geqslant$ 1.15 to 1.25.

2. Model test method

For bottom upheaving , load q, C, ϕ ,r of subsoil, embedment depth D, and excavation depth H, Tongji University has set up the following relationship through model tests. Having determined allowable bottom upheaving on the basis of given geological and environmental conditions, D can be determined. on the contrary, from D we can get. The empirical formular is shown as:

$$D/h = \frac{1}{[0.08[\delta]+2.33+0.0134\gamma H'-0.423\gamma C^{-0.04}(tg\phi)^{-0.54}]^2} \qquad (2\text{-}4)$$

where: [δ] is allowable bottom upheaving; H'=(H+p/r),it is the substitutive height. C. ϕ.r are the cohesion, interal friction angle and unit weight of the soil. P is the surcharge on surface.

2.3 Calculation methods for pit supporting

With regard to the development of calculation, the earliest is the classic methods. Soil pressure is known and the deformation of wall and struts are not considered. In another method later developed, strut deformation isn't involved, yet displacement of the wall is considered. Two sorts of finite element method are introduced:

1. Axial forces of lateral struts and bending moments of wall change with displacement of wall and deformation of struts. This method will take soil pressure as known, generally using bar system FEM.
2. Considering that soil pressure changes with the displacements of wall and struts, plane system FEM is used, so bottom upheaving and surplace displacements can be determined. A bar system will be discribed as following. It involves many factors, such as lateral soil pressure changing with structure displacement, deformation of structure and struts, and excavation process interfering internal forces and displacements of structure. This method, compared with those not considering the factor of wall displacements and strut displacemnts, is more practical.

In case of soft soil in the coastal area, active earth pressure coefficient is generally close to at rest earth pressure coefficient. In situ meamurement papers have also proved this point. So in such area. in case of multi-strut surported structure without precompressed ,

earth pressure acting on the back of the structure can be considered as active earth pressure. However, to front side earth pressure under the bottom, elastic resistance method should be used, due to the pressure less than passive pressure. Soil resistance is proportional to the displacement, "m", "k", "c" method could be adopted.
The relation of surport structure displacement, the increase of lateral earth pressure and excavation process is given in Fig. 2-2.

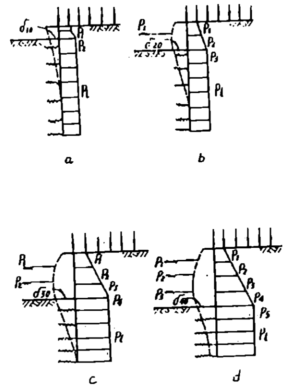


Fig. 2-2 Relation of structure displacement and excavation process

Calculation steps:
1st step: Set up bending curve equations of individual part of the pile.
2nd step: Calculate lateral earth pressure on the pile and the corresponding value of loading function.
3rd step: According to embedment depth L of piles related to certain excavating depth, calculating $\lambda=\alpha l$ to determine the parameters in the general solution of different points of the bending curve, α is deformation coefficient of the pile. $\alpha=\sqrt[5]{m/EI}$, in "m" method; $\alpha=\sqrt[4]{K/EI}$, in "k" method.
4th step: determine the initial deformation δ_{10} at the second strut through loading function for the first stage and the corresponding parameters without dimention.
5th step: calculate initial deformation δ_{20} at the third strut, struts forces and the biggest bending moment using the value of the second stage loading function, the related non-dimention parameters and δ_{10} .
6th step: do as the fifth step to calculate the values of the third stage, then the fourth stage stage, calculate the displacements of every strut, substract the corresponding initial displacements, and get the practical compresive deformation according to which any strut force can be determined, thus the deformation and internal forces can be calculated.

3 Design of tunnel

3.1 Measured ground pressure

The 11 meter subaqeous tunnel is buried in soft strata and covered with 8 meters of soil and 13 meters of water. The open area of the breast plate is about 4 percent. Under such geological and construction conditions, measured ground pressures are close to theoretically calculated loads.

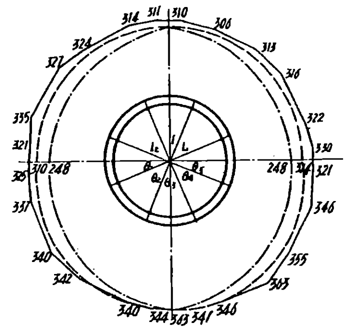


Fig. 3-1 External loads of a tunnel lining

3.2 Design theory

Design loads acted on a shield tunnel include ground loads, self weight G of structure, vertical and horizental earth pressures P_V and PH, lateral earth resistance P_K , external water pressure P_W of the lining, internal loads of structure, bottom pressure R, and construction loads.

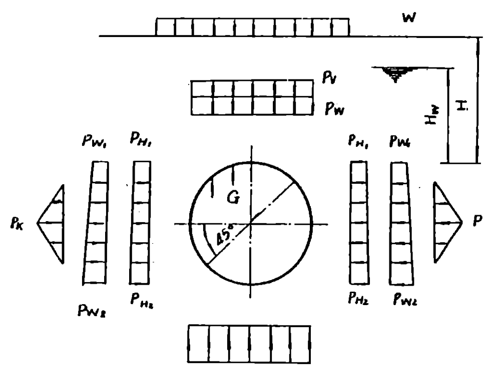


Fig.3-2 Design loads on a sheld tunnel

Loads on a sheld tunnel are as follows:
1. Vertical earth pressure in cohesive soil is calculated in light of the total overburden pressure. In sand, it is calculated by either unloading theory or the total overburden pressure according to practical condition.
2. Lateral earth pressure in cohesive soil strata is commonly calculated by adding soil pressure and water pressure, and then take lateral earth pressure coeffcicent as 0.65 to 0.75. In sand strata, it is obtained by seperating soil pressure from water pressure, the active earth pressure coefficient can be used as lateral earth presure coefficient.
3. Under the condition that tunnel lining is flexible and rigidity of strata is big (N63.5 > 2 to 4), it's suitable to take earth resistance P_K caused by lining deformation into consideration. The diagram of resistance is assumed to be a triangle as shown in Fig. 3-2. The resistance can be computed with subgrade modulus:

$$P_K=K\ Y \qquad (3-1)$$

where: P_K is earth resistance; K is subgrade modulus; Y is displacement of lining at the spring points.
Computation method of a fabricated ring structure can be determined according to joint construction method. If it's saturated soft stratum whose elastic resistance is small or or lateral loads are known according to the existed local experience, the tunnel can be calculated as a monolithic uniform free deformation ring which is shown in Fig.3-3. It's assumed that the horizental projection of earth resistance around the ring is uniformly distributed.

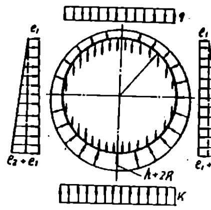
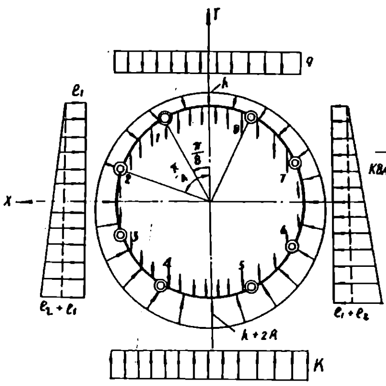


Fig. 3-3 Monolithic uniform free deformation ring

Elastic center method is used to calculate internal forces. Both the structure and the loads are symmetric.
According to the condition that there isn't any relative angular and horizental displacements in the place of elastic centre, force method equations can be listed and the internal forces can be calculated out.
Because the joint rigidity can't be equal to that of segment cross section, so the ring can't be considered as a coutinum. In analysing the internal forces of a ring, it's necessary to consider the effect of joint rigidity. In fact, there exists an elastic hinge which can bear part of bending moment. An elastic hinge is neither an rigid one nor a complete hinge. The amount of bending moment it can bear is directly proportional to the joint rigidity. Computation shetch of the ring with elastic hinge is drawn in Fig.3-4 and Fig. 3-5.



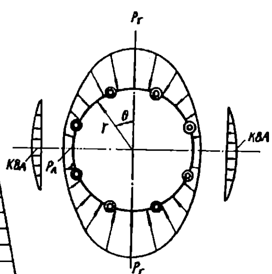


Fig. 3-4 Fig. 3-5 Computation sketch

Computation sketch of the lining is shown in Fig. 3-4. The indeterminate structure is an uniform ring with elastic hinges. The hinges are uniformly arranged. The structure is symmetic to axes of X and Y, but loads are only symmetic to axis of Y. In order to analyse a uniform ring with elastic hinges and compute it with an computer, force method can be used to analyse internal forces of the structure. Fig. 3-6 is the baic computation diagram.

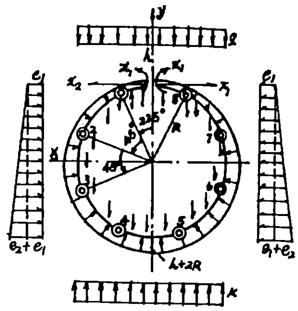


Fig. 3-6 Basic computation diagram.

Force method equations can be established:

$$\begin{cases} \delta_{11}X_1 + \delta_{12}X_2 + \Delta_{1P} = 0 \\ \delta_{21}X_1 + \delta_{22}X_2 + \Delta_{2P} = 0 \end{cases} \quad (3\text{-}2)$$

It's assumed that:

$$\Delta = \delta_{11}\delta_{22} - \delta_{12}\delta_{12}$$
$$\Delta_x = \delta_{12}\Delta_{2P} - \delta_{22}\Delta_{1P}$$
$$\Delta_y = \delta_{12}\Delta_{1P} - \delta_{11}\Delta_{2P}$$

X1 and X2 can be calculated out:

$$\begin{cases} X_1 = \Delta_x/\Delta \\ X_2 = \Delta_y/\Delta \end{cases} \quad (3\text{-}3)$$

In the above-memtioned equations, the coefficents δ_{12}, δ_{11}, δ_{22}, Δ_{1P} and Δ_{2P} can be calculated according to their physical and mechanical meanings which include joint rigidity of elastic hinges. Rigidity coefficient K reprents the joint rigidity of an elastic hinge and it can be seperated into three parts: coefficient of bending rigidity K_θ, coefficient of axial rigidity K_δ and coefficient of joint chean rigidity K_γ. K_θ (K_δ, K_γ) can be defined as bending moment (axial force, shearing force) in the direction of deformation under unit rotaty angle (axial displaceement, shear deformation). Model test and prototype test can be carried out to determine joint rigidity of the ring. Under the action of given axial forces and bending moments, the joint déforms correspondingly. In time of the joint being accentrically loaded, the relation between rotary angle θ and bending moment M is shown in Fig.3-7. Joint rigidity coefficient K_θ can be calculated form the experimental results in Fig. 3-7.

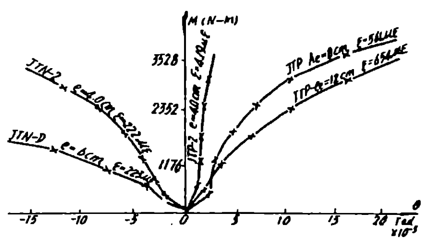


Fig. 3-7 M-θ Curve of joint when accentriclly loaded

4 Ground movement

4.1 Ground movement: Pattern for excavation

After excavating and bracing, the horizontal displacement of wall owing to earth pressure is induced and meanwhile ground surface will settle down, As a result of the release of initial stress, bottom heave will arise. In addition,consolidation settlement of the ground caused by dewatering measures will arise.

Calculating method for ground morement of excavation:

1. The FEM

We cannot get the result of ground settlement with bar system FEM, but with continum FEM we can.With conventional method, (i.e. defining boundary condition and soil's constitutive relation, then dividing into grid), the result will be considerably different from that of fact. The following aspects is very important in calculating ground movement with FEM. So we deal with it as follows:

1st problem: The effect of construction time

If strut is not placed in time, the displacement of the wall and the ground settlement will increase. During constructing, deep excavation of subway station in Xujiahui, Shanghai, field tests are carried out to measure the increasing settlement due to delayed bracing. shown as follows:

$$\Delta S_{max} = \Sigma\, d_i t_i \qquad (4\text{-}1)$$

Where: d_i is increasing displacement owing to one day's delay of the 1st strut; t_i is the delayed days of the 1st strut.

Through measuring, we can get that at the place of second strut d_2=2.3mm/day ; at the 3rd strut d_3=2.4mm/day; at the 4th strut d_4=3.4mm/day; at the 5th strut d_5 =4.1mm/day; at the bottom, d =1mm/day.

2nd problem: Effect of interface element

When we analyze mutual action of ground soil and wall with finite element method, we always assume the interface element between the wall and the ground is so coarse that there is no relative slide between ground and wall; or we can assume the interface is so smooth that there is no shear stress to prevent relative movement between them. Under this assumption the soil outside the underground wall will cause movement which is not in accordance with practice. In fact the mutual action is between the two assumptions. To simulate the transmission ability of a limit stress on the interface of these two lergely different materials. If we analyze with interface element, we will get a satisfactory result.

3rd problem: Effect of residual stress.

Soil stress measurment carried by Japan and Shanghai shows that even after soil is dredged, soil under the excavated surface will still remain quite a part residual stress. The unloaded effect of upper ground to the stress of deeper ground only exists in a definite depth and scope. Under this depth, stress changes little. If we do not consider this case, the displacement of soil around excavation and bottom heave value which we caloulate will be higher than measurement. For example, a 17m.-depth excavation of a subway station in Shanghai gets a bottom heave about 10 cm. But we will get a result about 51.40cm without considering residual stress. When we estimate it with consideration of residual stress, the result will be satisfactory.

2. Ground-loss method.

Based on consolidation theory of Biot, we can do some simulant calculation for deep excavation with FEM and infinite element method.There are two types of surface settlement curve, one is triangular type, the other is parabole type. We always think that the area of ground settlement is approximately eqyel to that of the lateral displacement of bracing. The scope of ground settlement may be thought as the width of slide wedge at the back of braced structure.

1st type: for triangular type: the scope of surface settlement is:

$$x_0 = Hg\ tg(45^\circ - \varphi/2) \qquad (4\text{-}2)$$

The maximum settlement is on tne edge of the excavation and it is about $2S_\omega/x_0$. The calculating model is shown in Fig. 4-1.

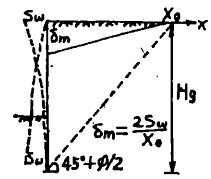


Fig. 4-1 Estimation model of triangular ground movement

2nd type: for parabole type: we can estimate the exponential function of surface settlement as shown in Fig. 4-2 (a) (b) . The form of exponential function may be:

$$\delta_{s\sigma}(x) = a\left\{1 - \exp\left(\frac{x + x_m}{x_0} - 1\right)\right\} \qquad (4\text{-}3)$$

Where: a is a constant of displacement to be calculted; Xm is the place where maximum surface settlement lies. (distance from the edge of excavation).

1st condition:

$$\delta_{s\sigma}(x)\big|_{x=x_m} = \Delta\delta \qquad (4\text{-}4)$$

2nd condition:

$$\int_0^{x_m} \delta_{s\sigma}(x)\,dx + \int_0^{x_0 - x_m} \delta_{s\sigma}(x)\,dx = S_\omega \qquad (4\text{-}5)$$

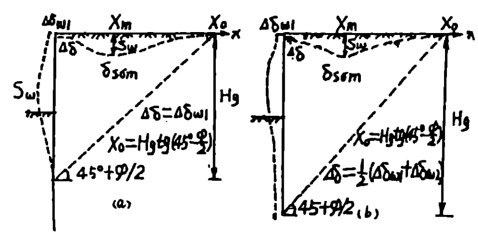


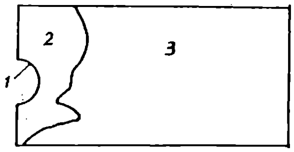
Fig.4-2 (a) (b) Estimation model of parabola ground settlement

4.2 Ground movement pattern for tunnel

The ground loss and the reconsolidation of remolded soil is the main reason of surface settlement. Two kinds of method to calculate ground movement is shown as follows.

1. Soil consolidation method with revised

Combridge Model, elasto-plastic FEM analysis of Biot consolidation theory and field test, we find that excessive pore water pressure will be indused in soil due to excavating and grouting, as shown in Fig. 4-3.



1---tunnel
2---area of increasing pore water pressure
3---area of decreasing pore water pressure
Fig. 4-3 Pore water pressure in soil around the tunnel

On the basis of effective stress principle, dissipation of the excessive pore water in the soil above tunnel will lead to corresponding increase of effective stress, and then causes consolidation settlement. The shape and scope of the settlement trench and inflection-point is similar to the calculation result by Peck Method. Its formulae is as:

$$\delta(x,t) = \left(\frac{V_l + H\bar{K}_x t}{\sqrt{2\pi}\cdot i}\right)\exp\left(-\frac{x^2}{2i^2}\right) \qquad (4-6)$$

2. Four-stage method.
From the analysis of ground stress field, we can see that ground will cause displacemnt all the time from in front of shield, surrounding shield, after shield tail, and even after a long period of time. The displacement curve of surface point above the tunnel axis of the 20th ring of Pu Dong, Yanan East Road in Shanghai is shown in Fig . 4-4. From the figure, the process of surface displacement can be divided into four stages at large: ealier-stage displacement, settlement when shield is advancing, settlement when shield tail has passed and the long term settlement.

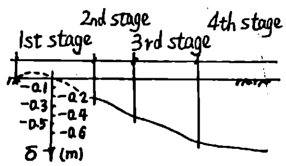


Fig. 4-4 Curve of settlement with time

The first stage: ealier stage displacement δ_1.
The disturbed ground and the released ground stress caused by the develution of excavative surface and excessive dredged soil will lead to ground settlement. The excessive static soil pressure owing to ground being squeezed by shield or dredged soil is not enough will result in bottom heave. We can conclude in Eq. (4-7) by mathematical statistic through the data corresponding to the average front soil pressure P and the ground settlement value 2d distance from it.

$$\delta_1 = (\bar{P}-650)/1560 \qquad (4-7)$$

The second stage: settlement δ_2 when shield is advancing.
The ground disturbance caused by shearing and squeeze to soil layers the deformation modulus of soil. Then the settlement will be produced, shown in Eq. (4-8):

$$\delta_2 = \Sigma \bar{\sigma}_{zi} h_i \left(1/E^R_{si} - 1/E_{si}\right) \qquad (4-8)$$

Where: $\bar{\sigma}_{zi}$ is soil initial stress of every layer; h_i is soil thickness of each layer; E^R_{si} is modulus of deformation after disturbance; E_{si} is the original modulus of deformation of every layer.
The third stage: settlement when shield tail has passed δ_3.
As a result of shield tail gap, the ground pressure releases which will lead to . According to the principle that the volume of ground settlement should be equal to that of tail gap (considering the actual volume of gap after the injection material has been used), we conclude:

$$\delta_3 = \frac{\sqrt{2\pi}}{4i}(1-\rho)\left(R_s^2 - R_R^2\right) \qquad (4-9)$$

Where: i is coefficient of settlement trench; H is depth of covering soil; R_T is the radius of tunnel; ρ is filled ratio; R_s is the outer diameter of tunnel; R_R is the outer diameter of segment; $i = (H+R_T)/[\sqrt{2\pi}\,tg(45°-\varphi/2)]$.
The 4th stage: long term settlement δ_4
It is owing to the dissipation of pore water, the creep of soil and the deformation of segments etc. It can be described with Voigt Model:

$$\delta_4 = \frac{R_T}{G}\cdot\tau_0\left(1-e^{-\frac{G}{\eta}t}\right) \qquad (4-10)$$

Where: G is modulus of shear deformation; τ_0 is coverage shear stress of the ground. η is modulus of cohesion; t is time.
The maximum total settlement of every point on the ground is the sum of the four parts:

$$\delta_{max} = \delta_1 + \delta_2 + \delta_3 + \delta_4 \qquad (4-11)$$

5 Monitoring and back analysis

5.1 Monitoring

Monitoring aims at:
1. According to certain measured data, measures can be taken to prevent engineering damage and accidents of circumstances.
2. Monitoring can be used to guide field construction, to determine and optimize construction parameters in order to carry out informationlize construction.
3. Field measured results can be used to improve design and carry out informationlize design.
The following is an example of monitoring.
There is a four-storey frame building near Shanghai Metro. Line No.1. The distance between them is only 2.49 meters. In order to reduce ground settlements and lateral movements the diaphragm wall in the process of excavation, several measures were take in design and construction; depths of four pieces of disphragm wall were increased to 24 meters from 20 meters; jet grouting was carried to consolidate the bottom soil.
The size of excavation is 12 metres in depth and 12 metres in width. Five struts were propressively installed at 1.7m, 3.0m, (field concrete), 4.42m., 7.3m. and 9.3m. below the ground elevation. In the process of construction, settlement instrument, ground

stakes, inclinometer, piezometres, pressure transducers and other instrument for measurement were installed. Fig. 5-1 is the plan of measuring points, cross-sections of excavation and frame building are shown in Fig. 5-2 and Fig. 5-3.

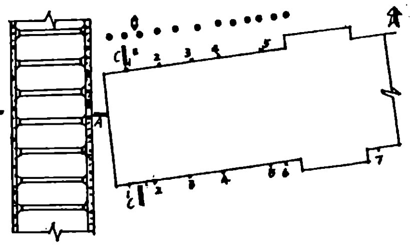
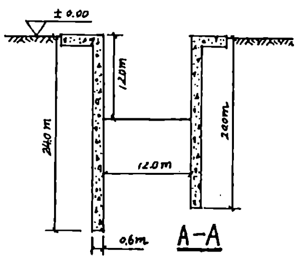


Fig. 5-1 Plan of measuring points



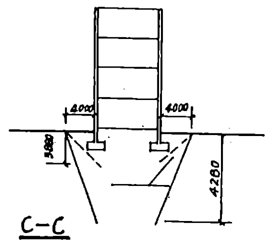


Fig. 5-2 Fig. 5-3 Cross-sections of excavation and building

Measuring works mainly include:

1. Horizental and vertical movements measureement of diaphragm wall

Horizental movement of the wall during every excavation stage can be measured with the aid of inclinometer buried in the wall. Vertical settlement can be measured with survey instrument from a great distance. Lateral movement curve of the diaphragm wall is in Fig. 5-4.

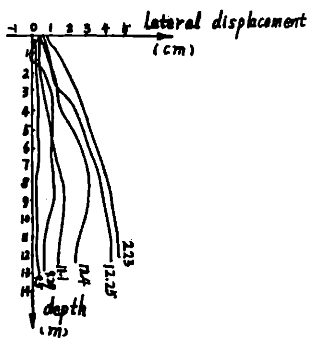


Fig. 5-4 Lateral movement curve of the disphragm wall

2. Ground settlement measurement

Ground settlement and deep earth movement can be measured with survey point and settlement instrument respectively. Fig. 5-5 shows the curve of ground settlement.

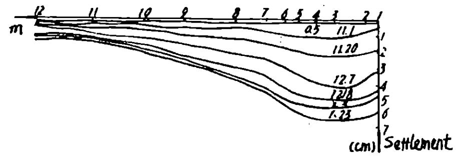


Fig. 5-5 Curve of ground settlement

3. Monitoring of the building

It included measurement of settlement and observation of internal cracks. The curve of building settlement is shown in Fig. 5-6.

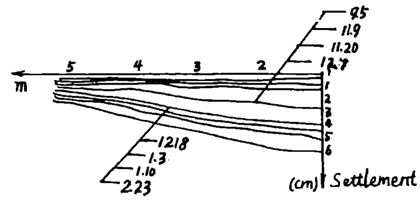


Fig. 5-6 Curve of building settlement (Northern side)

4. Measurement of axial forces of struts and pore water pressure. Laboratory tests were carried out on soil samples before and after consoildation.

5.2 Back analysis

Back analysis is to calculate mechanical parameters and loads acted on strata and structure reversly according to the in situ monitoring data such as displacements and stresses. The method of calculating mechanical parameters and loads reversly according to measured displacements is call displacemental back analysis. It has two ways called direct back analysis and indirect back analysis.

1. Direct back analysis.

With the aid of direct analysis method, direct back analysis can be achieved to make the calculated displacements close to the memeasured by adjusting mechanical parameters and loads. It's assumed the total rigidity matrix [K] is made up of functions of geometric and mechanical parametres being back analyzed, {u} is the total displacement vector and {p} is the total load vector. A set of displacement vector can be calculated by Eq. (5-1) when a set of mechanical parameters and loads are given.

$$[K]\{u\} = \{p\} \qquad (5\text{-}1)$$

But the calculated displacements are not necessarily accorded with the measured data, it's necessary to optimize parametres to minimarize the following error function.

$$f = \sum_{i=1}^{m} (u_i - \bar{u}_i)^2 \qquad (5\text{-}2)$$

Where: m is the number of measuring points; ui is calculated displacements ; $\bar{u}$i-measured displacements . In order to fullfill the above mentioned optimum, various direct searching methods can be used.

2. Indirect back analysis

Equations for calculating mechanical parameters and loads can be dirctly derived, measured displacements are considered as known condition. For example, in tunnelling, elastic modulus and initial earth stress can be inversly calculated according to measured displacements.

3. Example of back analysis

Engineering background is the example of monitoring above meationed. Soil parameters were calculated by back analysis according to in situ measured data when the excavation was carried to 10m. below the ground elevation. So the ground settlements and lateral displacements of the diaphragm wall when the excavation carried to 12m. below the ground elevation were perdicted. Fig. 5-7 gives the curves of perdicted and measured settlements in time of exvavating to 12 metres below the ground elevation; Fig. 5-8 gives the curves of perdicted and measured lateral displacements of the diaphragm wall.

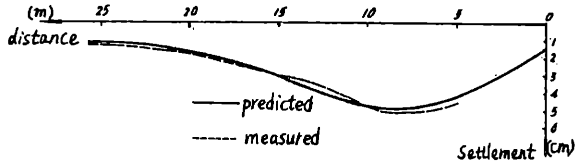


Fig. 5-7 Curves of ground settlements

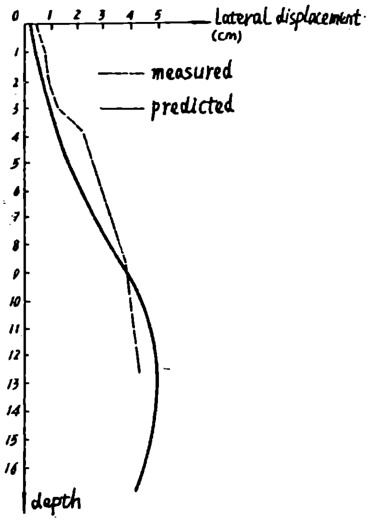


Fig.5-8 Curves of lateral displacements of the disphragm wall