BEHAVIOR OF AN EARTH RETAINING WALL DURING DEEP EXCAVATION IN SHANGHAI SOFT GROUND

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ABSTRACT

In this paper, simulation methods and measurement systems for earth retaining structures during deep excavation work in Shanghai, China are discussed. Comparisons between the measured and simulated data are made, and the inverted construction method for underground works is also evaluated.

The main results are as follows: (1) The maximum wall displacement was more than 10 cm in the first excavation stage, and reached a value of approximately 13 cm in the final stage excavated to a depth of 18 m below G.L. (2) In the simulation analysis, the stiffness of the ground on the excavation side was reduced to model the presence of vertical voids induced from the installation of plastic-board drains and other pre-excavation construction. The stiffness of the earth retaining wall was also reduced due to the cracks in the concrete. With these modifications, the results of the simulation analysis showed values quite similar to the measured data. (3) The measuring system worked flawlessly except for minor problems in the installation of the sensors and occasional breakdowns of the inclinometers. (4) During the excavation, real time simulations were conducted by using the most current measurement results to predict behavior in subsequent excavation stages. Consequently, the originally planned No. 4. temporary struts were eliminated, and the excavation was successfully completed to the designed depth.

Key words: case history, deformation, (earth retaining), excavation, measurement, soft ground (IGC: E2/H2)

INTRODUCTION

The Huangpu River, a branch of the Changjiang River (Yangtze River), flows through the center of Shanghai, China. The western side of the river is called the "Puxi" area, while the eastern side is called the "Pudong" area. The Puxi area has long been urbanized, but the Pudong area, which was formerly farmland, has been developing into a new city area since 1990. Figure 1 shows a city map of Shanghai and the location of the construction site.

In the Pudong area, many high-rise buildings have just been completed or are under construction. Among these Pudong projects, some have involved Japanese designers and construction engineers. From this experience, it is recognized that careful consideration of technologies, construction materials, labor circumstances and customs between the two countries are vital for efficient international technology transfer.

This paper examines the deep excavation work of a high-rise building project in the Pudong area. Simulation methods, measurement systems, comparisons between the measured data and the simulated data, and the applicability of underground construction methods are discussed. The adopted simulation methods and measurement systems have already been utilized in Japan by the

Fig. 1. A Shanghai city map and the location of the site

authors. However, the inverted construction method has never been applied in China before this construction.

The duration of the project was from April 1995 to the end of April 1998.

GEOLOGICAL CONDITION

Shanghai is located at the lower reaches of the Chang-

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Fig. 2. Geological profile

jiang River. Therefore the Shanghai ground mainly consists of very soft alluvial marine clay. Figure 2 shows the outline of the geological condition of the construction site by Kitajima et al. (1997). The Shanghai ground is generally classified into nine layers, numbered 1 through 9, where layer No. 9 consisted of diluvial deposits. The layer formation is uniformly distributed over the area except in some cases where some layers have different thicknesses.

The layers to a depth of 28.5 m below G.L., mainly consist of normally consolidated clay as shown in Fig. 2. The No. 3 layer and No. 4 layer to G.L. - 17.0 m are very soft, but below G.L. - 17.0 m, the strength increases with depth. Layers 7-1 and 7-2, deeper than G.L. - 28.5 m, consist of dense sandy confined aquifer. The water level in the clay layers is at G.L. - 1.0 m and the artesian head in the sand layer is at approximately G.L. - 9.0 m.

The characteristics of Shanghai alluvial clay is that the clay layers include sand seams, so that the cohesion of the clay is smaller than expected, and that the strength of the clay has been weakened by disturbance. In general, the natural water content is approximately 40%, less than that of Japanese alluvial clay in the alluvial plain, and the wet density is relatively large.

BUILDING OUTLINE

The building is mainly used for office spaces and shops. It can be considered to have two parts, namely a high-rise building part with 46 stories above ground and height of 207.55 m and a low-storied building part with two stories above ground. There are 4 underground levels used for parking lots and machine rooms with widths of 66 m and lengths of 91 m per floor. The foundation bottom for the high-rise part is located generally at G.L. - 17.75 m, but at the pit part (with width of 20 m and length of 20 m) the bottom is located at G.L. - 20.55 m. The foundation bottom of the low-storied part is located at G.L. - 16.75 m. The mat slab foundation was adopted and the foundation was supported by a total of 836 steel piles (ϕ 600 mm) installed in the ground at the depth of G.L. - 40.0 m or - 45.0 m by diesel pile hammers. Figure 3 shows various structural types adopted in this building.

EARTH RETAINING AND EXCAVATION WORK PLANNING

Overall Planning

RC continuous diaphragm walls or RC continuous column walls with temporary RC struts have usually been adopted as earth retaining structures in Shanghai. There-

fore blasting methods are sometimes used to demolish the RC struts. Struts with steel pipes are sometimes adopted, but H-beam steels commonly used in Japan have not been used in Shanghai yet, and neither have pre-loading methods. For excavation works, temporary slopes are usually constructed instead of ride-in stages commonly used in Japan.

In Shanghai, there have been a few excavation projects with excavation depths of more than 15 m in which earth retaining wall failures and slope failures of the excavation surface have occurred. Thus, for excavation work with a depth of more than 7 m, a special technical committee consisting of about 10 specialists, including university professors and senior engineers of design institutes, was appointed to check for safety. In general, the committee offered some suggestions, such as that the length of earth retaining wall should be twice as long as the excavation depth, but they did not make mandatory requirements.

In this project, the inverted construction method was adopted. With this method, the frame construction of underground levels are carried out from the ground surface to the excavation bottom, following the progression of excavation. But in the frame construction of the high-rise building part, which was influential in the overall construction schedule, the frame construction of the core part had to be carried out first, because the steel beams lo-

cated along the outer side of the RC core had to be attached to the RC core afterward. Also, the construction of the 1st floor slab prior to the construction of the B1 floor slab was not to be recommended, because the 1F slab steel beams are more easily installed after the erection of steel columns of underground parts. For this reason, the B1 floor slab was adopted as the top slab for the underground structures and the openings in the floor slab were widened to facilitate the construction of the RC core. Therefore the B1 floor slab, which is a permanent structure, was used as the No. 1 strut. Figure 4 shows the plan view of the earth retaining structures, and Fig. 5 shows the perspective view of the underground structures during the construction of the B4 frame.

Excavation work prior to the No. 3 excavation stage was carried out using temporary slopes which enabled dump trucks to easily go down and up between the ground surface and the excavation bottom. At the final No. 4 excavation stage, the excavation work was carried out by dividing the excavation area into several sections horizontally and vertically, because the excavation depth was nearly 6 m. Moreover, the leveling concrete was placed immediately after finishing the excavation in each section in order to minimize the wall displacements.

Fig. 3. Structural configuration of the project

Fig. 5. Perspective view of the underground structures

Fig. 4. Plan view of the earth retaining structure and the location of sensors

Fig. 6. The cross section of the earth retaining structures

If the same excavation work was carried out in Japan under similar conditions, or if the steel strut method was more prevalent in Shanghai, a forward construction method with 4-stage steel struts would have been adopted, since the underground structure types were very complex.

Earth Retaining Wall and Drainage Planning

In this project, an RC continuous diaphragm wall was adopted. The wall was used only as a temporary earth retaining wall and not used as a permanent structure. The trenches for RC continuous diaphragm wall elements were excavated with clamshell bucket type machines.

The wall was designed with a thickness of 1 m and a length of 30 m. The width of one element was 6 m, and vertical structural joints between each element were not installed. Figure 6 shows the cross section of the earth retaining structure of the high-rise part. In this figure, a No. 4 temporary strut is shown. This No. 4 strut was initially planned to be installed in the design stage, but actually this temporary strut was eliminated based on data analyses from the simulation results.

This construction site was located in urban renewal areas, so there were many neighboring construction projects such as buildings and infrastructure construction. Usually, the clearance between earth retaining walls and outer frame walls of underground structures has to be determined by considering neighboring circumstances.

In Japan, a clearance of 75–100 mm is generally adopted in excavation work with RC continuous diaphragm walls. However, in this project, various uncertainties existed such as installation accuracy and assumed displacements, which made it inadequate to rationally design the clearance, so a conservative value of 200 mm was used to compensate for the unknown factors.

Bearing piles of the building foundation, plastic-board drains, vacuum deep wells and gravity-type deep wells were installed, before constructing the earth retaining wall. The plastic-board drains were installed to disperse the excess water pressure in the clay layers induced during installation of the bearing piles of the building. The

vacuum deep wells were installed to increase the strength of the clay layers. These two drain methods are commonly used in Shanghai to deal with problems of soft ground. Two gravity-type deep wells were installed into the foundation ground of the RC core part to prevent the foundation ground from heaving by artesian ground water and to drain inflow water from around the bearing piles.

Measurement Planning of the Earth Retaining

In this project, special attention was paid to the measurement and monitoring systems for the following reasons: (1) We, Japanese engineers, had not much experience in excavation works in Shanghai soft ground. (2) The RC continuous diaphragm wall was designed as a temporary structure, so the safety factor against the stress within the structures was not set up with high values. (3) It was desirable to eliminate the lowest temporary RC strut (No. 4 strut).

The hardware, such as measurement instruments, and software, such as measurement monitoring systems made in Japan (e.g., Onishi et al., 1991) were adopted, due to the reliability of sensors and the safety management framework. But, sensors made in China were also used to compare the difference of accuracy of sensors made in both countries. The installation of sensors, the measurement of wall displacements by inclinometers, and the compilation of the measurement results were conducted by a local company in Shanghai.

The measurement items and locations of sensors are shown in Fig. 4. The measurement system was designed to enable engineers in Japan to gather and transmit various data and also to control data loggers from Japan through public telephone lines.

DESIGN METHOD OF THE EARTH RETAINING WALL

Calculation Method for the Design

The stress and displacement of the wall and the reactions of the struts were calculated by the methods proposed by Onishi et al. (1985 and 1996). The section design was conducted to meet both Japanese and Chinese design criteria for allowable unit stresses for temporary loadings.

The calculation method is based on the beam on elastic subgrade theory, in which the wall is modeled as a beam and the ground is modeled as a set of discrete Winkler springs. The fundamental equation for beam bending is as follows:

$$\frac{d^2}{dx^2} \left(EI \frac{d^2 y}{dx^2} \right) + p(x) = w(x) \quad (1)$$

where, y : horizontal displacement of the wall

x : depth

EI : flexural rigidity of the wall

$p(x)$: subgrade reaction

$w(x)$: distributed load

In conventional retaining wall calculation methods, $w(x)$ and $p(x)$ in Eq. (1) model lateral distributed pressure loaded from the back wall and lateral distributed pressure loaded from the excavation-side respectively. But, using the conventional calculation methods, it is difficult to model the lateral pressure below the excavation surface. Also, it is very difficult to rationally model the continuity of calculation conditions for the succeeding excavation stage at each excavation stage.

The characteristic of this calculation method is that the lateral pressures can be given at both sides of the wall with different values respectively, and the pressures are variable according to the amount of the wall displacement. In the calculation, the lateral pressures are evaluated by the sum of the water pressure and the earth pressure. The basic method of evaluating earth pressures which are dependent on the amount of the displacement is shown below.

In this calculation method, as shown in Figs. 7(a), (b) and (c) respectively, three types of earth pressure distributions are defined, namely (a) earth pressure at rest before excavation, (b) earth pressure at rest after excavation with the condition that the wall is rigid, (c) earth pressure after wall deformation. The solution of Eq. (1) is obtained by defining $w(x)$ and $p(x)$ as follows;

$w(x)$: difference between pressure (a) and pressure (b) which models distributed load as shown in Fig. 7(d)

$p(x)$: difference between pressure (b) and pressure (c) which models subgrade reaction as shown in Fig. 7(e)

In Fig. 7(b), Regions I to III on the excavation side are defined as follows.

Region I: region in which the ground is excavated

Region II: region in which the pressure at rest decreases according to the decrease of over burden pressure

Region III: region in which the over burden pressure decreases, but the earth pressure at rest keeps the same value before excavation

Fig. 7. The variation of earth pressure distributions and basic concepts for load and subgrade reaction

Figure 8 shows the relationship between earth pressure (p) and displacement (y) with regard to Fig. 7 at some depth. Here, the maximum and minimum earth pressures are defined as the active and passive Rankine-Resal's earth pressure respectively. In this case, the following equations from (2) to (7) were obtained (here, the other symbols follow the definition of Fig. 7).

$$w = p_{fo} - p_o \quad (2)$$

$$p = \Delta p_f + \Delta p_b \quad (3)$$

$$\Delta p_f = p_f - p_{fo} \quad (4)$$

$$\Delta p_b = -(p_b - p_{bo}) \quad (5)$$

$$p_f = p_{fo} + k_h y \quad (\text{here, } p_{fa} \leq p_f \leq p_{fp}) \quad (6)$$

$$-p_b = -p_{bo} + k_h y \quad (\text{here, } -p_{bp} \leq -p_b \leq -p_{ba}) \quad (7)$$

where, k_h : coefficient of horizontal subgrade reaction

p_{fa}, p_{ba} : Rankine-Resal's active earth pressure (here, $0 \leq p_{fa} \leq p_o$ and $0 \leq p_{ba} \leq p_o$)

p_{fp}, p_{bp} : Rankine-Resal's passive earth pressure (here, $p_o \leq p_{fp}$ and $p_o \leq p_{bp}$)

Fig. 8. The relationship between earth pressure and displacement (The Region III)

Table 1. The material constants of the ground for the simulation analysis

Layer No.	Depth (m)	Soil	ρ_t	Excavation side			Backward side			K_0
				c	ϕ	k_h	c	ϕ	k_h	
1	0.0~1.5	Fill	1.88	9.0	10	1.0	18.0	20	2.0	0.50
2	1.5~2.5	Clay	1.88	9.0	10	1.5	18.0	20	3.0	
3	2.5~7.5	Silty clay	1.83	6.0	10	1.5	11.0	20	3.0	
4	7.5~17.5	Silty clay	1.71	7.0	5	1.5	14.0	10	3.0	
5	17.5~24.0	Fine silty clay	1.86	12.0	22	4.0	same to the left			
6	24.0~28.5	Silty clay	1.99	57.0	19	7.0	same to the left			
7-1	28.5~34.0	Fine silty sand	1.87	2.0	25	12.0	same to the left			

ρ : wet density (g/cm³), c : cohesion (kPa), ϕ : internal friction angle (deg.), k_h : coefficient of horizontal subgrade reaction (MN/m³), K_0 : coefficient of earth pressure at rest

In Fig. 7(c), by replacing pressure $p_f(x)$ and $p_b(x)$ with $p_o(x)$ for the next excavation stage, and by adding springs which model struts, the same calculation procedure is applied for subsequent excavation stages. Using this method, the results including actual excavation processes are continuously obtained.

Calculation Condition

As mentioned previously, the lateral pressure was evaluated as the sum of the water pressure and the earth pressure. The water pressure was evaluated from a triangular distribution of the pressure below G.L. -1.0 m with the water pressure coefficient being $K_w=0.8$. This evaluation was deduced from the water table in the clay layer and confined aquifer. The effective earth pressure at rest before excavation was evaluated from a K_0 consolidation test, and the averaged value of $K_0=0.5$ was adopted for each layer, respectively. The soil strength should normally be determined by effective stress if water pressure is considered. But, in this paper, cohesion c and internal friction angle ϕ obtained from direct shear tests were adopted as expected minimum values. The material constants used in the calculation are shown in Table 1. In Table 1, for the c , ϕ and k_h of the ground on the excavation side, the reduced values from those of the backward side ground were adopted. The reason for the reduction will be explained afterward, but in the design stage the reduced values were not adopted.

Regarding beam modeling, the retaining wall was treated as an elastic beam in both the design stage and construction stage calculations.

Calculation Results before Excavation Works

From the design stage calculation, a maximum displacement of 3–4 cm of the wall was obtained. Whereas, for the calculation using the values shown in Table 1 just before starting the excavation works, a maximum displacement of 7–8 cm was obtained. However, the section design was not modified since the stress of the wall was within the allowable level in both calculation results.

Prior to excavation, it was presumed that the elimination of the No. 4 temporary strut shown in Fig. 6 would

be almost impossible. But, since it was recognized that the elimination of the No. 4 strut would significantly reduce construction time and cost, efforts were made in the planning stage for the elimination with the aid of the measurement control method.

MEASUREMENT RESULTS OF EARTH RETAINING STRUCTURES

Regarding the measured data, Fig. 9 shows displacement distribution, Fig. 10 shows reinforcing bar stress and Fig. 11 shows the axial force of center slabs. In these three figures, the values are shown in each excavation stage and at selected points.

As shown in Fig. 9, the maximum displacement in the first stage was 11.7 cm at the top of the wall, and in the final excavation stage the maximum displacement was 12.9 cm. Maximum values obtained from other measurement points were almost the same.

The interesting point in the first excavation stage is that the displacement profile above G.L. -20 m shows a straight-line. This profile was also observed at excavation works in soft ground in Japan, but the tendency in this project was more marked than in Japan. In the final excavation stage, a displacement of 2.5 cm was obtained at the wall bottom even though the embeded depth of the wall into relatively hard ground with N -value of 20–30 was 6 m. This value of 2.5 cm was back-calculated from the displacement at the top of the wall, considering lateral displacements at the top of the wall obtained from surveying.

Regarding the stresses in the wall, the maximum value was obtained at a depth of G.L. -20 m in the first excavation stage, and this tendency corresponded well with the displacement profile. In the final excavation stage, a maximum tensile stress of 253 MPa was obtained at the depth of G.L. -15.3 m. Corresponding to the stress distribution, several cracks were observed in the section between the B3 floor slab and the excavation bottom in the final excavation stage.

Fig. 9. The measured displacement of the wall

Fig. 10. The measured stress of the reinforcing bars

Fig. 11. The variation of axial forces of the center slabs

SIMULATION ANALYSIS

Modeling of Flexural Rigidity

In the simulation analysis, the reduction of flexural rigidity of the wall was considered due to cracks in the concrete and yielding of reinforcing bars. As shown in

Fig. 12. The relationship between M and ϕ of the wall

Fig. 12, a tri-linear relationship between Moment (M) and curvature (ϕ) was modeled, and based on this relationship, the cracking moment M_{cr} , yield moments M_y and M'_y , and their corresponding curvatures ϕ_{cr} , ϕ_y and ϕ'_y , were evaluated, respectively. In this M - ϕ relationship, hysteresis was not considered. Table 2 shows fundamental data of the wall for the simulation. Here, the compressive strength of concrete ($F_c=30$ MPa) and the yield strength of the reinforcing bar ($f_y=320$ MPa) were determined based on design values.

Effects of Pre-excavation Construction on Wall Behavior

From the displacement and stress distributions of the wall in the first excavation stage shown in Figs. 9 and 10, it was considered that the resistance in the excavation ground below the excavation surface down to the depth of the final excavation bottom was very small. Also, the distributed load was active to a relatively deep part of the wall compared to the excavation depth. In the simulation analysis, several tests with different material constants were conducted, but it was very difficult to simulate the measured data since the same material constants were used for both sides of the wall. As a result, it was concluded that pre-excavation works in the excavation ground had significant influence on the behavior of the earth retaining wall. These pre-excavation works are as follows: (1) the installation of plastic-board drains to decrease the excess water pressure in the clay layer due to the installation of foundation steel piles, and (2) the installation of the foundation piles conducted with the aid of follower piles. For the latter case, it was observed that when the follower piles were withdrawn, the ground around the pile holes collapsed and the collapsed soil clogged the holes. As a result, the back filling of the hole was quite difficult. Therefore, vertical voids existed in the excavation ground from the ground surface to the excavation bottom before excavation.

It is well known that these vertical voids cause excessive displacement of the wall, but reasonable calculation methods to simulate the excessive displacement have never been proposed. In the simulation analysis, the material constants of the ground on the excavation side in the No. 1 layer to No. 4 layer were reduced by 50% of the

Table 2. The fundamental data for the wall

Depth (m)	Thickness (cm)	Tension reinforcement ratio (%)		M_{cr}	ϕ_{cr}	M_y	ϕ_y	M'_y	ϕ'_y
		Excavation side	Backward side						
0.0~6.0	100	0.410	0.210	520	2.42	1090	22.0	567	20.3
6.0~15.0		0.820				2120	24.5	565	20.1
15.0~22.0			0.620			2130	23.9	1620	22.6
22.0~30.0		0.210				566	20.2	1620	23.3

Bending moment: (kN·m/m), Curvature: ($\times 10^{-6} \text{ cm}^{-1}$)

original ground as shown in Table 1.

Comparison between Measured and Simulated Results

The measured displacements and curvatures of the wall in the first excavation stage and final excavation stage are compared with the simulated data, as shown in Figs. 13 and 14, respectively. The measured data of curvatures are calculated from the strain of reinforcing bar stress transducers and the distance of main reinforcements.

In order to adjust calculation results to be similar to the measured data, the pressure in excavation (the difference of the earth pressure at rest between both sides of the wall) had to be extended to a relatively deep part of the wall. Moreover, it was very difficult to simulate the measured displacement profile (in which the displacement of the tip of the wall was relatively large) of the latter excavation stages. This implies that the calculation method based on beam on elastic subgrade theory may not be suitable for excavation analysis in dealing with large scale excavation which includes large scale movement of ground around the earth retaining wall.

From Figs. 13 and 14, it is concluded that the M - ϕ relationship of the wall was reasonably modeled. In both figures, the curvatures ϕ_{cr} , ϕ_y and ϕ'_y in Table 2 are also shown. The measured maximum curvatures are $19.2 \times 10^{-6} \text{ cm}^{-1}$ at G.L. -16.3 m in the final excavation stage, and the value corresponds to 1780 kN·m/m in terms of moment. The ratios of these measured values against ϕ_y and M_y are 80% and 83% respectively at the section. The measured value from reinforcing bar stress transducers at the same point showed 253 MPa, and this indicates 79% of $f_y = 320 \text{ MPa}$.

RESULTS OF UNDERGROUND EXCAVATION WORKS

From the measured data, a wall displacement of more than 10 cm was observed from the first excavation stage, but this was actually expected in advance by considering the effect of the installation of plastic-board drains and foundation steel piles executed prior to the excavation. In spite of this large displacement at the beginning of the excavation, the excavation work was continued. This is because the average ground shear strain of the backward

Fig. 13. The displacement and curvature in the 1st excavation stage (01/11/1995)

Fig. 14. The displacement and curvature in the final excavation stage (01/02/1996)

side of the wall evaluated from the displacement mode of the first excavation stage was approximately 0.6% (= 12 cm/20 m), and this value was less than the failure strain of the ground. A value of 15% was used for the shear failure strain of the ground in this project, since most of the values of axial failure strain obtained from triaxial compression tests were greater than 15%.

Regarding the measurement system, the following observations were made. Measurements were smoothly conducted aside from some problems caused by the incorrect installation of some sensors and the breakdown of the inclinometers (made in both countries). Regarding the reinforcing bar stress transducers made in China, there were more problems than for sensors made in Japan due to the lack of temperature correcting circuits and the problem with water tightness.

In this project, measurement control was the key management point for smooth and economic execution of the excavation works as expected in the planning stage. As a result, the temporary No. 4 strut was successfully eliminated after reviewing the results of the simulation analysis, which successfully predicted the behavior of future excavation stages using the measured data.

CONCLUSIONS

At a deep excavation site for a high-rise building project in Shanghai, China, the simulation methods and measurement systems for earth retaining structures, which the authors had previously developed in Japan, were applied, and the inverted construction method for underground works, which had never before been used in China, was also applied.

The main results are as follows:

(1) Starting from the first stage of excavation to a depth of G.L. - 4.3 m, noticeable displacement of the earth retaining wall was measured. The maximum lateral displacement was 11.7 cm at the top of the earth retaining wall in the first excavation stage, and reached a value of 12.9 cm at a depth of G.L. - 15 m in the final excavation stage. The maximum tensile stress measured by stress transducers installed on the main reinforcement bars of the earth retaining wall was 253 MPa, which corresponds to 79% of the yield stress of the bar.

(2) The large displacement of the earth retaining wall, especially during the first excavation stage, was induced by the vertical voids in the ground on the excavation side. These were created prior to the excavation during installation of the plastic-board drains and removal of the followers used for driving the foundation steel pipe piles.

(3) In the simulation analysis, the stiffness of the ground on the excavation side was reduced by 50% of the original, due to the presence of the vertical voids mentioned above. The stiffness of the earth retaining wall was also reduced due to cracks in the concrete of the earth retaining wall. With these modifications, the results of the simulation analysis showed values quite similar to the measured data.

(4) The measuring systems worked smoothly except for minor troubles in the installation of the sensors and occasional failures of the inclinometers.

(5) During the excavation, real time simulations were conducted by using the most current measurement results to predict the behavior in following excavation stages. By using the results of these simulations, the originally planned No. 4 temporary struts were eliminated, and the excavation was successfully completed to the designed depth.

As a result, it is concluded that the simulation methods, measurement systems and execution methods previously applied to underground excavation works in Japan are applicable to excavation works in Shanghai soft ground.

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