Geotechnical Aspects of Underground Construction in Soft Ground – Ng, Huang & Liu (eds)

Consideration of design method for braced excavation based on monitoring results

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ABSTRACT: A comparison between observed data and design value of earth retaining wall deflection due to braced excavation was carried out in soft and sensitive clay ground of some construction sites of Osaka Subway Line No.8. The beam-spring model was employed in the braced design method, and it was taken into account the characteristics of the Osaka soft ground, and there was good agreement between the observed data and design value in past results. According to this comparison, the observed wall deflection was larger than the designed one in some construction sites consisted of the soft and sensitive clay layer with 10 to 20 m thickness. In this paper, the measuring process of the horizontal coefficient of subgrade reaction k_h in the excavation side of soft clay layer is discussed. As the value of k_h became small depended on the wall deflection, the new knowledge has been employed on the design method. It is found that the calculation with the revised design method agree well with the monitoring data.

1 INTRODUCTION

In densely populated city, it is necessary to use the underground space highly and effectively for the development of city. It is believed that the demand for much deeper underground excavation will increase gradually. Therefore an applicable design method is demanded for deep, safe and economical excavation. Osaka Municipal Transportation Bureau (OMTB) suggested an original design method for braced excavation based on the characteristics of the Osaka ground and subway constructions. At each construction site (elevens stations and railway depot) where open cut method was adopted in Osaka Subway Line No.8, braced excavation design based on this original design guideline was carried out, and the observational method was also utilized effectively.

In this paper, some comparisons between observed data and design value of earth retaining wall deflection due to braced excavation have been carried out on soft and sensitive clay ground of two construction sites of Osaka Subway Line No.8. The evaluation method for design has been described based on the ground properties.

2 CHARACTERISTICS OF THE MODIFIED BEAM SPRING MODEL

East and West sides of Osaka ground are consistent with the Holocene layers (soft clay and loose sand), but Pleistocene layers exist around the ground surface of Uemachi plateau. The water levels are high in unconfined and confined aquifers, also the permeability of these aquifers are large.

The beam-spring model for the braced excavation, which is indicated in "Standard Specifications for Tunnel [Open Cut Tunnel]" published by the Japan Society of Civil Engineers (JSCE, 2006), is frequently implemented as a widely usable method in Japan. However, since the result of the prediction of wall deflection and strut force are not always consistent with the observation data, OMTB proposed the modified beam spring design method (OMTB, 1993) (Kishio *et al.*, 1997) which can consider "the characteristics of Osaka ground" and "the conditions of subway construction". The outline of the OMTB model is shown in Figure 1.

2.1 *Active lateral pressure above the excavation bottom*

Because there are some possibilities of gap between the braced wall and back ground, the active earth pressure is estimated by Rankine-Resal's equation with the water pressure. In sandy layer, the water pressure is assumed as hydrostatic pressure. In clayey layer, the water head of the upper sandy layer is extended in this clayey layer.

2.2 *Active lateral pressure below the excavation bottom*

In the case of sufficient penetration depth of braced wall, the wall deflection near the tip is small and the lateral pressure is kept as the initial condition. So, if the active lateral pressure is defined basically only by the limit equilibrium theory, there are some cases which the wall deflection is overestimated by giving much lateral pressure. Therefore, the active lateral pressure which is deeper than the bottom layer of excavation is gradually decreasing in the area of triangle formed from the lateral pressure at the bottom of the excavation to the tip of wall.

2.3 *Resisting lateral pressure of the excavated ground*

Resisting lateral pressure of the excavated ground is the multiplication of the coefficient of horizontal

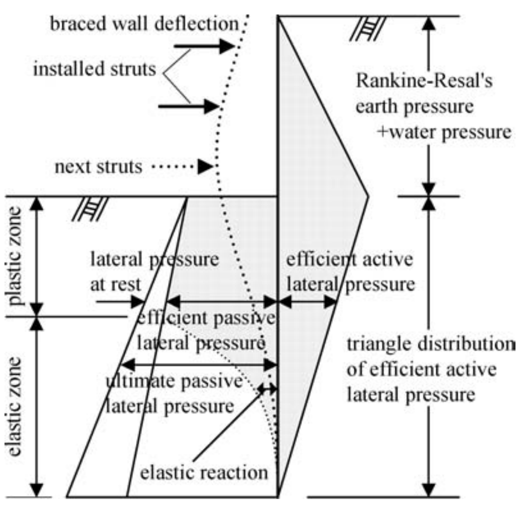

Figure 1. The concept of the modified beam spring model (presented by OMTB, 1993).

subgrade reaction and the wall deflection. However, this value should not exceed the coefficient of the passive lateral pressure which is the subtraction from limit passive lateral pressure defined by Coulomb's equation to the lateral pressure at rest.

2.4 *Water pressure in clayey ground*

Since it is difficult to distinguish the water pressure from the lateral pressure in clayey ground, lateral pressure is often identified as the integration of soil and water. On the other hand, it is considered that soil is separated from water in modified beam-spring model. Because the pore water pressure acts on the braced wall hydrostatically, the water path is possibly formed between the wall and the back ground due to braced excavation.

For these reasons, the effective stress method is adopted in both sandy layer and clayey layer. The groundwater table in clayey ground is taken on the higher gravitational pressure distribution by comparison between the upper water-bearing layer and down side water-bearing layer.

2.5 *Supported effect of covering plates*

Because the effect of depressing the wall deflection is recognized when the covering plates are constructed in the same direction as struts, the supported effect of covering plates is considered by reducing 10% of the spring-beam coefficient.

2.6 *Horizontal coefficient of subgrade reaction of excavated side*

The coefficient of subgrade reaction k_h used in the JSCE model is taken into consideration the geometrical effect related to the difference of loaded width based on some plate loading test results performed near the ground surface. However, the lateral pressure acts on the horizontal direction against the earth retaining wall, because the wall is installed to the vertical direction in subway construction site. Therefore, it is not always appropriate to apply the coefficient of subgrade reaction used in the JSCE model to braced excavation design directly. So in the OMTB model, the coefficient of subgrade reaction is expressed as equation (1) and (2) (Yanagida *et al.*, 1981) empirically.

sandy layer: $k_h\,(\mathrm{MN/m^3}) = 10/16 \times N$ (1)
(N : standard penetration test N-value)

clayey layer: $k_h\,(\mathrm{MN/m^3}) = \alpha \times q_u$ ($\alpha = 1/20$) (2)
($q_u\,(\mathrm{kN/m^2})$: unconfined compressive strength)

3 COMPARISONS BETWEEN OBSERVATION AND DESIGN OF BRACED WALL DEFLECTION

The comparison between observed data and design value of earth retaining wall deflection due to braced excavation was carried out in soft and sensitive clay ground of No.8 Line construction sites.

In general, the design value can estimate the observed data appropriately in most construction sites. But in some sensitive and soft alluvial clay layer accumulate from 10 m to 20 m thick, observed data are larger than the design value due to braced excavation. The causes for these phenomena are described as follows.

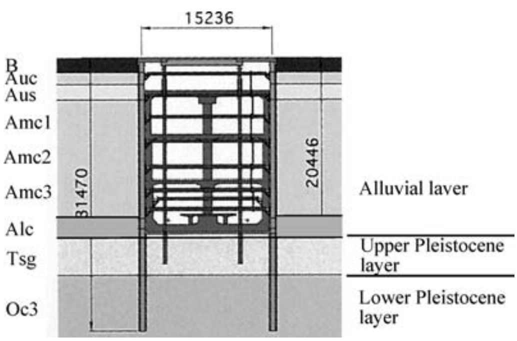

Figure 2. Cross section (A site).

3.1 *A-site*

The layer of this A-site ground consists of the alluvial layer, upper lower Pleistocene from the ground surface.

The uniformity coefficient of this fine sand Aus is small, and is called the first water-bearing layer. The alluvial clayey layer Auc is sensitive (*N-value* = 0 to 3, liquid limit $I_L = 0.4$ to 1.0, cohesion $c = 20$ to 100 kN/m^2), and is typical soft ground in this construction site. On the other hand, under the alluvial layer, the upper Pleistocene sandy and gravel layer Tsg (the second water-bearing layer), the lower Pleistocene clay layer Oc3 (Osaka Group, Marine Clay Ma3, $c =$ about 400 kN/m^2) and the lower Pleistocene sandy layer Os3 (Osaka Group, the third water-bearing layer, *N-value* > 60) are deposited continuously to horizontal direction.

The cross section of A-site is shown in Figure 2, the wall and struts conditions are shown in Table 1 and the soil parameters are shown in Table 2. In this construction site, the seepage control method was adopted by extending the earth retaining wall to the low permeable layer Oc3, excavation width is 16.2 m, the final excavation depth is GL-21.5 m and the penetration depth is 4.8 m.

Figure 3 shows the comparison between observed data and design value of the earth retaining wall deflection. The observed wall deflections in east and west sides are symmetric till the 4th step. It is confirmed that the design value can estimates the actual phenomenon adequately. However, since the 5th step, the observed wall deflection of west side was larger than the east side, which can be seen from the results of the 8th step

Table 1. Earth retaining wall and each strut (A site).

Soil mixing wall (H-steel) condition				
Size (mm)	Pitch S(m)	Length L(m)	EI (kN·m^2/m)	Area A(m^2)
H-588 × 300 × 12 × 20	0.60	27.25	399000	0.01925

Excavation condition		Strut condition					
Depth		Depth					
Step	(GL-m)	Stage	(GL-m)	Size (mm)	Span L(m)	Pitch S(m)	Area A(m^2)
0th	1.51	Cover beam	0.51	H-588 × 300 × 12 × 20	16.25	2.00	0.01925
1st	4.50	1st	3.50	H-300 × 300 × 10 × 15	14.76	2.50	0.01048
2nd	7.00	2nd	6.00	H-300 × 300 × 10 × 15	14.86	2.50	0.01048
3rd	8.70	3rd	7.70	H-300 × 300 × 10 × 15	14.66	2.50	0.01048
4th	11.20	4th	10.20	H-300 × 300 × 10 × 15	14.76	2.50	0.01048
5th	13.90	5th	12.90	H-350 × 350 × 12 × 19	14.46	2.50	0.01549
6th	16.45	6th	15.45	H-350 × 350 × 12 × 19	14.46	2.50	0.01549
7th	19.05	7th	18.05	H-350 × 350 × 12 × 19	14.46	2.50	0.01549
8th	21.55	–	–	–	–	–	–

in Figure 3. It can be assumed that the cause of this phenomenon was the difference of construction process and developmental pattern of creep deformation. Moreover, the observed data exceeded the design value at the west side. It was considered that this disparity occurred for the reason that the plastic zone under the excavation bottom expanded in the Amc and Tsg layer from the 5th step drastically. In addition, it was confirmed that the stress in the wall was controlled within the allowable stress.

In the excavation stage at the Amc layer, the calculation result considering the 75% stress reduction under the 5 m from the excavation bottom was shown together in Figure 3. During an excavation in cohesive soil, if an excavation stage takes a long time, the suction of the subgrade soil will disappear due to swelling caused by the water infiltration from continuous rainfall, which leads to reduction in strength (Hashimoto *et al.,* 1997). In conjunction with this arrangement, the ultimate passive lateral pressure and coefficient of subgrade reaction were reduced. This phenomenon was verified by the consolidation with un-drained triaxial compression test, in which shear strength reduced to 70% after the suction was disappear completely and after measuring the water pressure and suction. In short, it is proved that there is a possibility that this phenomenon may occur (Kato *et al.,* 2006).

Table 2. Soil parameters (A site).

Soil layer	Bottom depth (GL-m)	*N-value*	Cohision c (kN/m^2)	Internal friction angle ϕ (°)	E_{50} (MN/m^2)
B	1.8	2	0	19.3	–
Auc	4.9	4	42	0	4.1
Aus	8.3	2	0	19.3	–
Amc1	13.8	0	29	0	4.7
Amc2	16.8	1	60	0	6.9
Amc3	19.4	4	91	0	15.9
Alc	21.8	6	108	0	15.9
Tsg	25.4	26	0	32.7	–
Oc3	31.6	14	360	0	83.6

In the 8th step, the calculation result considering the stress reduction exceeded the design value which could explain the observed data appropriately to some extent. However, under the bottom of the excavation, especially in Tsg layer, the tendency that the design value and calculation result exceeded the observed data. The wall deflection distribution was different between the observed data and design value and calculation result. It would appear that one of the reasons for these tendencies is the deformation at the bottom of the wall towards the excavation side.

3.2 *B-site*

The layer of this B-site ground constitutes the alluvial layer, upper lower Pleistocene from the

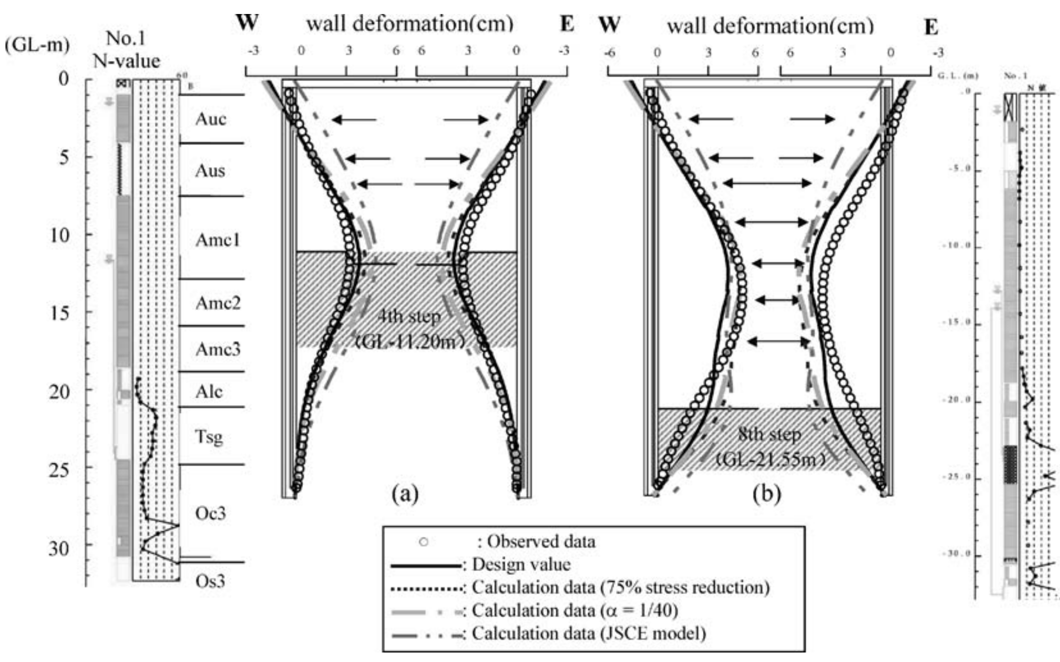

Figure 3. Comparison between observation and design value of braced wall deflection (B site, (a) : the 4th step, (b) : the 8th step).

ground surface. Especially, this construction site was located in the Neyagawa lowland, and it is peculiar that the very soft and sensitive alluvial clay layer (*N-value* $\cong 0$, liquid limit $I_L = 0.6$ to 1.5, cohesion $c = 30$ to 100 kN/m^2), which is specific for the East side of Osaka Plain, deposited with 15 to 20 m thickness. The upper Pleistocene sandy and gravel layer Ts & Tg and the lower Pleistocene sandy layer Os3 (Osaka Group) constitute the second water-bearing layer under the alluvial layer.

The cross section of B-site is shown in Figure 4, the wall and struts conditions are shown in Table 3 and the soil parameters are shown in Table 4.

In this construction site, the seepage control method was adopted by extending the earth retaining wall to the low permeable layer Oc7 (about GL-42 m), too. The bottom depth of Soil Mixing Wall (H-steel) was extended to the Os8. The excavation width is 17.2 m, the final excavation depth is about GL-22 m and the penetration depth is about 5 m.

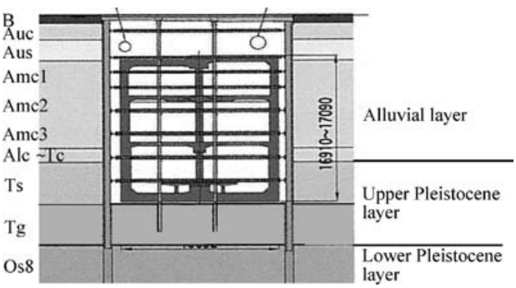

Figure 4. Cross section (B site).

Figure 5 shows the comparison between the observed data and design value of the earth retaining wall deflection in the 4th and 8th excavation steps (Oota *et al.*, 2007). The wall deflection distribution mode was similar both side in the 4th excavation step. However the amount of the observed wall deflection was two times of the design value. Moreover, in the 8th excavation step, the wall deflection distribution mode was different in both and observed data exceed the design value. In addition, it was confirmed that the stress in the wall was controlled under the allowable stress.

Table 4. Soil parameters (B site).

Soil layer	Bottom depth (GL-m)	*N-value*	Cohision c (kN/m^2)	Internal friction angle ϕ (°)	E_{50} (MN/m^2)
B	0.8	2	0	19.9	–
Auc	2.0	0	27	0	2.2
Aus	4.0	2	0	19.9	–
Amc1	8.0	0	42	0	2.2
Amc2	13.0	0	63	0	5.5
Amc3	16.0	0	76	0	7.4
Alc	19.0	3	73	0	5.6
Tc	20.8	7	129	0	–
Ts	23.3	42	0	37.5	–
Tg	26.0	45	0	38.2	–
Os8	39.1	60	0	41.8	–

Table 3. Earth retaining wall and each strut (B site).

Soil mixing wall (H-steel) condition

Size (mm)	Pitch S(m)	Length L(m)	EI (kN·m^2/m)	Area A(m^2)
H-588 × 300 × 12 × 20	0.60	27.52	399000	0.01925

Excavation condition		Strut condition					
Depth		Depth					
Step	(GL-m)	Stage	(GL-m)	Size (mm)	Span L(m)	Pitch S(m)	Area A(m^2)
0th	1.42	Cover beam	0.42	H-488 × 300 × 11 × 18	17.15	2.00	0.01592
1st	2.81	1st	1.81	H-300 × 300 × 10 × 15	16.55	2.59	0.01048
2nd	5.96	2nd	4.96	H-350 × 350 × 12 × 19	16.45	2.59	0.01549
3rd	8.26	3rd	7.26	H-350 × 350 × 12 × 19	16.45	2.59	0.01549
4th	11.51	4th	10.51	H-350 × 350 × 12 × 19	16.45	2.59	0.01549
5th	14.51	5th	13.51	H-350 × 350 × 12 × 19	16.45	2.59	0.01549
6th	17.21	6th	16.21	H-400 × 400 × 13 × 21	16.35	2.59	0.01977
7th	19.61	7th	18.61	H-400 × 400 × 13 × 21	16.35	2.59	0.01977
8th	21.70	–	–	–	–	–	–

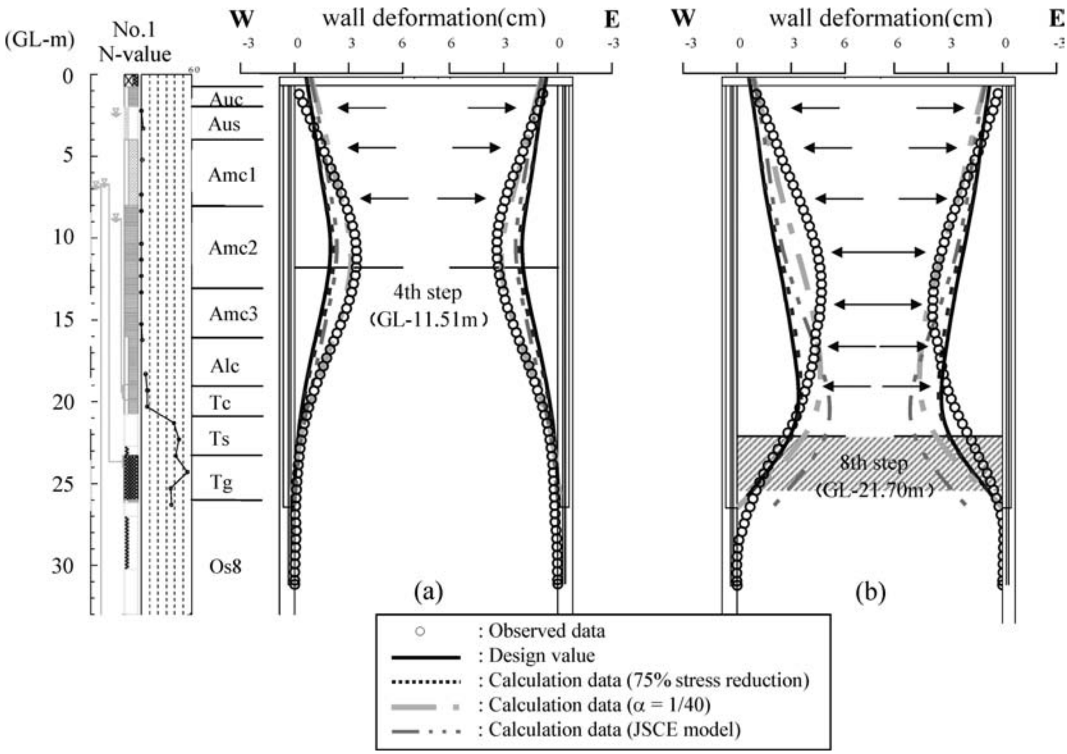

Figure 5. Comparison between observation and design value of braced wall deflection (B site, (a) : 4th step, (b) : 8th step).

As the ground condition under the bottom of the excavation is considered as the plastic zone in the calculation using JSCE model, the bottom of the wall deformed towards the excavation side in a larger value and the wall deflection distribution had different phenomenon compared to the observation. In the excavation stage at the Amc layer, the calculation considering the 75% stress reduction under the 5 m from the excavation bottom was shown together in Figure 5. Unlike the comparison result in A-site, this calculation was similar to the design because wall deflections around the bottom of the excavation are in the plastic zone. It was impossible to explain the observed phenomenon adequately used by some calculation model.

The horizontal coefficient of subgrade reaction k_h of clayey ground for excavation side in the OMTB model is determined by equation (2). This setting method of k_h was the empirical equation based on many observed data in the case that the wall deflection was about 3 cm (Yanagida *et al.*, 1981). This reference bring up the problem that k_h is tend to decrease due to the increase of the wall deflection.

In the actual construction site, as k_h is depended on the ground mechanical characteristics and some boundary conditions and so on, it is known that k_h changes every second due to braced excavation. For example, k_h is determined as solid line by the wall deflection function taking into consideration the nonlinearity (Japan Road Association, 1986).

The inverse analysis based on some earth retaining monitoring results was carried out to estimate the value of k_h. Modified Pawell Method was employed for the inverse analysis. It is possible to obtain the optimized solution stably on the many unknown parameter problem (Kishio, *et al.*, 1995). Input values for the inverse analysis are earth retaining wall deflection (angle of inclination) and axial force of struts, and output values are lateral pressure on the earth retaining wall and k_h.

Figure 6 shows the inverse analysis results based on the monitoring data in Osaka Subway Line No.8 touched to the Kishio, *et al.*, 1997. The vertical scale is the ratio of the estimation value k_h by the inverse analysis to design value of k_{h0}. In other words, $k_h/k_{h0} = 1$ means the inverse analysis results and design value are the same.

In the case that the wall deflection was about 1 cm, the relation between k_h and k_{h0} was about the same in both past actual results and Line No.8 results. In short, the applicability of k_h in the design is reasonable.

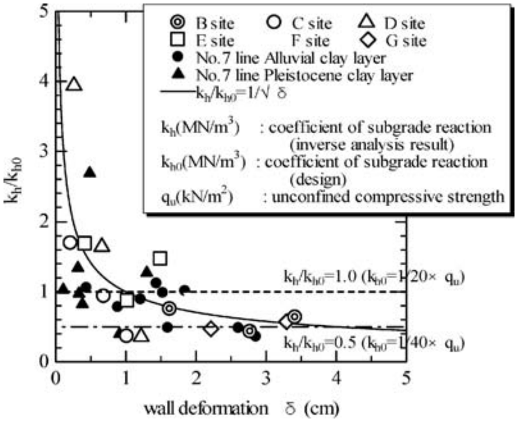

Figure 6. Dependence for brace wall deformation of k_h (touch in Kishio, *et al.*, 1997).

However in the case that the wall deflection was about 2 to 4 cm, inverse analysis results k_h are smaller than the design value k_{h0}, the ration k_h/k_{h0} decreased to about 0.5.

The relation of $\alpha = 1/40$ was presumed on the assumption that k_h decrease due to the increase of the wall deflection. Figure 5 shows the recalculation results under this relation.

In the case of the 4th excavation step, the relation between observation and recalculation was in good agreement. In the case of the 8th excavation step, the wall deflection distribution mode between observation and recalculation was still different, but the maximum amount of wall deflection was similar. It is believed that the cause of differences in the wall deflection distribution mode is the limit explained by the beam spring model in design.

In accordance with these estimations results, it is preferable to consider the k_h setting method carefully as equation (3) with considering the traditional design idea in the case of earth retaining design in the soft and sensitive clayey layer, which *N-value* is about 0 to 2, with thick layer (about 10 to 20 m).

$$k_h\,(\mathrm{MN/m^3}) = \alpha \times q_u\ (\alpha = 1/20 \text{ to } 1/40) \qquad (3)$$

4 CONCLUSIONS

The results are shown as follows;

1. At the A-site, the observed wall deflection in the east and west side are symmetric till the 4th step, and it is confirmed that the design value estimates the actual phenomenon adequately. However since the 5th step, the observed data exceeded the design value. It was assumed that the plastic zone expanded drastically to the penetration area in design.
2. It was possible that the calculation result considering the 75% stress reduction under the 5 m from the bottom of the excavation explained the observed data to some extent. However, under the bottom of the excavation, the tendency that the design value and calculation result exceeded the observed data. It was considered that one of the reasons for these tendencies is the deformation at the bottom of wall towards the excavation side.
3. At the B-site, the wall deflection distribution was similar between the observation and design in the 4th excavation step. However the observed wall deflection is two times of the design. In the 8th step, the wall deflection distribution mode was different in both, and observed wall deflection exceeded the design value.
4. Owing that $\alpha = 1/40$ was presumed on the assumption that k_h decrease due to the increase of the wall deflection, the relation between observation and recalculation was in good agreement in the case of the 4th excavation step. In the 8th excavation step, the wall deflection distribution mode in both was still different, but the maximum wall deflection was close.
5. It is recommended that the k_h setting method carefully as equation (3) with considering the traditional design idea in the case of earth retaining design in the soft and sensitive clayey layer, which *N-value* is about 0 to 2, with thick layer (about 10 to 20 m).

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