Geotechnical Aspects of Underground Construction in Soft Ground, Kastner, Emeriault, Dias, Guilloux (eds)
 ISBN 2-9510416-3-2

Behavior of Earth Retaining Structures with Wall-type Soil Improvement

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ABSTRACT: In relatively close proximity in Tokyo, two types of soil improvement method were undertaken where excavation depths and ground conditions were nearly equal but the floor area was different to reduce the effects of excavation on adjacent structure. This paper describes the detail behavior and the evaluation method which using a beam-spring analytical model of earth retaining wall with soil improvement. We found that both types of soil improvement methods were highly effective in reducing the displacement and this analytical method fairly evaluated actual behavior of earth retaining wall with soil improvement.

1 INTRODUCTION

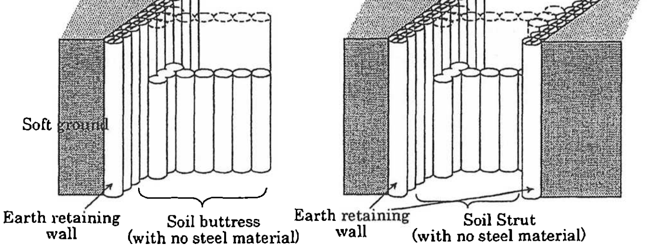
In recent years as cities have become more compact, there has been an increasing amount of excavation work that has come close to adjacent structures. To reduce the effects of this excavation on adjacent structures, it is becoming increasingly important to use auxiliary methods that will reduce the displacement of earth retaining walls. To reduce such displacement in soft ground, various methods have been used, including reinforcement of wall rigidity, increasing the number of levels of supporting struts, and soil improvement under the excavated bottom (Chang et al., 1996, Uchiyama et al., 2000). Soil improvement work is roughly divided into methods such as quicklime piles method that are designed to strengthen the ground, and methods such as cement stabilization that create a hard ground. Cement stabilization can be further divided into improvement of the entire excavated bottom, and partial improvement such as wall-type improvement, column-type improvement, etc. While the displacement reduction effect of the method that improves the entire excavated bottom is relatively well documented, there is currently no method for satisfactorily evaluating the effectiveness and designing the configuration of partial improvement.

Two types of soil improvement method were undertaken in relatively close soft grounds in Tokyo for construction projects where excavation depth and ground conditions were nearly equal but the floor area was significantly different (Shimizu et al., 1997, Tanida et al., 2000). At both projects, the excavation work was done next to adjacent structures, so soil improvement was employed to reduce displacement, but the improvement method depended on the planar scale of the excavation. This paper describes the detailed measurements taken of the earth retaining walls and the improved soil and examines the displacement reduction effect of the soil improvement method. In addition, beam-spring analysis model is used to evaluate the displacement of earth retaining walls with soil improvement.

2 OVERVIEW OF THE EXCAVATION WORKS AND GROUNDS

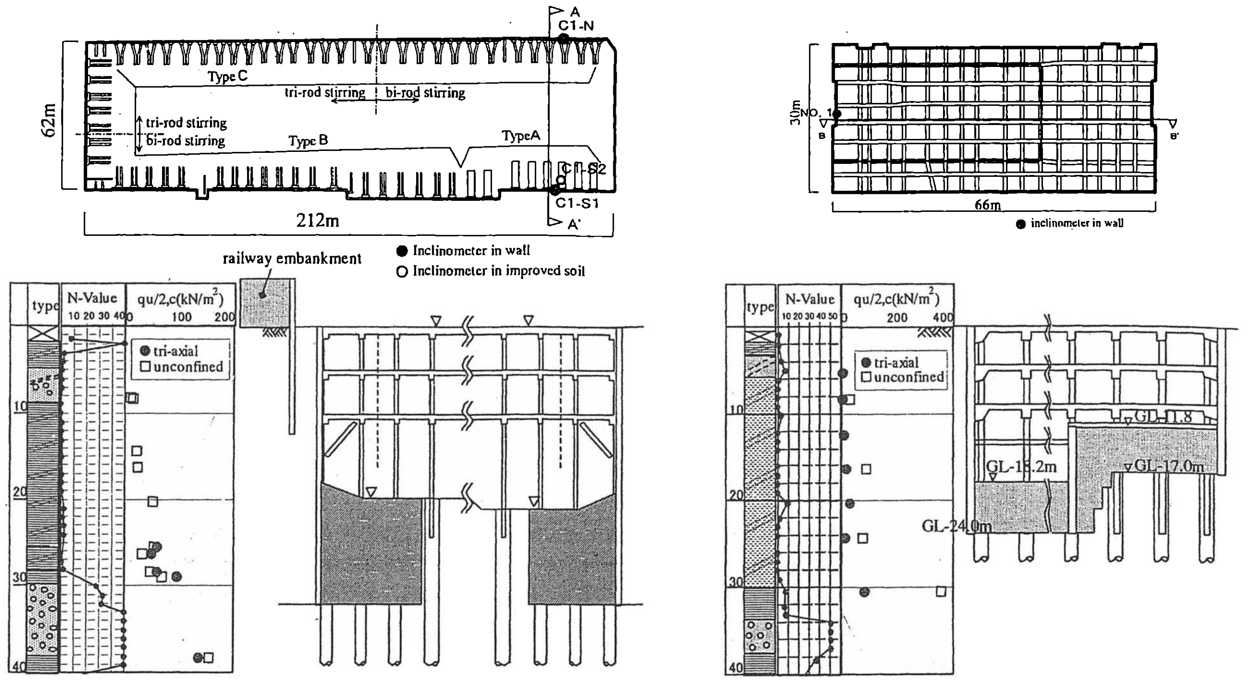
Schematics of earth retaining wall displacement reduction methods using wall-type soil improvement are shown in Fig. 1. Ground is improved by either constructing soil improvement walls roughly perpendicular to walls at appropriate intervals and at a certain width from the wall (soil buttress method, Fig. 1a), or by making continuous soil improvement walls between earth retaining walls that face one another (soil strut method, Fig. 1b).

Fig. 2 shows plan and section of excavation and the soil profile of the two projects. The excavation in Project A covered an area of 212m×62m, at a depth of GL-19.7m to 21.0m, and the soil buttress method was used here. In contrast, the excavation in Project



(a) Soil buttress method (b) Soil strut method

Figure 1. Overview of the soil improvement



(a) Project A (b) Project B

Figure 2. Overview of the excavation works and soil profile

B covered an area of 66m×30m,at a depth of GL-18.2m (GL-11.8m in some places), and the soil strut method was used. A railway embankment was located about 5m away from the earth retaining wall in Project A, while in Project B there were office buildings and housings next to the earth retaining wall, so the wall displacement caused by excavation had to be strictly controlled. Excavation was done using a total of four levels of supporting struts, including the first aboveground floor and two basement floors, as well as final support of diagonal steel struts (Project A) and a temporary RC slab (Project B). The top down method was used to build these supports.

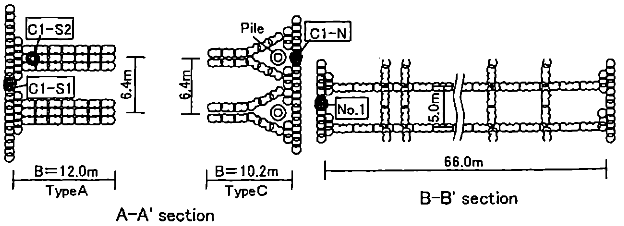
Both project locations were between the mouths of the Sumida and Arakawa rivers. The sedimentary surface of the delta and/or shallow sea floor heaved, forming this alluvial coastal plain. The soft clay layers extend from the ground surface to depth of about 30m.

The measurement items for excavation were as follows: In Project A, the lateral displacement of the earth retaining wall and the improved soil, and the stress of the reinforcing beams of the wall, and the vertical displacement of the ground. In Project B, the lateral displacement of the earth retaining wall. In both projects, measurements were taken from the completion of the earth retaining walls construction to the completion of the excavation work.

3 OVERVIEW OF THE WALL-TYPE SOIL IMPROVEMENT

3.1 Summary of the improved soil construction

Details of the improved soils in both the projects are given in Fig. 3. The improved soil walls were roughly perpendicular to the earth retaining walls and improved soil columns were built to maintain a constant lap width. In Project A, the allowable displacement was determined based on the adjacent structure. This in turn was used to establish three different types of buttresses by modifying the width B of the ground improvements, the number of walls and configurations. It should be noted that the buttresses for Type C were constructed at an angle due to their relationship with the pile position. The improvement depth was down to GL-31 to 32m, and the improved soil walls were into a hard ground of about 2m in thickness. In one row there were two or three improved soil columns of ϕ1000 mm and pitch of 800 mm that were constructed by deep cement mixing method using 2-shaft or 3-shaft digging wings. In Project B, continuous beam-like improved soil walls were constructed between the parallel earth retaining walls. The improved depth was designed to match the depth of the excavation, which was GL-17m to GL-24m. The improved soil walls were not into hard ground. In one row there were three soil cement columns of ϕ900 mm and



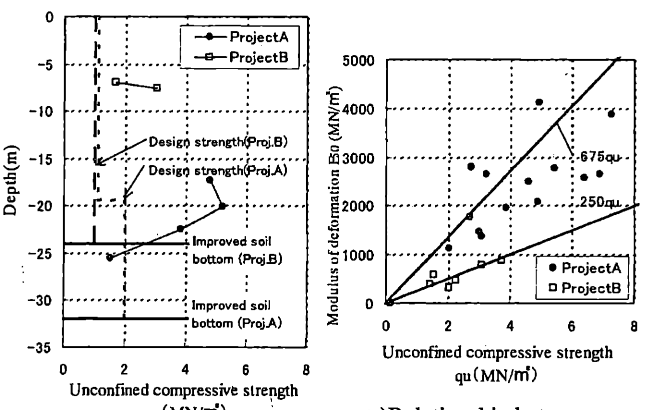
(a) Project A (b) Project B

Figure 3. Details of the improved soil

pitch of 600 mm that were constructed with the 3-shaft cement column wall method. In both methods, connection soil cement column were constructed to join the earth retaining walls and the improved soil walls.

3.2 Improved soil strength

In Project A, the cement slurry was set at W/C=100%, and the design strength of improved soil was 2.0MPa (or 1.0MPa at depths above the final excavation bottom). In Project B, however, the cement slurry was set at W/C=240%, and the design strength was 1.0MPa. The external force exerted on the earth retaining walls in the soil buttress method is conveyed hard ground through the improved soil by mainly shear stress, while in the soil strut method, it is conveyed to the parallel earth retaining walls mainly by compressive force. Therefore, the design strength below the final excavation bottom was increased in the soil buttress method, in which high shear stress was at work. After constructing improved soil walls, core boring was done to take samples from ground surface. Fig. 4 shows the unconfined compressive strength and the modulus of deformation E_{50} of the samples. Fig. 4(a) shows that, with the exception of the vicinity of GL-25m in Project A, the strength of the improved soil was higher than the design strength. It has been pointed out that it is difficult to enhance the strength of high organic content soils with cement, so that might explain why there was no noticeable improvement at GL-25m in Project A. Fig. 4(b) shows the relation between E_{50} and the strength of the improved soil. In Project A it was E_{50}=675×qu, while in Project B it was E_{50}=250×qu.



(a) Improved soil strength

(b)Relationship between q_u and E_{50}

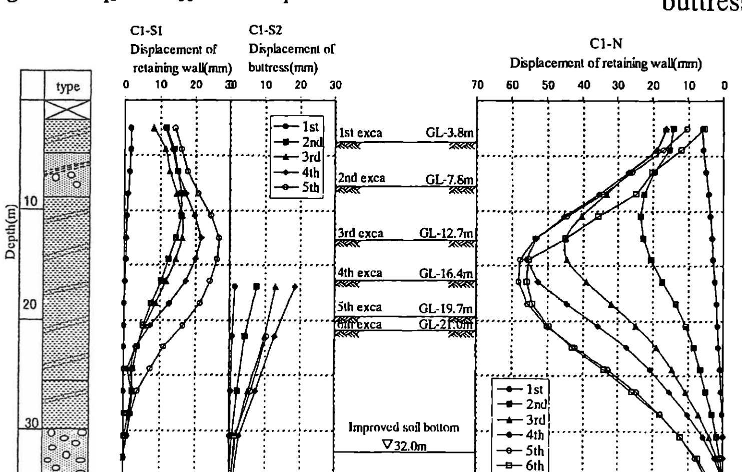
Figure 4. q_u and E_{50} of the improved soil

4 RESULTS OF MEASUREMENT

In order to compare the soil buttress and soil strut methods, we show the measured values for earth retaining wall displacement at Points C1-S1 and C1-N in Project A and Point No. 1 in Project B, where excavation depth was fairly equal. For added comparison, we also include measured values for displacement at Point C1-S2 (see Fig. 3a) of an improved wall near Point C1-S1 in Project A.

4.1 Project A (Points C1-S1, C1-S2, C1-N)

Figs. 5(a) and (b) show the depth distribution of lateral displacement of the earth retaining walls. Lateral displacement gradually increased during excavation, reaching its maximum near the excavation bottom. At Point C1-S1 near the railway embankment, the maximum was 27mm, while at the normal Point C1-N with no adjacent railway embankment the maximum was 58mm. This was relatively small for displacement of an earth retaining wall at an excavation of about 20m depths in extremely soft ground, confirming that the soil buttress method effectively reduced displacement.



(a)Point C1-S1, S2

(b) Point C1-N

Figure 5. Displacement of earth retaining wall and improved soil wall (Project A)

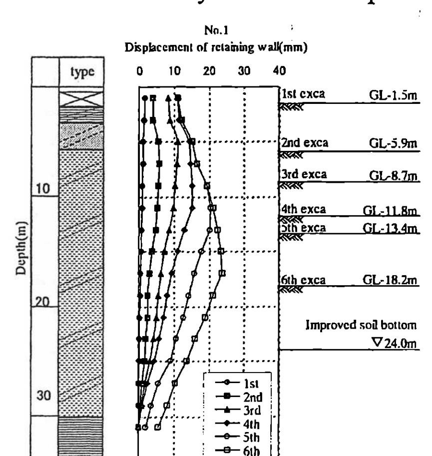


Figure 6. Displacement of earth retaining wall (Project B)

Here we should note that the head displacement at the fifth and sixth excavations at C1-N tended to be rearward, so corrections were made to make this displacement equal to that of the fourth excavation. At that time, the temperature during the fourth excavation, and the fifth and sixth excavations, rose about 10℃, so the correction was made taking into consideration the expansion of the first floor in the top down method that was caused by the temperature rise. As for the depth distribution, the depth-displacement curves in the figure are nearly linear, especially below the excavated bottom. We speculate that this was because the buttress improvement increased the flexural rigidity of the earth retaining wall. While there were significant differences in deformation between Points C1-S1 and C1-N, this may have been due to differences in buttress configurations. In addition, Fig. 6(a) shows the displacement distribution of Point C1-S2 in the improved soil wall near Point C1-S1. Although the planar positions were different, the displacement at Points C1-S1 (earth retaining wall) and C1-S2 (improved soil wall) show the same distribution pattern and size. This indicates that buttress-type improved soil walls spaced at a certain interval were able to reduce displacement of the wider part of the earth retaining wall than the width of the improved soil wall itself.

4.2 Project B (Point No. 1)

Fig. 6 shows the depth distribution of lateral displacement of the earth retaining wall at Point No.1. As with Project A, lateral displacement gradually increased as excavation progressed, reaching its maximum (about 24mm) near the excavation bottom. This was a relatively small value for displacement of an earth retaining wall with a 7m-long wall penetration, 18m-deep excavation in extremely soft ground, confirming that the soil strut method was effective in reducing displacement. Here we should note that as with Project A, the head displacement at the fifth and sixth excavations tended to be rearward, so corrections were made to make this displacement equal to that of the fourth excavation. As for the depth distribution, the depth-displacement curves in the figure are nearly linear, especially below the excavated bottom. We believe that this was because the strut type improvement increased the flexural rigidity of the earth retaining wall.

4.3 Comparison between the soil buttress and soil strut methods

To compare the two methods, each displacement of the earth retaining walls was defined as in Fig. 7. The relation between depth of excavation and maximum displacement of the earth retaining walls is shown in Fig. 8. During the early stages of excavation in the soil buttress method (about 7 m at Point C1-S1, about 15m at Point C1-N), the gradient was steep, but became gentler as excavation progressed. The reason that displacement was not reduced in the early stage may have been because improved width B was small in relation to the depth of wall penetration D_n. In contrast, the gradient in the soil strut method was relatively stable, and the displacement reduction effect was large from the early stages onward.

Next, we compared the displacement of the improved soil walls. Lateral, rotational, bending and shear displacement composed the excavation bottom displacement of the earth retaining walls with soil

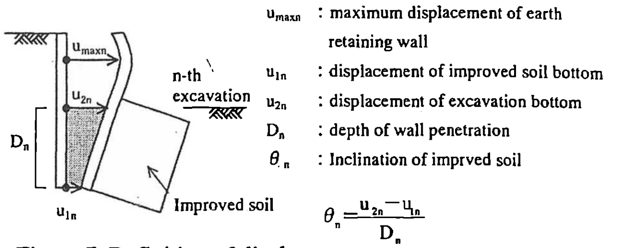


Figure 7. Definition of displacement

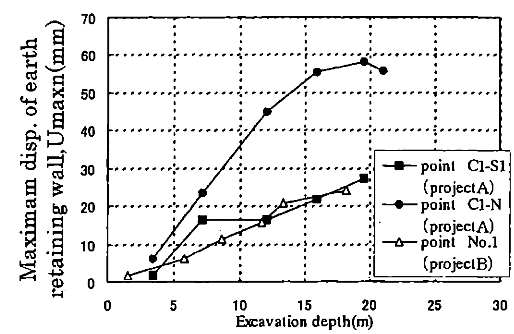


Figure 8. Changes in displacement of earth retaining wall

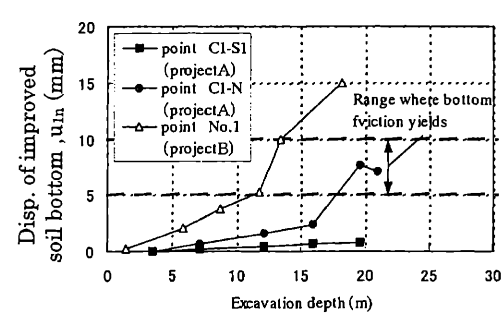


Figure 9. Changes in displacement of improved soil bottom

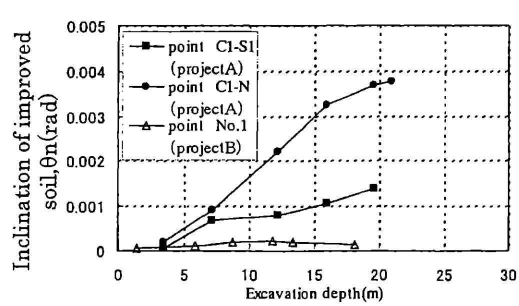


Figure 10. Changes in inclination of improved soils

improvement. Bending and shear displacement are usually small enough to be ignored, so only displacement occurring at the bottom (u_{1n}) and excavation bottom (u_{2n}) are considered. Fig. 9 shows the relation between bottom displacement of an improved soil wall and excavation depth. At Point C1-S1, bottom displacement was small, and increased linearly. However, at Points C1-N and No. 1, bottom displacement increased dramatically at a certain excavation stage, and it appears that friction on the improved soil wall bottom yielded. The displacement at this time was about 5mm to 10mm and the displacement pattern resembled the skin friction behavior of the piles (Sakaguchi, 1978). Fig. 10 shows the inclination of an improved soil wall, θ_n in which rotational displacement ($u_{2n} - u_{1n}$) is divided by depth of penetration D_n. At Point C1-N, the gradient of the line in the figure became gentle starting at the point in Fig. 9 where bottom displacement increased dramatically. A possible explanation for this is that lateral soil resistance (passive soil resistance) on the side of the improved soil wall was working because friction at the bottom of the improved soil wall yielded, and the resistance accounted for by rotation decreased. In contrast, in the soil strut method the gradient of the line in the figure did not change even after the displacement of the improved soil wall bottom had increased dramatically. This may have been because there was much lateral resistance (compression resistance) of the side of improved soil wall, so rotation would account for little resistance from early on.

5 EVALUATION OF MEASURED BEHAVIOR USING THE SPRING-BEAM ANALYSIS MODEL

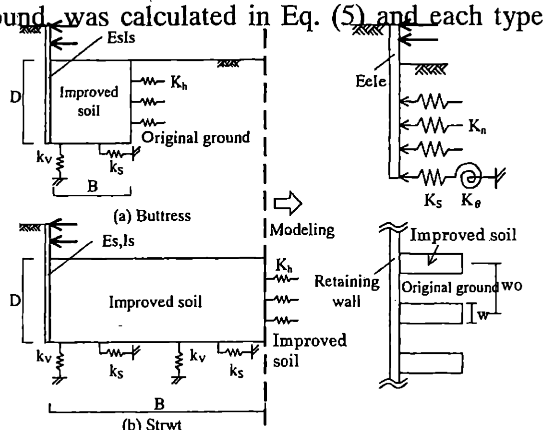
5.1 Method for analyzing earth retaining wall displacement using a spring-beam analysis model

To predict the displacement of an earth retaining wall that has wall-type soil improvements, FEM analysis is often used as an effective means of evaluating the stress of an improved soil wall, modeling of ground resistance. However, since the spring-beam analysis model is usually used in designs, we investigated an analytical method that employs this model. As we can see in Fig. 11, the resistance of improved soil walls in both methods is divided into bottoms and sides.

The bottom resistance consisted of rotational and shears resistance, which was expressed by rotation and shear springs of the bottom edge of the earth retaining wall in the analysis. Eq. (1) was used to calculate rotation spring. In the equation, k_v, the vertical subgrade reaction coefficient of the bottom, takes into account the decline in ground rigidity that is caused when excavation removes a load. While shear resistance was not affected by the improved width, shear spring in the beam-spring analysis model was calculated with Eq. (2) using improved width B (upper limit of D). In addition, k_s, the shear subgrade reaction coefficient of the improved soil wall bottom was calculated in Eq. (3) using the analytical method of the skin friction behavior of the piles (Akino, 1992).

The resistance of the side-end of the improved soil wall was calculated using the lateral subgrade reaction coefficient of the original ground in the case of the soil buttress method, and the compression spring of the improved soil wall (Eq. 4) in the case of the soil strut method. In addition, for the ultimate resistance the value for original ground was used in the case of the soil buttress method, and the value calculated using adhesion c that was increased by the improvement was used in the case of the soil strut method.

The flexural rigidity of the beams in the sections with the improved soil walls derived from a combination of steel materials and improved soils. However, as the improved width increased, it did not appear to behave as a rigid body, so the maximum improvement width B was set at depth of penetration D. Although the improved soil walls and the ground between them are considered to behave as one unit, to make a safety evaluation, the coefficient of decline of improvement effect, which considered spacing between the improved soil walls and the ground, was calculated in Eq. (5) and each type of



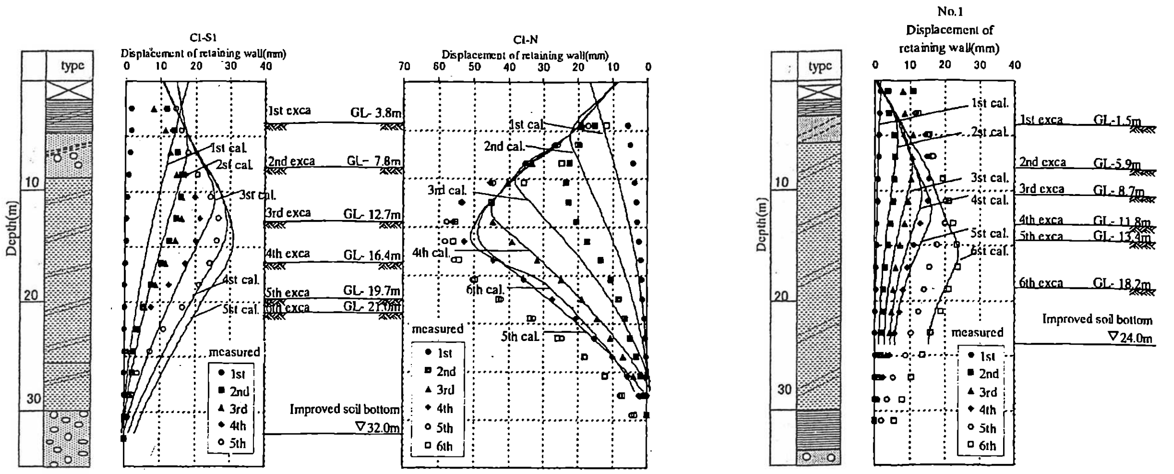
$$K_\theta = \frac{1}{12} k_V B^3 \quad (1) \qquad K_h = \frac{E}{B} \quad (4)$$

$$K_S = k_S B \quad (2)$$

$$k_S = \tau_{max}/1.0 \quad (3) \qquad \alpha = \frac{w}{wo} \quad (5)$$

K_θ : rotational resistance spring
K_S : shear resistance spring at the bottom
k_V : vertical subgrade reaction coefficient
k_S : shear subgrade reaction coefficient
K_h : latelal resistance spring
EeIe : flexural rigidity of a combination of beam and improved soil

Figure 11. Modeling in spring-beam analysis



(a)Point C1-S1(Project A) (b)Point C1-N(Project A) (c)Point No.1(Project B)

Figure 12. Comparison between analytical and measured values

resistance was decreased. The rotation and shear spring of the improved soil wall bottom in both methods and the ultimate resistance and lateral spring in the soil strut method were decreased using α.

5.2 Comparison with measured behavior

Fig. 12 shows a comparison between the analytical values obtained with the beam-spring analysis model and the actual measured values. Table 1 shows the ground and improved soil properties used in this analysis. For Project A, the analyzed value was much higher than the measured value at the first excavation, but the two values were very similar at the final stage of excavation. The reason for the discrepancy at the first stage may have been that soil buttress resistance was underestimated because it was evaluated based on the improved soil sizes of the final excavation stage. The analytical values for Project B showed good agreement with the measured values at each stage of excavation. In the soil buttress method, the improved soil wall was penetrated into a hard ground so there was little bottom displacement. However, in the soil strut method the improved soil wall was not embedded into a hard ground so the bottom of the earth retaining wall moves. The characteristics of both methods can be evaluated accurately using the beam-spring analysis model. This leads us to conclude that the use of the beam-spring analysis model and the methods of establishing the constants were appropriate.

Table 1. Ground and improved soil properties

	ground			soil
	kv (kN/m^3)	ks (kN/m^3)	kh (kN/m^3)	E (MPa)
Project A	90000	7000	12000	1350
Project B	26500	6900	-	675

6 CONCLUSIONS

In relatively close proximity in soft grounds in Tokyo, two types of soil improvement, soil buttresses and soil struts were undertaken for construction projects where excavation depths and ground conditions were nearly equal but the floor area was significantly different. This report has examined the displacement behavior of improved soil walls and earth retaining walls during such work, with the following findings:

1) In excavation work of about 20m depths in soft ground, we found that both methods were effective in reducing displacement.

2) Using a beam-spring analysis model in which the resistance of the improved soil walls is replaced with the spring at the bottom of the earth retaining wall, we found that, to a certain extent, we could evaluate the displacement of earth retaining walls with two-types of soil improvement method.

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