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Deformation behaviour and heaving analysis of deep excavation

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ABSTRACT : This paper presents the results of an experimental study for an application of the deep mixing method using fly ash and cement (referred to as FGC-DM) to a deep braced excavation. Centrifuge model tests on three bottom conditions (unimproved, and two improved grounds with q_u = 100 and 400 kPa below a 16 m deep excavation) were conducted. As results, a small heave of about 5 cm in prototype scale was indicated in the case of q_u = 400 kPa. In other two cases, magnitudes of heaving of over 20 cm due to the excavation were observed. It is confirmed that the deep braced excavation stabilized by the FGC-DM method in strength of about 400 kPa is enough stable against the basal heave.

1 INTRODUCTION

The deep mixing method of ground improvement using coal ash (referred to as FGC-DM) is now being developed as one of the technology making effective use of coal ash (Asano et al., 1996). In this method, which was derived from the existing cement deep mixing (CDM) method, slurry composing of coal ash, cement and water is injected and mixed with soils in the ground. The FGC-DM method is expected some advantages as follows;

1) high quality performance
2) controllable strength

They mean the improved ground can be uniformly stabilized even in relatively low strength.

Some applications of the FGC-DM method to the excavation work have been performed. Azuma et al. (1999) carried out a trial excavation of 4.5 m deep, 10 m long and 5.2 m wide in the ground improved partially beneath the excavation. Ohishi et al. (1997) performed the centrifuge tests on the 2D excavation of 6 m deep with sheet pile and one strut in the improved ground. They obtained the fundamental data for such a design and confirmed its safety and availability.

The objectives of this study are the confirmation of stability of 16 m deep braced excavation combined with the FGC-DM and steel pipe sheet pile walls in soft ground, and the check of the assessment method for the excavation in the ground involving stabilized layer. Especially the stability of the improved soil below the excavation against bottom heave is focused.

2 MATERIALS USED AND MODELLING

The materials used for this tests were a cohesive soil sampled from Isogo faced to Tokyo Bay (Isogo clay), normal Portland cement and coarse fly ash. The physical properties of Isogo clay are listed in Table 1.

Table 1. Index properties of material used.

	ρ_s g/cm³	w_L %	w_P %	I_p	< 75 μm %	< 5 μm %
Isogo clay	2.708	56.5	30.4	26.1	70.3	26.2

A deep excavation was planned at Isogo where the material was sampled. An outline of the deep excavation was 16 m deep, 50 m long and 25 m wide with five struts. A steel pipe sheet pile with a diameter of 700 mm and a thickness of 9 mm was used as an earth retaining wall. The ground below the excavation was improved by the FGC-DM method to a depth of 7 m beneath excavation level. The retaining wall extended to the same depth as the FGC-stabilized part and its toe did not reach a hard stratum. A strength of the improved soil under the bottom of the excavation was designed to be q_u of 400 kPa, which is a relatively low strength compared with other ordinary CDM methods.

A schematic diagram illustrating the layout of centrifuge model tests is shown in Figure 1. The points of modelling are as follows;

i) A 1/100 scale model is set in 100 G acceleration field.

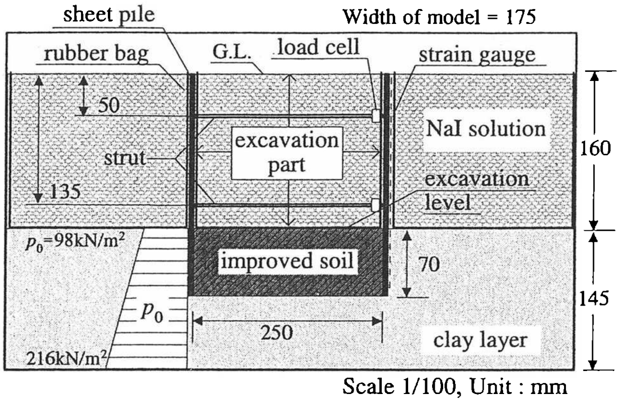


Figure 1. Layout of centrifuge model.

ii) A 2D braced excavation with 16 m deep and 25 m wide is modelled.
iii) Number of strut is changed to two from five. The position of the lowest strut, however, does not change.
iv) A model ground is combined with the soil layer below the excavation and heavy liquid parts above the base of the excavation. The preconsolidation pressure in the soil layer increases with depth as same as the distribution of effective overburden pressure predicted in the construction site. The heavy liquid (NaI solution) has ρ = 1.6 g/cm^3 as same as total unit mass of the clay ground.
v) The excavation is simulated by discharging of heavy liquid in the excavation part.
vi) The sheet pile used in the tests is made of steel plate with 2.6 mm in thickness which has the unit flexural rigidity in flight equivalent to that of the steel pipe sheet pile with 700 mm in diameter and 9 mm in thickness.

3 PREPARATION METHOD OF MODELS

Three cases having different bottom condition below the excavation were planned for the centrifuge model tests. Case-1 was an unimprovement case with q_u = 60 kPa at the upper part of ground. Case-2 was a designed stabilization case where the unconfined compression strength below the excavation was 400 kPa. Case-3 was also an improved case where the designed improved soil strength was q_u of 100 kPa.

3.1 *Preparation for clay ground*

The clay ground in all the cases were prepared by the combination with one-dimensional consolidation, seepage consolidation and centrifugal self-weight consolidation at 100 G.

The clay grounds before the seepage consolidation were prepared by the paste-preconsolidation method. Its procedure was as follows; Isogo clay with water content of 90 % was poured into the container with 800 mm in length and 175 mm in width. The piston with a drainage was inserted in the container. The pressure of 98 kPa was applied on the piston by air pressure. In this one-dimensional preconsolidation process, the pore water in the clay was drained from the top and bottom for 10 days. Then the seepage consolidation was conducted, in which water pressure of 108 kPa was applied as seepage force from top to bottom with constant loading of 98 kPa. By this process, the preconsolidation pressure p_0 in the clay layer was increased with depth as shown in Figure 2. It was confirmed by the self-weight consolidation to the time determined by 3 t method.

3.2 *Procedure of excavation tests*

The improved soil blocks for the case-2 and 3 were prepared in the other container with 250 x 70 x 175 mm. The suitable mixing proportions of clay-cement-fly ash having the designed strength after 14 days curing were determined by trial and error. The unconfined compression strengths of improved soils tested in case-2 and 3 were 368 and 110 kPa, respectively.

The sheet piles in all the cases and improved soil blocks in the case-2 and 3 were inserted in the clay ground trimmed the shape of those structures. Three rubber bags for the heavy liquid were installed on the clay ground. Then the strut system was set in the excavation part. This strut system had two struts with load cells and wales to keep the 2D condition. The strut was extended until the wale touched the sheet piles through the rubber bag. The sheet piles were connected with the counterweight to prevent the penetration of sheet piles due to their own weights.

The heavy liquid modelled as total unit mass of the upper ground was a solution of NaI, and was poured into three rubber bags with low gap among those liquid levels.

To measure the deformation of the ground, the lattice marks with 5 mm in diameter arranged at the

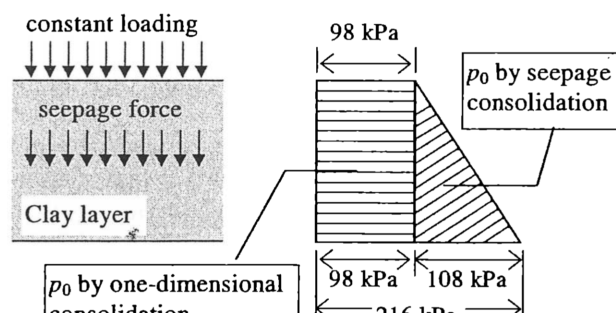


Figure 2. Distribution of p_0 in clay layer during seepage consolidation.

side of the ground that faced to glass wall in the container. And two vertical deformation indicators were also set at the excavation part.

The model was applied to acceleration of 100 G. And then the excavation with rate of about 8 cm/min in model scale (about 1.1 m/day in prototype scale) was started by discharging of the heavy liquid. During the excavation and its subsequent stage, the reaction of struts, the deformation of ground and strain in the sheet pile were measured.

The results of the centrifuge tests are represented in the prototype scale.

4 TEST RESULTS AND DISCUSSIONS

4.1 *Confirmation of clay ground*

Figure 3 shows the tip resistance distributions obtained from miniature cone penetration tests on the clay grounds after the excavation tests. The tip resistance almost linearly increases with depth of ground except for 10 mm from the ground surface. Kido et al. (1997) stated that the tip resistance distributions on the grounds applied and released centrifuge acceleration of 60 G were almost the same. From the previous knowledge, the distribution of the tip resistance shows the ground condition where the preconsolidation pressure increases with depth.

Figure 4 shows the water content distributions of the clay grounds tested. Water content in all the cases decreases with depth of ground with small scatter. Water contents at the top and bottom parts of the clay ground are calculated 47 and 42.4 % using the preconsolidation pressure in the seepage consolidation and the e - log p relation obtained from the oedometer tests. These calculated values are almost mean values at both the positions. These results prove that the clay ground had the designed soil profile.

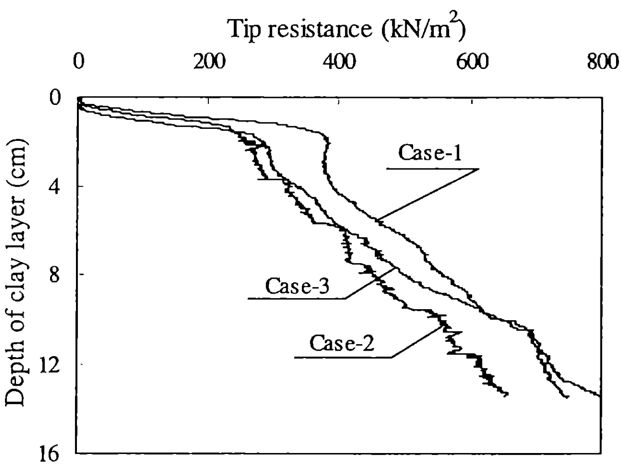


Figure 3. Cone penetration tests on clay layers.

4.2 *Behaviour of strut loads during excavation*

Figure 5 shows the responses of strut loads during excavation. The level of heavy liquid corresponds to the excavation level. The strut loads acting on the lower strut increase with the progression of excavation. The magnitudes of the lower strut load in the case-2 and 3 are about 40 and 80 ~ 90 % of that in the case-1 at any time, respectively. The strut loads acting on the upper strut in all cases increase up to excavation of 6 ~ 7 m deep although that in the case-2 is extremely small. With the progress of excavation, the upper strut loads in the case -1 and 3 decrease, but that in the case-2 is kept a constant.

The resultant force of strut loads at the end of excavation in the case-1 and 3 are almost the same. These values are approximately equivalent to the resultant force of the liquid pressure on the back side which is acting on the upper part of sheet pile from the excavation bottom. In the case-2, however, the resultant force of strut loads at the end of excavation is only about 35 % of that in the case-1 or 3. This means that a considerable part of the backfill pressure is sustained by improved soil.

As the later statement, large deformation during excavation occurs in the case-1 and 3. The deformation pattern of the ground is regarded as the rotation with its center at the lower strut. The pressure of the backfill is, therefore, thought to be concentrated at the lower strut.

4.3 *Behaviour of bending moment in sheet pile*

Figure 6 shows the bending moment distributions of the sheet pile in the case-1 during excavation. The distributions of bending moment change largely at the position of the lower strut. This indicates the sheet pile deforms with a fulcrum at the lower strut. Since the bending moment increases remarkably between the 9 m and 13.5 m excavation, it is considered that large deformation of the ground occurred in this term. The maximum value of bending moment

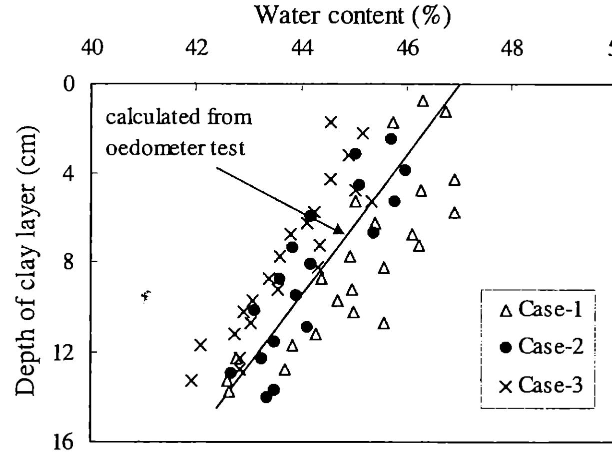


Figure 4. Distributions of water content in clay layer.

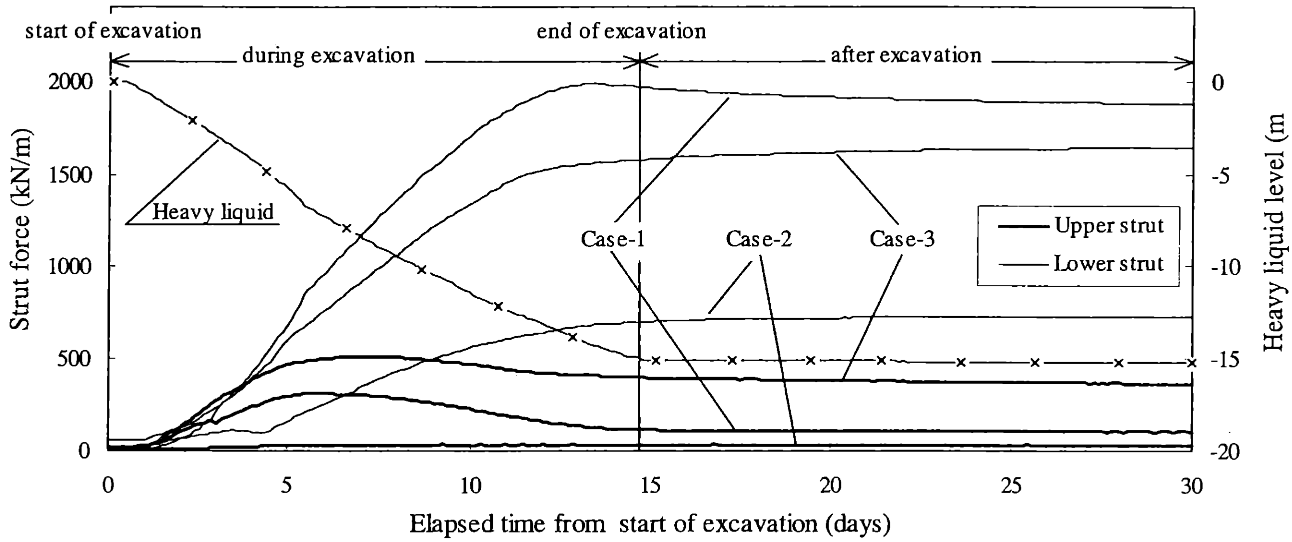


Figure 5. Changes of strut loads during excavation.

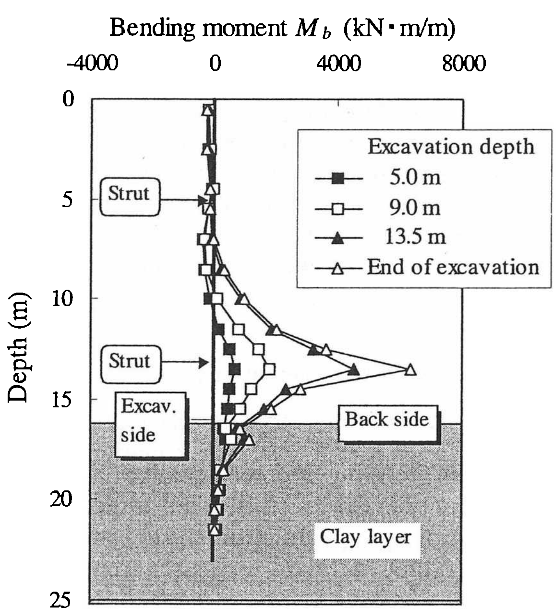


Figure 6. Bending moments due to excavation in test case-1.

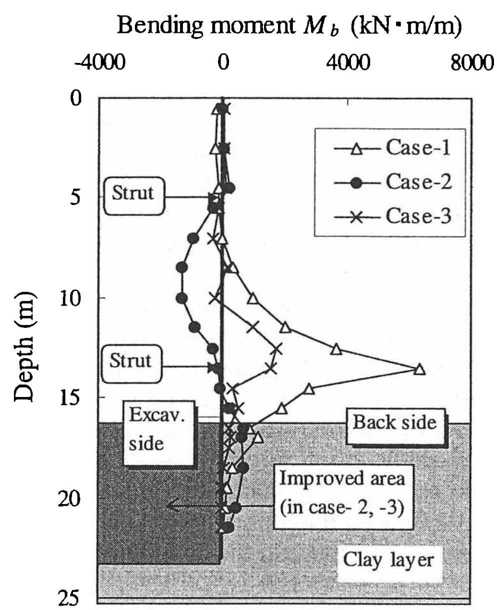


Figure 7. Comparison of bending moment distributions at the end of excavation.

reaches 6500 kN·m/m at the end of excavation.

Figure 7 shows the comparison of bending moment distributions at the end of excavation in three cases. The distribution in the case-2 is quite different from that in the case-1, and the maximum bending moment occurs at the middle position between the two struts. The maximum absolute value is not so large, a fifth of that in the case-1, this case is thought to be stable. The distribution in the case-3 presents the intermediate pattern between case-1 and 2. What the maximum value concentrates at the lower strut in the case-3 means the behaviour of the ground is close to that in the case-1. Therefore, the improved soil strength of 110 kPa can be thought to be not enough for the stability of the ground.

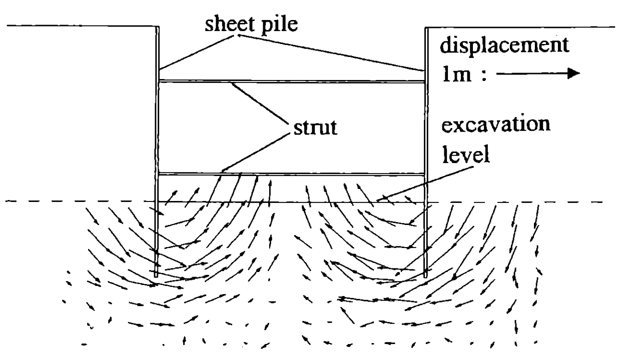
4.4 *Deformation behaviour of ground during excavation*

Figure 8 shows the ground displacements from start to end of the excavation using the photo-analysis. In the case-1, the ground deforms largely, the heaving of the base of excavation reaches 50 cm or more in prototype scale. The heaving in the case-3 is 20 cm or more. These cases clearly show circular sliding deformations centred at the position of the lower strut. On the other hand, the case-2 is not recognized large deformation.

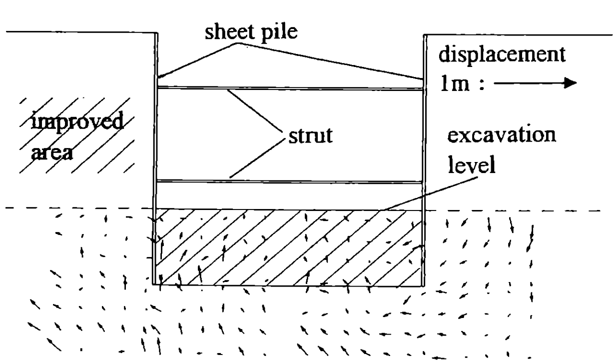
The bottom heave due to excavation in the case-2 shows in Figure 9. The vertical displacement close to the sheet pile is smaller than that far from the sheet pile. This behaviour also shows a convex round shape as same as the shape imaged from the deformation vectors in the case-1 and 3.

4.5 *Assessment of stability against bottom heave*

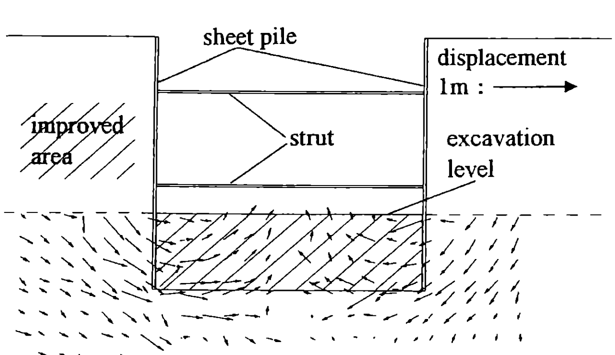
As shown in Figure 8, the deformation pattern of ground due to excavation can be characterized by the circular sliding shape centred at the position of the lower strut. In the method which was recommended by the Architectural Institute of Japan (1988), the circle with its centre at the lowest strut is assumed to pass below the wall, as shown in Figure 10. The safety factor is driven from the equilibrium of resisting and sliding moments. Using this concept, an assessment of the stability of each ground tested in centrifuge is tried. In this procedure, the strength



(a) Case-1



(a) Case-2



(a) Case-3

Figure 8. Ground displacements at the end of excavation.

of ground whose distribution is shown in Figure 10 is determined by the results of unconfined compression tests on specimens prepared under the designed conditions.

Figure 11 shows the relationship between excavation depth and safety factor. The x' which is the diameter of circular slip surface is the value at the minimum safety factor in any depth of excavation. In the case-1, the depth of excavation at safety factor of unity is read 14.4 m from this figure. Although large deformation in the ground occurred up to 13.5 m excavation in the test, this 14 m is close to the test result. Taking safety factor of 1.2 considering several uncertain factors, which is established by the Architectural Institute of Japan, the depth of excavation at stable condition is 12 m. It is good corresponding to the test result.

In the case-3, since the safety factor does not reach 1.0 even at excavation depth of 16 m (F = 1.05 at this depth), ground failure does not occur from this calculation. Taking safety factor of 1.2, however, the ground is to be critical when excavation depth is over 14 m and large deformation of the ground is predicted to occur. As shown in Figure 8 (c), the basal heave reached 20 cm in prototype scale and relatively large deformation of the ground occurred. The assessment by this method is in good agreement

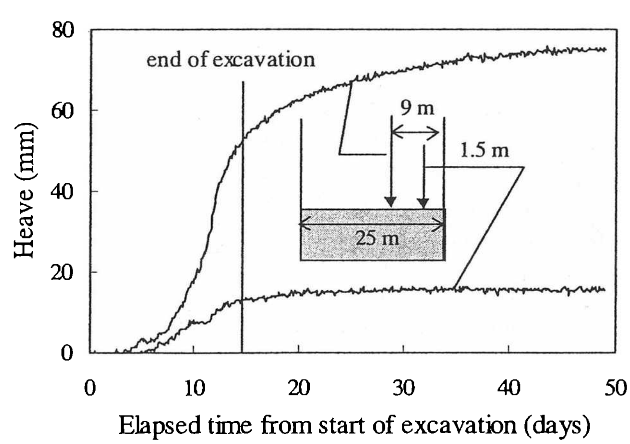


Figure 9. Bottom heave due to excavation (case-2).

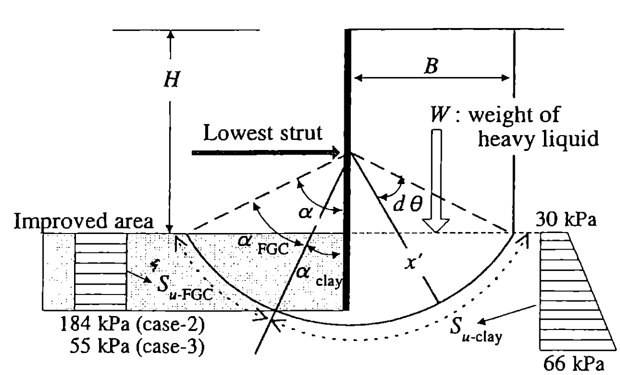


Figure 10. Examination against bottom failure by the present method.

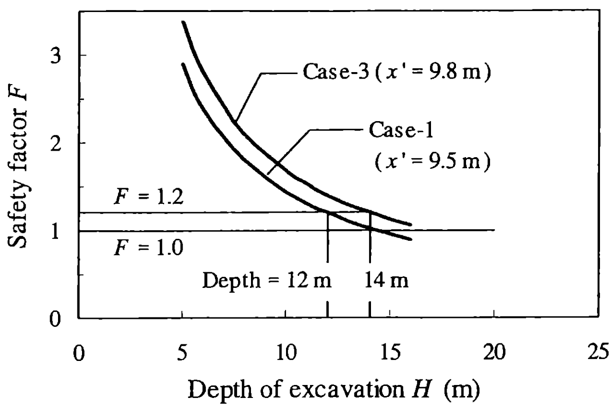


Figure 11. Relationship between H and F.

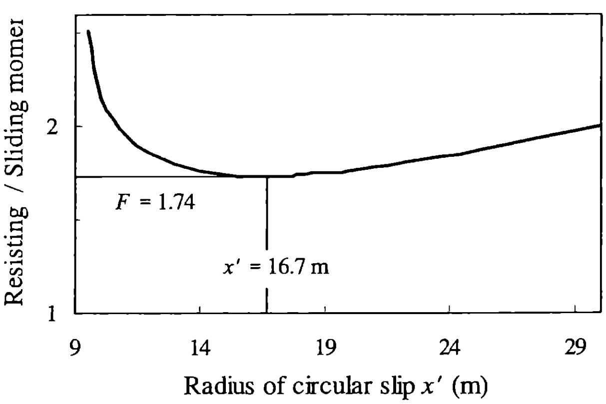


Figure 12. Estimation of the safety factor in the case-3.

with the test result of the case-3.

In the case-2, the safety factor could not obtain directly because the ground did not deform largely. Then the safety factor is the minimum value obtained from calculations with different radius of circular slip surface. Figure 12 shows the relationship between radius and ratio of resisting moment to sliding moment. The minimum value, that is safety factor, can be recognized 1.74 at x' = 16.7 m. This value can be explained the deformation behaviour in the case-2.

It is concluded that the assessment method based on the circular slip analysis can be useful for such a low strength stabilized ground. On the other hand, it may be difficult to apply this method to ground improved in high strength, because a deformation behaviour or a failure pattern is not possibly a circular shape through a hard improved part. In such a case, a numerical analysis, for example FEM, or another assessment method will need for the estimation of stability against heaving.

5 CONCLUDING REMARKS

The excavation tests on simplified models composed of clay ground and heavy liquid were performed to study the stability of the deep braced excavation with improved soil in excavation bottom. As results, it is confirmed that the deep braced excavation stabilized with such a low strength of 400 kPa is enough stable against the basal heave. The assessment method based on the circular slip analysis is useful for these type of improvement with such a low strength.

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