WORK FOR VOLUME REDUCTION OF CLAY IN THE NEW WASTE DISPOSAL AREA IN TOKYO PORT

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The New Waste Disposal Area is the final disposal site in Tokyo Port to dispose of the wastes such as municipal/industrial wastes, construction waste soil and dredged soil. Therefore, to utilize this area for a long time, measures for expanding its capacity or reducing the volume of wastes have been taken. The work for volume reduction of disposed clayey soils was carried out with the vacuum consolidation method at block C of the area to prolong its service life. In this project, the prefabricated vertical drains (PVDs) with a width of 150 mm and a thickness of 3.9 mm were used. The PVDs were installed in the clay layer at the average elevations of +1.5 to +33.8 m with a square spacing of 1.8 m with the negative pressure of 65kN/m², which is equal to the consolidation pressure increment, was applied for 310 days.

The trial period began in 2005 and the main project period started in 2007 and was completed in 2015. During this period, work was performed on 383,000 m²; the mean settlement volume was 5.13 m and the volume obtained through settlement consolidation was 2,167,000 m³. This means that the service life of this disposal area was extended for 2.3 years. This paper provides the general overview of this project with a focus on geotechnical engineering.

Key Words: volume reduction, vacuum consolidation, prefabricated vertical drain, clayey soil, execution

1. INTRODUCTION

Landfill sites have been secured for the processing and disposal of waste and other materials in the frontal sea area of Tokyo Port. Efforts to maintain and develop urban functionality as well as preserve the comfortable lives of the residents of Tokyo have been continued. Currently, the work to prepare a waste disposal area as a final disposal area within Tokyo Port is ongoing. There is a strong desire among the people of Tokyo for this area to have sufficient capacity to be used as a disposal area for as long as possible.

The New Waste Disposal Area consists of seven blocks. Each block started its construction work and was in operation one by one. Each landfill allows the disposal of not only waste, including household garbage produced within Tokyo’s 23 wards, but also the soil dredged from rivers within the metropolitan area and Tokyo Port. The dredged soil accepted by the New Waste Disposal Area is limited to that which cannot be used effectively elsewhere, including the soil that is dredged to maintain the water depth in areas such as rivers, canals, and harbors.
However, there is a limit to the volume that can be put into the landfill. Thus, it is important that we develop an approach to extend its life, in the same way as waste.

In this context, in block C (see Fig. 1) of the New Waste Disposal Area, volume-reduction work, including vacuum consolidation, has been carried out to increase the volumetric capacity of the landfill by causing major settlement of the clay layer, without loading the top of the clay layer with earth fill and other materials. Using this approach, the landfill can additionally hold a volume of dredged soil equivalent to the settlement volume. When selecting the method, since the volume-reduction work includes underwater work, a cap-type vacuum-consolidation method was applied. This method includes the use of the clay layer of the upper stratum as a sealing layer to control seepage without the need for an airtight sheet.

To undertake the volume-reduction project over the area as shown in Fig. 2, a trial period began in 2005 and the main project period started in 2007 and was completed in 2015. During this period, work was performed on 383,000 m²; the mean settlement volume was 5.13 m and the volume obtained through settlement consolidation was 2,167,000 m³.

This clay volume-reduction project was the first large-scale domestic project of its kind and significant knowledge was gained from a geotechnical engineering standpoint. Thus, in this paper, we provide a comprehensive summary of the main points related to the clay-volume-reduction project revealed over the course of this project.

The volume-reduction work was limited to the region 50–100 m inside of the revetments to prevent deformation of the revetments. Environmental factors were also considered. For example, the lower-end depth of the drains was set to 3 m above the lower-end of original seabed clay layer to prevent contaminated water from infiltrating the outer areas when filling with additional waste in the future. The original seabed clay layer and the layer of dredged soil above it were targeted for volume reduction.

1,18,546 drains, each having a width of 150 mm and a thickness of 3.9 mm, were arranged in a square formation 1.8 m apart at a mean depth between A.P. (abbreviation of Arakawa Peil) +1.5 m and –33.8 m (hereafter, A.P. is omitted). Negative pressure was continuously applied to the drains for 310 days.

In this paper, we discuss notable points about the volume-reduction project in section 2, explain the planned work for the project in section 3, describe the soil characteristics in section 4, discuss consolidation management in section 5 and evaluate the benefits of volume reduction in section 6.

2. SELECTING THE METHOD AND DEFINING THE SCOPE OF THE PROJECT

A cross-section of the planned landfill in block C is shown in Fig. 3. Block C is partitioned by the outer revetment and the middle-partition revetment. The managed landfill was intended to be used for waste products and dredged soil. The plan was to fill dredged soil up to +4.5 m and to further fill waste products and construction waste soil in alternating layers up to +30 m in the landfill.

When selecting the volume-reduction method, it was essential to avoid harmfully deforming the revetment. Further, it was important to control the seepage of the seabed (i.e., the clay layer) that fulfills a role as the water barrier in the managed disposal area, to keep waste products from infiltrating into the groundwater or ocean areas. The volume-reduction
method and the scope of the work are discussed below.

(1) Selecting the Volume-Reduction Method

The logic flow for selecting the volume-reduction method is shown in Fig. 4. The objective of volume reduction was to promote the settlement of clay due to consolidation, increase the capacity volume mainly for dredged soil and, thereby, extend the life of the New Waste Disposal Area. When adopting the direct surcharge method where the load such as embankment is directly applied, it is necessary to remove the embankment after consolidation. These methods were not adopted due to economic considerations and because of the duration of such projects. Instead, a vacuum-consolidation method was adopted, which includes applying negative pressure on the clay layer. The cap-type vacuum-consolidation method, which includes the use of capped prefabricated vertical drains (PVDs) and work to be done under water, was selected.

An overview of this method is shown in Fig. 5. As the characteristics of this approach, the surface layer of the clay layer, which is approximately 1 m thick and has a permeability coefficient of approximately $10^{-7}$ m/s or less, is used as a vacuum-sealing layer. This eliminates the need to lay an airtight sheet. A negative pressure is applied to the clay by connecting a drain hose from each PVD to a collecting hose which is connected to a vacuum pump via a header pipe; vacuum is applied to the header pipe.

(2) Defining the Scope of the Volume Reduction

Here, we define the horizontal and vertical ranges of the planned volume reduction.

a) Horizontal range

In vacuum consolidation for volume reduction, the isotropic stress works to cause both settlement in the areas surrounding the improved areas and horizontal displacement toward the improved area (see Fig. 6). This effectively displaces the revetment on the side of the improved area, leading to concerns that the seepage-control capabilities of the revetment may be lost. Therefore, we conducted a finite element analysis on consolidation focusing on the horizontal displacement amount of the caisson joints to predict the limit to the volume-reduction range which can be achieved while still securing the seepage-control capabilities of the revetment. A horizontal displacement pattern of the revetment was assumed to calculate the width of the joint displacement, $W$ (with a tolerance of 10 cm) according to equation (1) under the assumption that four 100 m box caissons will be affected by the ground deformation.

$$W = B \times \frac{L}{(n \times L_i)}$$  \hspace{1cm} (1)

Here, $B$ is the revetment horizontal displacement (as determined by the finite element analysis), $L$ is the caisson width, $L_i$ represents the length of one box caisson and $n$ represents the number of boxes being displaced. In the case of a caisson-type revetment, if the vacuum-consolidation areas are located 100 m or more from the revetment, the horizontal displacement amount of the caisson joints is within tolerance and seepage-control capability will be retained. As revetment formats differ, the vacuum-consolidation areas were planned to be 50–100 m inside the revetment, as shown in Fig. 2. The total area targeted for volume reduction was approximately 383,000 m². For further details regarding this analysis, see document 4).
b) Vertical range

When placing PVDs in the clay soil in the lower section of the waste disposal area and applying consolidation improvements, it is essential to ensure that the landfill functions as a controlled repository which requires contamination processing and other steps to be taken for waste products, based on the standard on the technology which affects final disposal of a final disposal place of non-industrial waste and industrial waste\(^5\). In particular, the landfill must have a vertical cross-section that would prevent the dispersion of pollutants, such as contaminated water from waste products, from infiltrating into the lower sandy layer via the PVDs.

According to the standard\(^5\), in areas where the permeability coefficient of the clay is \(10^{-7}\) m/s, it is necessary to ensure that the water seepage-control layer has a thickness of 5 m or more. Applying this consideration to the clay layers, which have a permeability coefficient of \(10^{-8}\) m/s and assuming that the permeation time is equivalent, the seepage-control barrier must have a thickness of 1.6 m or more\(^3\). To ensure that the lower water barrier base layer is secured, the following assumptions were made: there was a margin of error of 0.1 m within the 1.6 m thickness when placing the PVD; there was a variation of 0.5 m in the thickness of the lower sandy layer; the future settlement in the water barrier layer would be 0.6 m; and a safety margin of 0.2 m. Thus, the original seabed clay layer (permeability coefficient of \(10^{-8}\) m/s or less) of 3 m in thickness was ensured between the lower edge of the PVD and the upper edge of the lower sandy layer (see Fig. 7)\(^6\).

3. WORK UNDERTAKEN FOR THE VOLUME-REDUCTION PROJECT

The volume-reduction project included a trial period in 2005–2006 and a main project period in 2007–2015. A breakdown of the project is shown in Fig. 8. The trial period involved work on blocks A and B and the main project period involved work on 118 blocks except blocks A and B. The size of each block was determined based on the area to which a single vacuum pump was assigned, approximately 3,200 m\(^2\).

The volume-reduction methods for the trial period and the main project period are described as follows.

(1) Trial work

The volume reduction was carried out in block A (60 × 60 m) and block B (61.2 × 61.2 m). PVDs were placed within a 2.0 m square in block A and a 1.8 m square in block B. The PVDs normally used for ground improvements are 100 mm wide and 3.9 mm thick but the PVDs used for this project were 150 mm wide and 3.9 mm thick. The use of wider PVDs increased the effective diameter of the drain materials, \(d_w\) (\(d_w = 5\) cm with a 10-cm width and \(d_w = 8\) cm with a 15-cm width). This increases the drain interval, \(d\), required to achieve the same mean degree of consolidation in the same amount of time and reduces the number of PVDs installed. In most cases, \(d\) is 1.8 m or less, but for this application, a test in which \(d\) is equal to 2.0 m was performed to investi-
gate whether the settlement rate can be explained by Barron’s consolidation theory. The PVDs were placed between +1.0 and –29.0 m and approximately 30 m long with \( d = 1.8 \) m in one location and \( d = 2.0 \) m in another. The ground levels were measured using lead at each PVD position. Ensuring that a 1.0 m seepage seal layer was retained, the PVD cap sections were placed 1.0 m below the surface of the sea floor (see Fig. 7).

Negative pressure was applied for 204 days, which correspond to a mean consolidation degree of 80% when the horizontal and vertical consolidation coefficients, \( c_h \) and \( c_v \), were equal to 120 cm\(^2\)/day (see Fig. 13). The PVD placement was carried out using a PVD placement vessel (Photo 1). The parts of the vessel could be transported over land and assembled at the area, which was useful for operations in closed bodies of water. The placement vessel was 17.83 × 43.15 m in size and contained seven vertical and eight horizontal unifloat units. As the PVD placement system, a sideways rail system in which a rail is laid on the vessel and a PVD penetrating machine is positioned on the rail was adopted. The draft was shallow at 1.0 m (the water depth was approximately 2 m). The placement vessel was moved to designated location to place 18 PVDs with \( d = 1.8 \) m and 15 PVDs with \( d = 2.0 \) m. The vessel achieved an underwater placement capacity of approximately 70 units/day, which was virtually the same as that on land.

A projected working space of the negative pressure hose was installed in front of the placement vessel (as shown in Photo 3) and each drain hose protruding through the water from each placed PVD was connected to the collecting hose. Then, the collecting hose was connected to the header pipe and extended to the revetment. The vacuum pump facility, consisting of a vacuum pump, a water storage tank, a negative pressure tank, a submersible pipe and other intermediate infrastructure, was placed on a barge floating on the water. The negative pressure tank was placed 1.5 m below the surface of the water with a mount attached to the tip of the barge (see Fig. 9) to create a water level difference of 1 m between the disposal area and the inside of the negative pressure tank. In the presence of this water level difference, the water level within the PVD is 1 m below the water level in the clay layer, which leads to an additional consolidation load (seepage consolidation) of approximately 10 kN/m\(^2\) on the clay. To be conservative, the volume-reduction capacity in the settlement calculation did not consider the additional consolidation load due to the water level difference. The water level in the negative pressure tank is maintained using water level detection sensors and the submersible pump as needed. The consolidated effluent water from the clay entered the negative pressure tank and was discharged via the water storage tank through the operation of the submersible pump.
(2) Main volume-reduction work

In the main volume-reduction work \(^8\), a dedicated PVD placement vessel named “VCD-Triton” \(^9\) (Photo 2) was newly manufactured in Awajishima in Hyogo Prefecture before being brought into service at Tokyo Port. This placement vessel was 24 m wide and 60 m long and weighed approximately 1,400 tons. Thus, a 3,000-ton hanging crane ship was used to hang the vessel. After the two 50-m-long leaders of the PVD placement device attached to the vessel were removed, the vessel was hanged from the outside of the revetment into block C at night with little influence on aircraft. Then, the long leaders were reattached.

The placement vessel had a shallow draft of 1.5 m and could be used to place PVDs at a depth of approximately 40 m below the surface of the water. The mean water level in the ground was +3.0 to +3.5 m and the PVDs were placed at a length of approximately 30–38 m from the mean ground surface (1.5 m) to a mean depth of −33.8 m (in the range from −31.5 to −36.5 m) (see Fig. 8). The installed PVD length was varied according to the depth of the original seabed clay layer bottom, which was digging to the southwest direction.

The placement vessel included a sideways rail system with a rail laid on the vessel and two PVD placement devices positioned on the rail. The long leaders maintaining the casing were supported by two backstays. With a support wire fixed to the deck, the vessel was capable of withstanding storms with wind speeds up to 60 m/s. The PVDs with 150 mm wide and 3.9 mm thick were arranged in a 1.8m-interval square patterns. The placement vessel was equipped with a spud carriage system \(^9\), which inserts and fixes spuds in the clay ground to finely adjust the position of the placement vessel hydraulically by approximately 2.5 m in the horizontal direction using the spuds; the system enabled swift and accurate movement to each placement position. The vessel also included an anchor winch to fix the placement vessel in the designated location for a short period of time. The vessel could place 24 PVDs from a single position and could install an average of 194 PVDs per day (10 hours of operation).

A projected working space of the negative pressure hose was installed in front of the placement vessel and each drain hose protruding through the water from each placed PVD was connected to the collecting hose. (Photo 3, Photo 4). The collecting hose was connected to a header pipe and extended to the revetment. A vacuum pump was placed on the revetment and the negative pressure tank was placed 1.5 m below the surface of the water as in the trial work. Negative pressure was applied for 310 consecutive days (corresponding to an average consolidation degree of 90%) and much water was discharged from the clay (Photo 5).

4. SOIL CHARACTERISTICS

(1) Typical soil profile

The soil profile at this area is shown in Fig. 10. This soil has a Nanago formation comprising cohesive or sandy soil below from −45 to −40 m and a Yurakucho formation, which is herein referred to as
the original seabed clay layer, above it. The top layer from −12 to −10 m downward, corresponding to the former seabed, has a thickness of 30–35 m and includes the upper Yurakucho formation made up of a comparatively uniform clay layer and a lower Yurakucho formation in which cohesive soil and sandy soil exist are interbedded. In the areas targeted for volume reduction with the aim of increasing the landfill volume for dredged soil, the Yurakucho formation was removed down to approximately −17 m (see Fig. 3), and the dredged soil was filled on the original ground of the Yurakucho formation.

The landfill for dredged soil was started in 2001 and the addition of waste continued during the project period in areas that had already undergone volume reduction. The dredged soil derived from maintenance dredging and other sources. Quantitative data, such as the sand content, was not examined, but was revealed to contain a high content of sand from watch observation. The landfill disposal method at the area for dredged soil disposal is shown in Fig. 11 and described as follows.

1) The dredged soil was dredged by a grab dredger and transported outside the disposal area by an open bottom-type soil-carrying barge. Then, the bottom cover of the soil-carrying barge was opened above the seabed near the place placing the anti-pollution soil transporter within the pollution prevention frame and the dredged soil was poured in.

2) Using the sludge transportation pump on the anti-pollution soil transporter within the pollution prevention frame, the dredged soil was suck up along with the surrounding sea water and transported to the disposal area via the pump pipe.

3) There was a soil spreading vessel for the sediment within the disposal area. By moving the soil spreading vessel, the dredged soil can be sedimentated over a wide range with a thickness of approximately 1 m at a time.

When depositing the dredged soil into the landfill, it is thought the sand is sedimentated in the vicinity of the soil spreading vessel while the silt and clay components are sedimentated in a location far from the soil spreading vessel. Further, it is surmised that as the soil spreading vessel is moved over a wide range to prevent bottoming, the intermediate sand layer is formed within the dredged soil layer in a localized way. To evaluate the localized deposition of sand components, a new method of classifying and measuring the sandy layer and clay layer of the soil has been developed based on the mandrel penetrative resistance when placing PVDs. This approach has been used to assess settlement and differential settlement. See reference documents 11)–14) for additional details.

(2) Clay soil characteristics

The characteristics of the clay soil were evaluated in detail during the trial period to assess the consolidation characteristics and thereby predict the settlement. Assuming that the clay consolidation characteristics in the main project area would be the same as those in the trial areas, only the initial soil parameters were used to predict the settlement across the entire project area. Specimen sampling and radio isotope (RI) cone penetration tests, physical tests, unconfined compression tests and incremental loading consolidation tests were conducted before and after improvements in the trial area. However, during the main volume-reduction work, only RI cone penetration tests were carried out prior to the improvements. When conducting the RI cone penetration tests, as the void ratio (calculated with 100% of saturation degree), tip resistance, skin surface friction and the depth distribution of the pore water pressure were known, it was possible to classify the clay layer and sand layer and to determine the initial void ratio needed to predict the settlement in the clay
The soil characteristics before the clay soil consolidation in blocks A and B are shown in Fig. 12. The mean void ratio, $e_0$, for the clay in the dredged soil layer was 2.33, the mean unit weight, $\gamma_t$, was 14.7 kN/m$^3$ and the compression index, $C_c$, was 0.88. According to the measured consolidation yield stress, $p_c$, for both blocks, which had intermediate sandy layers within the dredged soil layer, the self-weight consolidation was virtually complete. For the original seabed clay layer, these characteristics were $e_0 = 3.07$, $\gamma_t = 13.7$ kN/m$^3$ and $C_c = 1.34$; according to the $p_c$ results, consolidation due to the self-weight of the dredged soil had not progressed so both blocks were in an unconsolidated state of approximately $\sim$40 kN/m$^2$. However, in block A, where the dredged soil layer was underlain by a sandy layer and the dredged soil layer acted as the load, self-weight consolidation had progressed in the surface layer of the original seabed clay layer within an area of approximately 5 m. The clay consolidation coefficient, $c_v$, was 120 cm$^2$/day in both the dredged clay layer and the original seabed clay layer. Fig. 13 compares the dredged soil layer and original seabed clay layer in terms of the values of $c_v$, and the relation between void ratio, $e$, and effective stress, $p$.

The results of the soil tests of sampled specimens carried out before and after the consolidation work are shown in Fig. 14. In the dredged soil layers in both blocks A and B, the unconfined compression strength and the consolidation yield stress increased as a result of the consolidation but no notable decrease in the void ratio was observed. In the original seabed clay layer, the unconfined compression strength and consolidation yield stress increased after improvement while the void ratio decreased.

The results of the RI cone penetration tests in block B are shown in Fig. 15. The sandy layer had a void ratio of approximately 1.0 smaller than the clay layer, a large tip resistance and skin surface friction, and a pore water pressure close to the hydrostatic pressure. Fig. 15 shows the position of the intermediate sandy layer after improvement in green. When comparing the soil characteristics of the clay layer before and after the consolidation work, although it cannot be determined based on the tip resistance results alone, there is the decrease of void ratio, $e$, the increase of skin surface friction and pore water pressure indicating the improved effect. The results from the sampled specimen tests, as well as the results measured using the RI method had the clear improved effect due to consolidation.

(3) Sandy soil characteristics

The particle composition through the depths of blocks A and B is shown in Fig. 16. In the dredged soil layer, areas with a high sand content forming a drainage layer are commonly seen. Based on these observations, it is concluded that the self-weight consolidation of the clay is virtually complete. On the other hand, the original seabed clay soil layer is a virtually uniform clay layer.
Fig. 14 Change of the soil properties with depth before and after consolidation.

Fig. 15 Results of the RI cone penetration tests before and after consolidation\(^{16}\).

Fig. 16 Change in grain size distribution with depth\(^{17}\).

When predicting settlement, it is necessary to add the amount of settlement of the intermediate sandy layer within the dredged soil layer to the amount of settlement of clay. When starting the main volume-reduction work, the settlement amount of the intermediate sandy layer was thought to vary according to the fine particle content and relative density. However, due to the need to reduce project costs, further detailed tests were not performed and the settlement of intermediate sandy layer was calculated from the changes in the mean void ratio measured via the RI cone penetration test before and after the trial period. According to the results shown in Fig. 17, the mean void ratio before consolidation, \(e_0\), was 1.25, and that after consolidation, \(e_f\) was 1.10. Therefore, the settlement amount in the sandy layer was calculated by using equation (2) and multiplying \(\Delta e\) by the layer thickness. Substituting \(e_0 = 1.25\) and \(e_f = 1.10\), equation (2) reveals that \(\Delta e\) was 6.7%.

\[
\Delta e = (e_0 - e_f) \times 100 / (1 + e_0) \quad (2)
\]

5. CONSOLIDATION MANAGEMENT

The most important tool in volume-reduction projects is the time-settlement curve, and using a hyperbola curve fitting to the actually measured settlement values to characterize the changes in the mean consolidation degree, it is necessary to confirm that the desired mean consolidation degree has been achieved. Additionally, when conducting vacuum consolidation, it is necessary to constantly monitor whether negative pressure is being applied to the clay and to take appropriate measures if pressure is not being applied in certain areas. This is because, while the fill load is constant, the negative pressure may fluctuate greatly due to the operation of the vacuum pump or other components. The continuous application of sufficient negative pressure leads to the de-
sirable settlement of clay layer in the resulting consolidation.

The negative pressure applied to the clay was measured by using a piezometer attached to the tip of the PVD in the center of each of the blocks as a representative value of the PVDs within the clay layer. The reason for this is that if the position measuring the negative pressure was far from the pump and there was a leak of negative pressure, the negative pressure measured would reflect the leak immediately.

(1) Negative pressure management

The negative pressure measurements from blocks A and B are shown in Fig. 18. The source pressure in the diagram represents the negative pressure measured in the vacuum pump room and was virtually constant between 80 and 90 kN/m². However, at the PVD tip, the consolidation pressure increment converted from the result of pore water pressure measured gradually increased from approximately 50 kN/m² to 70 kN/m² after 100–120 days in both blocks. This gradual increase is thought to be due to water being absorbed from the surrounding area via the intermittent sandy layer within the dredged soil layer due to the fact that drainage volume at the start of the project was high, approximately 10–100 times the settlement volume. This gradual increase in consolidation pressure was observed in virtually all of the improved blocks. Generally, for the cap-type vacuum-consolidation method, a seepage-control seal is applied at the manufacturing plant at the depth of the intermediate sandy layer (as identified in advance using a cone penetration test) and 1 m above and below this area to prevent water absorption from the intermediate sandy layer. This aims to prevent negative pressure from being propagated outside the improved area and thus prevent settlement from occurring in adjacent areas. However, the layer for volume reduction targeted in the main project period included the intermediate sandy layer. Because the revetment outside of the improved area serves as a seepage-control structure, water would not be absorbed from outside the revetment. For this reason, PVDs without seepage-control seals were used. The gradual increase in negative pressure after negative pressure loading is thought to additionally be because the density of the intermediate sandy layer increases as the negative pressure propagates and the permeability coefficient decreases. According to the plan, a mean consolidation pressure increment of 65 kN/m² was to be applied during the entire negative pressure period in all blocks; however, the actual mean consolidation pressure increment, as shown in Fig. 19 was 66.0 kN/m².

(2) Settlement management

The settlement curves for the central areas of blocks A and B are shown in Fig. 20. These were
based on the original forecast made in 2007; because there was no information about the settlement of the intermediate sandy layer, it was disregarded. The measured settlement occurring when negative pressure was applied after PVD placement was 1.3 m in block A and 1.6 m in block B. In the initial stages after negative pressure was applied, an instant settlement of approximately 0.5 m occurred followed by gradual settlement, reaching total settlement values of 3.40 m in block A and 4.04 m in block B by the time the negative pressure application was terminated. It is thought that the initial rapid settlement was consolidation settlement in the intermediate sandy layer.

Table 1 compares the calculated (initial) and actually measured settlement. Final settlement amount (measured) is obtained from measured settlement based on the hyperbola method. In the calculated (initial) results, the final settlement amount, $S_f$, was derived using the $C_e$ method defined in equation (3) and the settlement rate was derived using Barron’s approximate solution\(^{19}\), which can be derived from equation (4).

$$S_f = \frac{H}{1+e_0} \left\{ C_e \log \left( \frac{P_0 + \Delta P}{P_e} \right) \right\} \quad (3)$$

$$U_h = 1 - \exp \left( -8 \cdot \frac{T_s}{F(n)} \right) \quad (4)$$

$$F(n) = \frac{n^2}{n^2 - 1} \cdot \ln(n) - \frac{3n^2 - 1}{4n^2}$$

$$T_s = \frac{c_h \cdot t}{d_e} \quad n = \frac{d_e}{d_w}$$

Here, $H$ is the layer thickness, $P_0$ is the effective overburden pressure, $P_e$ is the consolidation yield stress and $\Delta P$ is the effective pressure increment (65 kN/m\(^2\)). Additionally, $U_h$ is the mean consolidation degree, $T_s$ is the time coefficient, $c_h$ is the consolidation coefficient in the horizontal direction ($c_h = c_v$), $d_e$ is the equivalent diameter (based on the 1.13 × 1.8 or 2.0 m square) and $d_w$ is the PVD equivalent diameter (8 cm).

It was thus theoretically predicted that the mean consolidation degree upon stopping the negative pressure would be 70% and 79% for block A and block B, respectively, which almost agree with the actual measurements of 71% and 80%, respectively. The final calculated (initial) and actual settlement amounts were 4.09 and 4.17 m, respectively, for block A, and, similarly, 4.43 and 4.54 m, respectively, for block B. The calculation (initial) values didn’t include the settlement of the sandy layer but still closely matched the actual measurement because the calculation for the final clay settlement amount included a margin of error. From the above results, with the PVD interval at 2.0 m (block A) and 1.8 m (block B), it is concluded that Barron’s solution can be applied to the consolidation rate for both blocks.

In the main volume reduction work, the PVD interval, $d$, was set to 1.8 m and the mean consolidation degree, $U$, was set to 90% because these conditions minimized the approximate project cost divided by the calculated settlement. Thus, the required negative pressure period for the main project was 310 days.

The settlement management flow for the main project period is shown in Fig. 21. The settlement management was carried out as follows. At first, in the center of each block, an RI cone penetration test was executed and the sandy layer and clay layer were classified based on the measured tip resistance, skin surface friction, pore water pressure and void ratio, $e_0$. In 2009, $e_0 \leq 1.6$ was judged to be the sandy layer, based on the fitting results of the settlement curve to the actual measurements. Next, using the obtained $e_0$ distribution, settlement calculations were performed to predict the time-settlement curve and the final settlement amount. For the clay layer, the settlement amount was calculated using the $C_e$ method and the settlement rate was calculated using the Carrillo’s method\(^{19}\) described in equation (5), which considers both the vertical drainage within the intermediate sandy layer based on the Terzaghi theory and the horizontal drainage based on the PVD based on Barron’s solution. Although the calculations in the test area...
were based only on Barron’s solution, Carrilo’s method was adopted for more logical reason for areas with many intermediate sandy layers.

\[ U(t) = 1 - \{1 - U_v(t) \} \{ 1 - U_h(t) \} \]  

(5)

Here, \( U(t) \) is the total mean consolidation degree and \( U_v(t) \) and \( U_h(t) \) are the mean consolidation degrees in the vertical and horizontal directions, respectively.

After obtaining the actual measurement settlement curve, the negative pressure stop period was determined based on the most recent measured settlement amount [%] being 90% or more of the final settlement value calculated using the \( C_c \) method or [2] being 90% or more of the final settlement amount derived from the hyperbolic approximation.

The results demonstrating the accuracy of the method in Fig. 21 in the test area blocks are shown in Fig. 22 including the calculated time-settlement curves for block A (\( d = 2.0 \) m) and block B (\( d = 1.8 \) m) . For the sandy layer, the settlement strain \( \Delta e = 6.7\% \) (see section 4.(3)) was used. The settlement of the original seabed clay layer occurs with the load as the self-weight of the dredged clay on it. When negative pressure is applied, settlement occurs in the dredged clay layer, the intermediate sandy layer within the dredged clay layer and the original seabed clay layer. The calculated total settlement curves agree well with the measured curves for both blocks A and B. Based on the calculated settlement amount in each layer when the negative pressure was stopped in block B (after 204 days), the settlement amount was 2.0 m of the dredged clay layer, 0.3 m of the intermediate sandy layer and 1.9 m of the original seabed clay layer, and the total settlement amount was 4.2 m. The calculated mean consolidation degree when the negative pressure was stopped was 85.3% with a total settlement amount, \( S_G \) of 4.78 m while the actual measurements were 82.7% consolidation and \( S_F = 4.94 \) m (derived from the hyperbolic approximation). When negative pressure was applied continuously for 310 days, the mean consolidation degree was confirmed to be 90% or more by the calculation using Barron’s approximate solution at the increase of the negative pressure loading period (increase to 310 days from 204 days).

For the main project period, however, some of the measured settlement amounts deviated from the calculated values. Therefore, the following adjustments were made to improve the accuracy of the total settlement calculations for the sandy layer and dredged clay layer. For the sandy layer, instead of assuming a uniform \( \Delta e \) value of 6.7%, as shown in Table 2, the settlement amount was calculated by assuming no compression for \( e_0 < 1.1 \), using equation (2) with \( e_f = 1.1 \) for \( 1.1 \leq e_0 < 1.4 \) and using the \( C_c \) method shown in equation (3) for \( 1.4 \leq e_0 < 1.6 \). The soils with 1.6 \( \leq e_0 \) were considered as the dredged clay layer. The results of \( C_c \) and \( C_e \) for the various test results are summarized in Fig. 23. Hence, the relationship in equation (6) shown in Fig. 23 can be used for \( e_0 \) and \( C_c \).

\[ C_c/(1+e_0) = 0.2 \]  

(6)
$C_c = 0.5$ corresponds to $e_0=1.5$ according to equation (6) as shown in Table 2. Additionally, soils with $1.4 \leq e_0< 1.6$ have greater permeability than clay and are therefore considered to be the drainage layer. Thus, rather than calculating settlement in the dredged clay layer based on a uniform $C_c$ value of 0.88, changes in the soil characteristics according to equation (6) were given.

Using the adjustments described above, the calculated mean settlement amount upon stopping the negative pressure reached $-9.6\%$ of the actual measured mean settlement amount in all 118 blocks. The calculated settlement amount remained lower than the measured value because the effect of the water level within the negative pressure pump at the disposal area (mean of approximately 1 m) was disregarded when estimating the increase in the consolidation load.

6. RESULTS OF VOLUME REDUCTION

In this section, the results of volume reduction to the project as a whole are discussed. The area in which volume reduction was carried out was 383,000 m² and the whole area was divided into 188 blocks, each approximately 3,200 m², the area that can be covered by the capacity of a single vacuum pump. A volume reduction was carried out using vacuum consolidation in each block. The settlement was measured in the center and in the surrounding area of each block to check the consolidation improvement result.

The first main volume reduction work in 2007 was conducted on 11 blocks to understand the effectiveness of consolidation improvements. For details, see reference documents 20)–22).

(1) Applied negative pressure

The mean consolidation pressure increment values due to negative pressure measured at the PVD tip in the center of each block were already shown as a frequency distribution in Fig. 19. The mean value for all blocks was 66.0 kN/m² with approximately 1.0 kN/m² below the target effective pressure increment of 65 kN/m². Based on the data, there were blocks in which the mean effective pressure increment didn’t reach 65 kN/m². It was noted that there was a thick intermediate sandy layer in these blocks.

(2) Settlement amount

The time-settlement curves for all blocks are shown in Fig. 24 where the x-axis shows the number of days elapsed since the PVD was placed and the application of negative pressure began, from 0 to 100 days. At the time of PVD placement (time zero), offset settlement was observed as a result of volume reduction carried out in neighboring blocks in the previous year (23). Based on the data shown in Fig. 24, although the settlement rate was similar across the whole area, the settlement amount upon stopping the negative pressure varied greatly. The settlement amount when the negative pressure was stopped satisfied the minimum requirement of 90% or more of the final settlement amount calculated by the $C_c$ method, but the settlement amount was 86.0% of the final settlement amount obtained using the hyperbola method as the overall average, falling short of the 90% goal.

The planar distribution of the settlement is shown in Fig. 25. According to this data, the settlement amount varied between blocks, and the reason was the varying thickness of the intermediate sandy layer. The frequency distribution of the settlement amount
is shown in Fig. 26. The settlement amount was within the range of 2.52–7.86 m with a mean of 5.13 m.

The relationship between the total thickness of the intermediate sandy layer and the settlement is shown in Fig. 27. As the thickness of the intermediate sandy layer increased, the settlement amount tended to decrease linearly.

(3) Amount for volume reduction

When the cap-type vacuum-consolidation method was applied to the clay soil, due to the isotropic application of negative pressure, settlement and horizontal displacement toward the targeted areas from non-improved areas is generated. As the result, the settlement on the ground surface forms a concave shape, including the regions outside the targeted areas\(^2\),\(^3\),\(^4\). In the technical manual for this method\(^2\), a simple method of calculating the settlement shape is proposed, as shown in Fig. 28. This approach assumes that displacement occurs within the distance equivalent to the improved depth, \(H\), from the edge of the improved area and settlement is reduced in the boundary area spanning from the target area to \(H/2\) from the target area.

In volume-reduction projects, the volume-reduction capacity is calculated as follows, as illustrated in Fig. 29:

1) Upon stopping the negative pressure, the settlement amount is measured in the center of the target area as well as at two additional points outside of the target area.

2) The volume-reduction capacity in the center of the improved area is calculated as the product of the improvement width, \(B\), and the settlement, \(S\). Normally, when the intermediate sandy layer is uneven, the settlement occurs in a non-uniform way and differential settlement is thought to occur\(^5\). Thus, it is preferable to calculate the settlement volume at any given time based on the surface measurements taken using methods such as sonic exploration. However, in this example, the dredged soil was continuously disposed of during the period for volume reduction, so due to the soil flowing into the improved areas, it was not possible to accurately measure the soil surface distribution of the settlement.

3) The volume reduction amount at the edge of the target area (shown as target area [3]) is calculated by assuming a linear distribution of settlement between two points outside of the improved area, with \(S_1\) being the settlement from the \(H/2\) points within the target area to the edge of the target area.

The settlement was measured at each of the two points outside the target area in order to accurately grasp the volume-reduction capacity outside the target area. For example, where the target area has settlement shape [1] (the broken blue line in Fig. 29), the volume reduction \(V_1\) is obtained by equation (7), and when it has settlement shape [2] (Navy blue
linked line in Fig. 29), the volume reduction $V_2$ is obtained by equation (7). In fact, the actual volume-reduction capacity outside the target area is closer to $V_2$ than to $V_1$ and the volume-reduction capacity for each year was approximately 9%–16% (average of 12%) of the total volume-reduction capacity. The volume-reduction capacity for the entire target area as a whole, assuming a plane strain state, was calculated by multiplying $V$ by the plane thickness.

$$V_1 = S_3 \times (B_3 - H_3/4) \quad (7)$$

$$V_2 = S_3 \times (B_3 + H_3/4)$$

To summarize the volume-reduction capacity for each year, the results of the above calculations are shown in Table 3. However, the rebound quantity was not measured and, thus, disregarded. In the original plan, the volume-reduction capacity was predicted to be 1,840,000 m$^3$, but the actual result was 2,167,000 m$^3$. Considering that the annual mean volume of dredged soil planned for landfill disposal in the New Waste Disposal Area (in the five years from 2012 to 2016) was approximately 942,000 m$^3$, a landfill disposal capacity for approximately 2.3 years was successfully secured, and in other words, the life of the disposal area was extended by 2.3 years.

(4) Cost versus benefit analysis

Dividing the cost required for volume reduction in one block by the volume-reduction capacity for that block yields the cost per space volume. The cost per block of space volume was in the range of 3,391–4,824 yen/m$^3$ and the mean cost from the 118 blocks was 3,967 yen/m$^3$.

### Table 3 Volume reduction amount of the ground obtained by vacuum consolidation.

| Work year | 2007      | 2008       | 2009       | 2010       | 2011       | 2012       | 2013       | 2014       | Total/Average |
|-----------|-----------|------------|------------|------------|------------|------------|------------|------------|---------------|
| Improvement area m$^3$ | 36503     | 26470      | 40151      | 35626      | 28840      | 26013      | 26578      | 25447      | 25447 34488 383035 |
| PVD Installation number | 11325     | 8171       | 12425      | 11025      | 8925       | 8050       | 8225       | 7875       | 7875 10675 118546 |
| Average settlement m | 4.36      | 4.35       | 5.55       | 4.66       | 4.68       | 5.36       | 5.78       | 4.77       | 4.83 6.40 5.67 |
| Volume reduction amount m$^3$ | 181243    | 134663     | 248980     | 186316     | 147596     | 154567     | 171169     | 137720     | 137525 10675 2167276 |

7. CONCLUSION

Volume reduction of clay soil was carried out at the New Waste Disposal Area, Block C. The work started with a trial period in 2005 and the main project work was in 2007–2015. The main geotechnical points learned through performing the volume-reduction project are summarized as follows.

1) The soil was a two-layer clay soil, with dredged soil disposed as landfill on top of an original seabed clay layer. While the original seabed clay layer soil was uniform, the dredged soil layer contained many intermediate sandy layers.

2) The volume-reduction project was carried out using a cap-type vacuum-consolidation method. In this approach, the surface 1 m of the clay layer, which had a permeability coefficient of approximately $10^{-7}$ m/s or less, was used as a vacuum-sealing layer. This proved effective in the project.

3) To prevent “dragging” due to neighboring ground settlement inside the revetment when performing consolidation work, the horizontal range of the volume reduction was limited to 50–100 m inside the revetment. To prevent the infiltration of the water from waste products in the future into the lower sand layer via the PVD, the lower edge of each PVD was positioned at a depth 3 m above the lower edge of the original seabed clay layer.

4) To reduce the number of PVDs placed, 150-mm-wide PVDs with thicknesses of 3.9 mm were used. The PVD placement interval and negative pressure loading period were determined according to conditions that would minimize the project cost. Dividing the approximate project cost for each placement interval by the calculated settlement amount, when the placement interval was 1.8 m (square arrangement) and the mean consolidation degree was 90%, the project cost became minimum. The negative pressure loading period required to reach the mean consolidation degree of 90% was 310 days.

5) The targeted consolidation pressure increment by applying vacuum pressure was 65 kN/m$^2$ on average. The consolidation pressure increment converted from the void water pressure
measured at the PVD tip was low at approximately 50 kN/m² when the negative pressure was applied. However, it gradually increased as the negative pressure loading period progressed, reaching between approximately 70 and 75 kN/m² after 310 days. The mean consolidation pressure increment for all blocks was 66.0 kN/m².

6) The measured settlement amount was in the range of 2.52–7.86 m and the mean settlement amount was 5.13 m.

7) The measured volume-reduction capacity was 2,167,000 m³, which was 11.8% above the initially predicted value of 1,840,000 m³. When considering that the annual mean amount of planned landfill disposal of dredged soil to this disposal area was 942,000 m³, the life of the disposal area was increased by 2.3 years.

8) The cost of the volume reduction work was 3,967 yen/m³.

As described above and as initially expected, this volume-reduction project brought about great benefits. Numbers shown in 6), 7), 8) in the conclusion are estimated values in April 2015. It is known that the settlement changes greatly according to the soil characteristics. In the original seabed clay layer, the layer thickness and consolidation characteristics, the degree of consolidation and the mass of the dredged soil disposed in the landfill impact the settlement. On the other hand, in the dredged soil layer, the layer thickness and consolidation characteristics, the existence of intermediate sandy layers and the degree of progress of the self-weight consolidation impact the settlement. If vacuum consolidation is adopted in future projects, a consolidation load equivalent to approximately 65 kN/m² should be applied, since higher negative pressures are difficult to attain. When investigating volume-reduction projects in soft ground, it is critical to carefully consider the above factors.

See Document 26) regarding additional details of the trial work, Document 27) regarding the work in 2007 and document 28) regarding the work in 2009.

REFERENCES
1) http://www.kouwan.metro.tokyo.jp (2017.9/6)
2) The Vacuum Consolidation Drain Method, Technical Report, Vacuum Consolidation Drain Method Association, 2011. (in Japanese)
3) Manual for the design, construction and management at a coastal disposal area: Waterfront Vitalization and Environment Research Center, pp. 1-20, 2010. (in Japanese)
4) Matsuyama, K., Shinsha, H. and Fujimori, S.: An example of the New Waste Disposal Area in Tokyo Port, Geotechnical Engineering Magazine, Division Lecture, Vol. 61, No. 7, pp. 53-60, 2013. (in Japanese)
5) The standard on the technology which affects final disposal of a final disposal place of non-industrial waste and an industrial waste, The Prime Minister’s Office and the Health and Welfare Ministry Joint Issue, 1998.6.16. (in Japanese)
6) Hatanaka, Y., Otsuka, Y., Shinsha, H. and Yamashita, T.: Consideration on securement of the water barrier base layer at the improvement a clayey ground by vacuum consolidation, Proc. of 44th Japan National Conference on Geotech. Eng., pp. 1933-1934, 2009. (in Japanese)
7) Takeya, K., Nagatsu, T. and Yamashita, T.: Trial tests for volume reduction of a clayey ground by vacuum consolidation, Part I, Proc. of 42th Japan National Conference on Geotech. Eng., pp. 917-918, 2007. (in Japanese)
8) Kado, T., Oskubo, Y., Nakagawa, D. and Taga, M.: Selection and execution of the vacuum consolidation method in the New Waste Disposal Area, Proc. of 44th Japan National Conference on Geotech. Eng., pp. 833-834, 2009. (in Japanese)
9) Nakagawa, D. and Hiroi, Y.: Overview of vacuum consolidation method and a drain placing ship called “VCD-Triton” used for life prolongation of the New Waste Disposal Area, Building Plan of Construction, pp. 36-41, 2008. (in Japanese)
10) Anti-Pollution Soil Transporter “Tenyu”, Technological pamphlet, 1999. (in Japanese)
11) Watabe, Y., Hatanaka, Y., Shinsha, H., Ko, C. and Kumagai, T.: Influence of the distribution of intermediate sand layers on consolidation settlement, Proc. of 46th Japan National Conference on Geotech. Eng., pp. 805-806, 2011. (in Japanese)
12) Watabe, Y., Naoi, T., Shinsha, H., Ko, C. and Shiraga, S.: Description of intermediate sand layers of dredged clay deposit using penetration resistance in installation of prefabricated vertical drains, The Symp. of JGS on the Closeness and Looseness of Soil Material and Ground Investigation, pp. 197-204, 2012.
13) Watabe, Y., Shinsha, H., Yoneya, H. and Ko, C.: Description of partial sandy layers of dredged clay deposit using penetration resistance in installation of prefabricated vertical drains, Soils and Foundations, Vol. 54, Issue 5, pp. 1006-1017, 2014.
14) Shinsha, H., Yoneya, H., Ko, C., Kumagai, T and Watabe, Y.: Evaluation of consolidation settlement of the clayey ground including partial sand layers, Journal of JGS, Vol. 71, No. 2, pp. 173-186, 2015. (in Japanese)
15) Wada, K., Shinsha, H., Ikeno, K. and Yoshimura, M.: RI cone penetration test for the ground improved by vacuum consolidation, Proc. of 43th Japan National Conference on Geotech. Eng., pp. 149-150, 2008. (in Japanese)
16) Takeya, K., Shinsha, H., Kumagai, T. and Miyamoto, K.: Trial tests for volume reduction of a clayey ground by vacuum consolidation, Part 4, Proc. of 43th Japan National Conference on Geotech. Eng., pp. 907-908, 2008. (in Japanese)
17) Miyakoshi, K., Shinsha, H. and Nakagawa, D.: Trial tests for volume reduction of a clayey ground by vacuum consolidation, Part 2, Proc. of 42th Japan National Conference on Geotech. Eng., pp. 919-920, 2007. (in Japanese)
18) Naoi, T., Shinsha, H., Hidaka, M. and Ko, C.: Consideration on suction volume of water from intermediate sand layers in a vacuum consolidation work, Proc. of 47th Japan National Conference on Geotech. Eng., pp. 875-876, 2012. (in Japanese)
19) Yoshikuni, H.: Design and Execution Management of the Vertical Drain Method, Gihodo Publication, pp. 37-40, 1979.
20) Kado, T., Shinsha, H., Yamashita, T. and Miyamoto, K.: Volume reduction work of a clayey ground in the New
21) Hatanaka, Y., Shinsha, H., Shiina, T. and Yamashita, T.: Volume reduction work of a clayey ground in the New Waste Disposal Area, Part 2, *Proc. of 44th Japan National Conference on Geotech. Eng.*, pp. 837-838, 2009. (in Japanese)

22) Wada, K., Shinsha, H., Yamashita, T. and Kumagai, T.: Volume reduction work of a clayey ground in the New Waste Disposal Area, Part 3, *64th Civil Society Annual Academic Lecture Outline Collection*, pp. 1009-1010, 2009. (in Japanese)

23) Naoi, T., Shinsha, H., Hidaka, M. and Nii, K.: On settlement of the neighborhood block induced by vacuum consolidation, *Proc. of 46th Japan National Conference on Geotech. Eng.*, pp. 795-796, 2011. (in Japanese)

24) Miyakoshi, K., Yamashita, T., Shinsha, H. and Shiina, T.: Trial tests for volume reduction of a clayey ground by vacuum consolidation, Part 3, *Proc. of 43th Japan National Conference on Geotech. Eng.*, pp. 905-906, 2008. (in Japanese)

25) Nakaoka, J., Yoneya, H., Nii, K. and Motonaga, H.: Application of the vacuum consolidation method to the under-consolidated ground reclaimed by dredged soil, Part 1, *Proc. of 40th Japan National Conference on Geotech. Eng.*, pp. 1053-1054, 2005. (in Japanese)

26) Tezuka, H., Takeya, K., Shinsha, H. and Yamashita, T.: Life prolongation measure for the New Waste Disposal Area, *Geotechnical Engineering Magazine*, Vol. 56, No. 9, pp. 14-17, 2008. (in Japanese)

27) Suzuki, K., Shinsha, H., Yamashita, T. and Shiina, T.: Volume reduction work of a clayey ground using the vacuum consolidation method for the New Waste Disposal Area, *54th Symp. of JGS*, pp. 551-556, 2009. (in Japanese)

28) Naoi, T., Watabe, Y., Shinsha, H., Hidaka, M. and Shiraga, S.: Life prolongation for the New Waste Disposal Area using the vacuum consolidation method, *Journal of JSCE*, Division B3, Vol. 68, No. 2, pp. 498-503, 2012. (in Japanese)

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