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THE SECOND PHASE CONSTRUCTION OF KANSAI INTERNATIONAL AIRPORT CONSIDERING THE LARGE AND LONG-TERM SETTLEMENT OF THE CLAY DEPOSITS

TERUAKI FURUDOI[i]

ABSTRACT

Kansai International Airport was planned to provide a fundamental solution to the aircraft noise pollution problem in the area surrounding Osaka International Airport (Itami Airport) and to the increasing demand for air transportation This man-made island was constructed 5 km offshore in Osaka bay to minimize noise pollution in residential areas. The airport commenced operations in September 1994. The second phase of construction work involved building an island further offshore than the island built in the first phase. Since the sea water is deep at the Kansai International Airport construction site and the layers below the seabed consist of a very soft layer of Holocene clay (immediately below the seabed surface) followed by alternate layers of Pleistocene clay and sand/gravel, the construction of an airport island was expected to produce a considerable amount of ground settlement. The amount of settlement during and after construction needed to be predicted in the design of the airport islands, and the results needed to be considered in the details of the land development work. This report outlines the second phase construction work at Kansai International Airport and describes the related geotechnical issues, with a particular emphasis on settlement.

Key words: Holocene clay, on-site monitoring, Pleistocene clay, pore water pressure, reclaimed ground, settlement, soil improvement (**IGC**: H5)

INTRODUCTION

The increase in demand for air transportation and resident's complaints about noise pollution surrounding Osaka International Airport (Itami Airport) were the two factors that gave impetus to the planning of Kansai International Airport. The airport on the man made island constructed 5 km offshore in Osaka bay as a man-made island (Fig. 1) opened in 1994 and was the first round-the-clock airport in Japan.

In the early nineties, the number of aircraft take-offs and landings at Kansai International Airport was close to handling capacity (30 per hour) during peak hours in the morning and in the early evening. Based on pressure from both home and abroad to increase its capacity and function as an international hub airport, the second phase of construction started in 1999. The second phase construction work involved the development of 545 ha of land at a site further offshore than the existing island, and the construction of a 4,000 m parallel runway and related facilities. Figure 2 shows an image of the airport after the completion of the second phase project. To reduce the financial burden on Kansai International Airport Co. Ltd. (KIAC) in the second phase of construction work, land development and superstructure construction was undertaken by different companies; the land development work was undertaken by Kansai International Airport Land Development Co., Ltd. (KALD) while the construction of the airport facilities, including the building of the new runway, was undertaken by KIAC. While doing the land development work toward the scheduled opening of the new runway, KALD took both harmony with the local environment and the functionality of the airport island into careful consideration. The new runway, the taxiway connecting the first and second islands, and related facilities were opened in August 2007. Figure 2 shows the parts which opened that year.

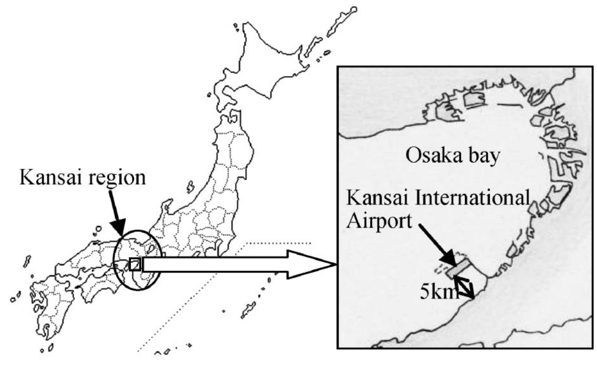

Fig. 1. Location of Kansai International Airport

[i] Kansai International Airport Land Development Co., Ltd., Osaka, Japan.
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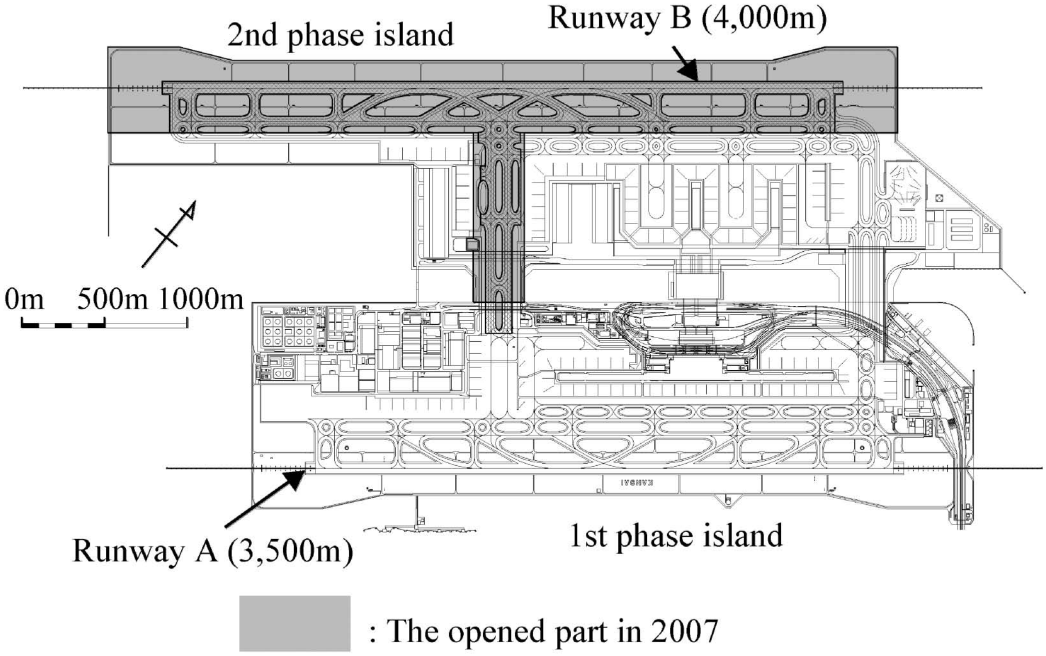


Fig. 2. Image of Kansai International Airport after completion of the second phase project

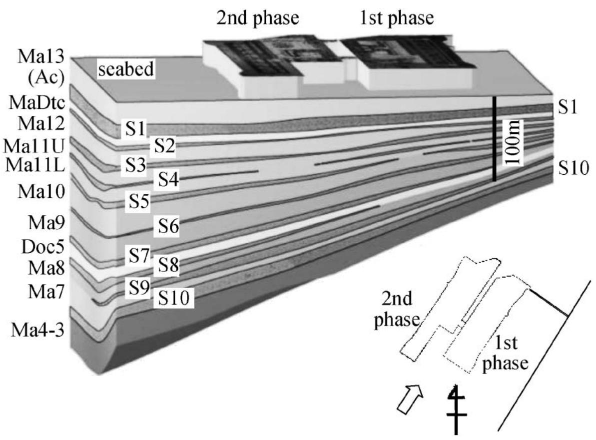


Fig. 3. General overview of seabed under airport islands

THE SECOND PHASE PROJECT OF KANSAI INTERNATIONAL AIRPORT

Geotechnical Conditions

The water depth at the construction site ranges from 18 m to 20 m. Figure 3 outlines the layers below the seabed of the construction site. The layers are formed in a similar manner throughout the site, with a monocline structure gentle inclining downward from the land side to the offshore side. The order and thickness of the layers are quite uniform along a line parallel to the shoreline. Immediately below the seabed, there is a soft layer of Holocene clay between 20 m and 25 m in thickness. The Holocene clay is mostly in normally consolidated condition with a natural water content (w_n) of 80% to 120% and unconfined compressive strength (q_u) of $4z$ (kPa), where z represents the depth from the seabed in meters. Immediately below this Holocene clay layer, there are alternate layers of Pleistocene clay, with an *OCR* of around 1.3, and sand/gravel.

Overview of the Second Phase Project

In the first phase of the project involved the reclamation of about 510 ha to construct the first airport island, complete with one 3,500 m runway, takeoff/landing facilities and terminal facilities. This first island was surrounded by an 11-km long seawall. About 180 million m^3 of soil was required for the construction of this island. The completion of large-scale reclamation work in deep waters and subsequent construction work of airport facilities within a little more than seven years was an extremely challenging and unprecedented task. It was possible thanks to various operational improvements and also innovations in marine technology.

The second phase construction was begun in July 1999 to complement the operation of the Kansai International Airport by adding an additional 4000 m runway and apron. This second phase project involved the construction of a larger man-made island of 545 ha off the existing airport island. Since the compressible clay layers were thicker at the second phase construction site than at the first one, the new second island was expected to settle more than the first island; the total amount of settlement of the second island was predicted to be about 18 m. Furthermore, deeper water and larger anticipated settlement required a thicker layer of reclamation soil. The average thickness of the reclamation layer amounted to more than 40 m and around 250 million m^3 of soil was required. Because of the deeper water and larger settlement, the second phase project was conducted under tougher conditions than those encountered in the first phase. Table 1 compares the first and second phase construction projects in terms of scale and natural condi-

Table 1. Comparison of scale and natural conditions between the first and the second phase construction

	Natural conditions		Scale		
	Water depth	Thickness of compressible clay	Reclamation		Seawall length
			Area	Volume	
1st phase	18 m	150–200 m	510 ha	180 Mm^3	11 km
2nd phase	20 m	250–300 m	545 ha	250 Mm^3	13 km

(M: million)

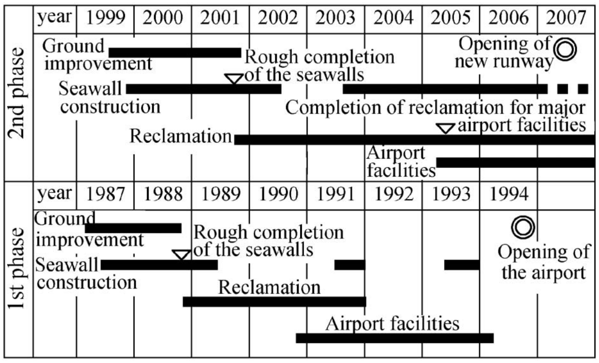


Fig. 4. Construction schedule of the second phase airport development

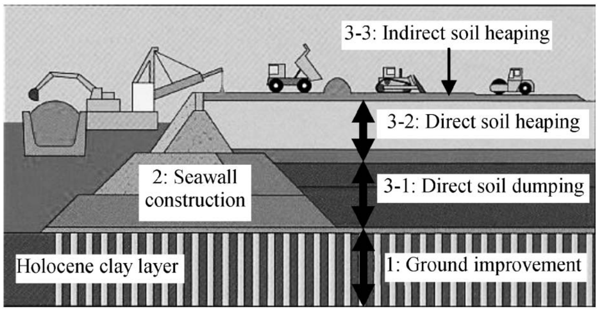


Fig. 5. Procedure of second phase construction

tions. In order to complete this larger project within a relatively short period under such conditions, the most advanced technologies were employed and combined with the experience accumulated during the first phase construction. Figure 4 compares the first and second phase construction schedules. The land development work was divided into the following phases which were carried out in this order: ground improvement, seawall construction and reclamation. The reclamation work consists of three stages: (1) soil dumping by hopper barges, (2) direct soil heaping by reclaimer barges and (3) indirect soil heaping by reclaimer barges, followed by soil spreading by bulldozers and ground compaction by vibration rollers. Figure 5 shows the procedure of the second phase of construction work.

The second phase of construction work proceeded smoothly and was completed on schedule. Figure 6 provides an overview of the first and second airport islands.

Fig. 6. Overview of the first and second airport island of KIA (Left: 1st island, Right: 2nd island)

Almost all of the second island had been reclaimed by March 2009.

Geotechnical Issues on Airport Island Development

Due to the thick clay layers under the seabed of the second phase construction site, sizeable settlement was expected to take place under the large load of reclamation soils. At the time, an estimated 12 m of settlement was expected in the period from the beginning of construction to the opening of the runway. An extra 6 m of residual settlement was predicted over a 50-year period after the opening of the runway.

The subsoil Holocene clay layer needed to be improved prior to construction of the island in order to minimalize the amount of residual settlement due to the thick compressive nature of this layer. It was also necessary to help attain the stability of the reclaimed soils during the construction work since this layer was immediately below the dumped and heaped soils and uneven settlement would have affected the ground surface. If the Holocene clay layer had not been improved, it would have been impossible to complete the airport construction within the given time frame; the reclamation work would have required longer intervals to allow the soil to consolidate. The sand drain method was employed to make it possible to complete the settlement of this layer and to obtain a stable seabed during the construction period. The amount of settlement of the Holocene clay layer due to the sand drain method is shown in Fig. 7, as, measured by a settlement plate (refer to Fig. 22) at a pre-monitoring point at the first island (refer to Fig. 8). Settlement was largely completed before the beginning of construction work.

Because the Pleistocene clay layers are deeper under the seabed, from a technical perspective, it is difficult to improve these layers. The settlement these layers, therefore, was expected to continue even after the opening of the runway. To maintain the operability of the airport island, it was necessary to account for the long-term settlement of the Pleistocene clay layers and the associated problems of uneven settlement. The long-term settlement predictions were incorporated into the design of the airport island, which had 50 years of airport function as a basis. The reclamation was carefully carried out to minimize

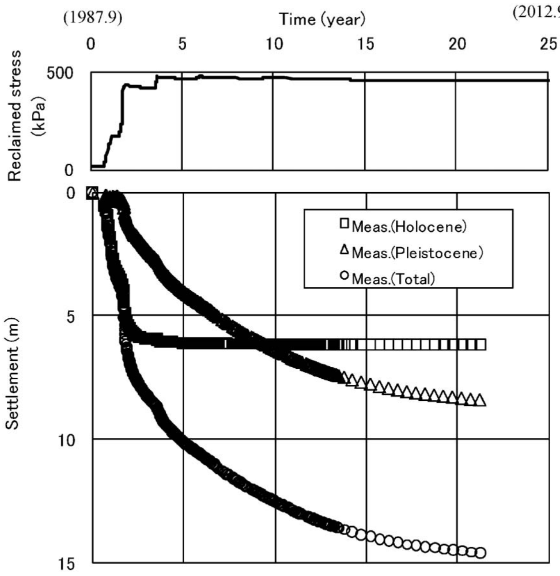


Fig. 7. Settlement observed at Holocene and Pleistocene clay layers at pre-monitoring point in the first island

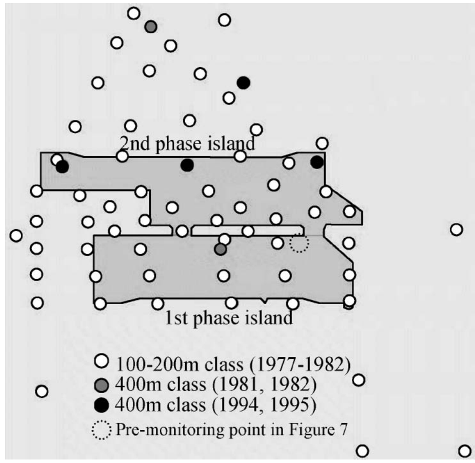


Fig. 8. Exploratory boring map of Kansai International Airport site

uneven settlement and to ensure the thickness of reclaimed soils by properly managing the reclamation work.

PREDICTING THE SETTLEMENT

Predictions about the long-term settlement of the Holocene layer and Pleistocene layers below the seabed were made, and these predictions were used to determine the total thickness of the reclamation layers comprising the airport island. The sea water level was also taken into consideration.

Settlement Prediction

In order to predict the amount of settlement, the geotechnical properties of the Pleistocene clay properties and layer configurations were investigated prior to the beginning of the first phase project. A total of 65 boring explorations were done, two of which were at the −400 m level, and the others at the −100 to −200 m level. Before the second phase of construction, an additional four −400 m-level boring explorations were carried out. Figure 8 shows the locations of the boring explorations. The wire line drilling method developed by the Port and Airport Research Institute (former Port and Harbour Research Institute) was used to ensure high quality and efficiency of boring explorations (Horie et al., 1984). In this soil exploration program, a series of laboratory tests and geological tests were carried out on all of the obtained soil samples. A database containing the soil profile and soil properties of layers of the construction site useful for settlement prediction was compiled based on the results of boring and geological studies. The nannofossil emergence patterns and the presence of volcanic glass were also taken into consideration (Kobayashi et al., 2005; Takemura et al., 2005).

The prediction of the long-term settlement at the second phase construction site was based on a one-dimensional consolidation analysis with the boundary conditions of pore water pressures in Pleistocene sand layers, which were calculated separately by a two-dimensional seepage flow analysis. The constitutive model, called the DB model, was developed from the results of various laboratory tests and observations of ground behavior at the first airport island, and takes such factors as settlement and pore water pressure into account. In this model, the e-$\log p$ curve was simulated by two straight lines (one for the overconsolidation region, and the other for the normal consolidation region). The results from the consolidation tests indicated a slight overconsolidation (Shinohara, 2003). The difference between the consolidation yield stress and the initial effective overburden pressure p_0 of the Pleistocene layers was assumed to be attributable to cementation. Furthermore, data obtained during the construction of the first airport island showed the settlement of deeper Pleistocene layers, which indicated the existence of a secondary consolidation. Therefore, a viscous settlement model with a secondary consolidation coefficient was assumed for predicting the settlement of deeper Pleistocene layers.

The validity of this prediction method using the DB model was confirmed by applying this method on the settlement observed during more than 10 years at the first airport island (Kobayashi et al., 2005). Its validity was reconfirmed in reference to the second island settlement data with cooperation from academic experts of an investigation committee for geotechnical issues for the second airport island. An approximate 8 m of settlement is predicted for the Holocene layer on average and approximately 10 m is predicted for the Pleistocene layers from the beginning of construction to 50 years after the opening of the new runway. A comparison of the measured and predicted settlements is shown in Fig. 9. The settlement was measured by hydraulic pressure gauges with magnetic transmitters. Details of these measuring devices will be described later. The predicted settlement and the measured settlement are in good agreement.

In order to ensure accurate settlement prediction,

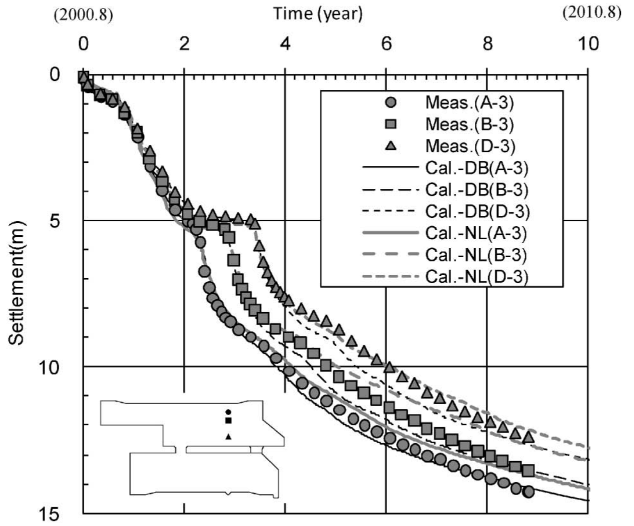


Fig. 9 Comparison of measured and calculated settlement (total settlement) (DB: DB model, NL: Nonlinear model)

KALD and Kobayashi (Kobayashi et al., 2005) are now developing a new method (called the nonlinear model) for long-term settlement prediction. The settlement predicted by the nonlinear model is also shown in Fig. 9. The prediction closely matches the actual settlement measured so far.

KALD is still studying the long-term consolidation in cooperation with the Port and Airport Research Institute using a new constitutive model (Watabe et al., 2008), which is capable of taking the strain rate effect into account (Šuklje, 1957; Leroueil et al., 1985).

Settlement Measurement

In making the most accurate long-term predictions of the settlement of Pleistocene layers, the amount of gradual settlement and the pore water pressure in each layer need to be monitored precisely. Offshore oil-rig platforms (*see* Fig. 10) were built at two locations for the installation of various measuring devices in the Pleistocene layers to depths of 350 m, The devices installed include anchor-rod type settlement gauges, magnetic element type differential settlement gauges, and air-balance type pore water pressure gauges. Figure 11 shows how the settlement gauges were installed at monitoring point 1 (offshore side).

Each anchor-rod type settlement gauge has its rod positioned towards the upper end of the target Pleistocene sand layers. The gauge measures the settlement of layers beneath the point where it is installed. Settlement is monitored from the projection at the upper end of the rod. Since the rod is quite long, the measurement of the inclination is compensated for by an inclinometer.

A magnetic element type multi-layer settlement gauge was installed in each of the Pleistocene clay layers that is compressed over time. In thicker clay layers, two or more magnetic elements were placed at different levels. By measuring the level of magnetized element(s) in each layer, changes in the thickness of each layer can be obtained

Fig. 10. Platform set up at monitoring point of behavior of Pleistocene layers

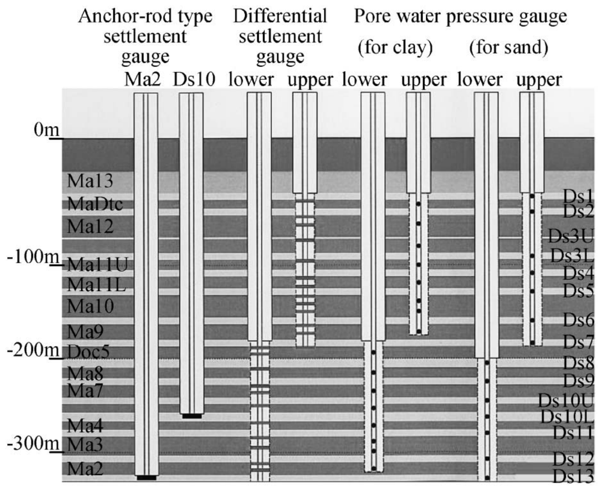


Fig. 11. Conceptual image of installation of measurement apparatuses at monitoring point 1 of behavior of Pleistocene layers

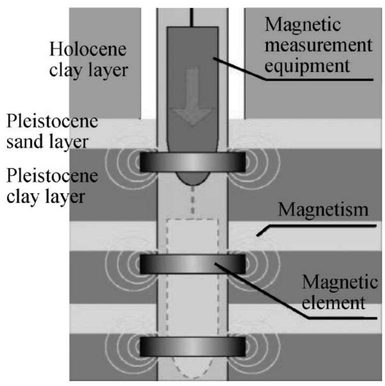


Fig. 12. Mechanism of differential settlement gauge

(Fig. 12). Data obtained from these devices are cross-checked with data obtained from the anchor rod type settlement gauges.

Each air-balance type pore water pressure gauge meas-

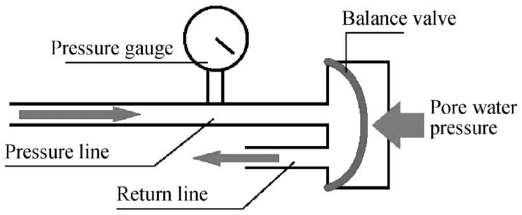


Fig. 13. Mechanism of pore water pressure gauge

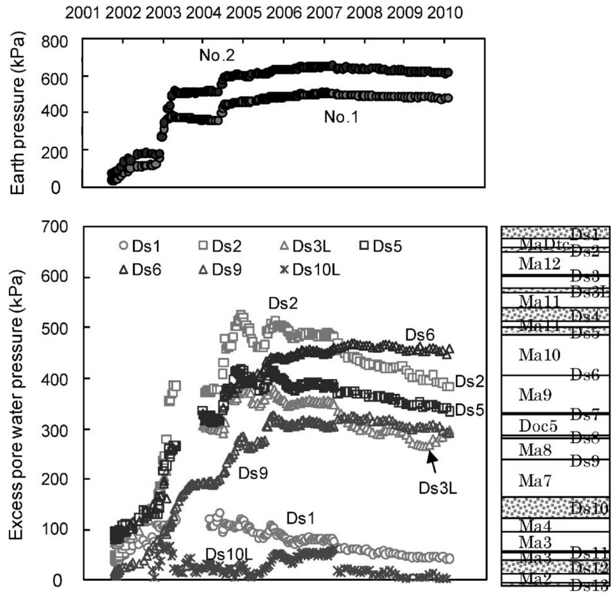


Fig. 14. Excess pore water pressure of Pleistocene sand layers observed at monitoring point 1

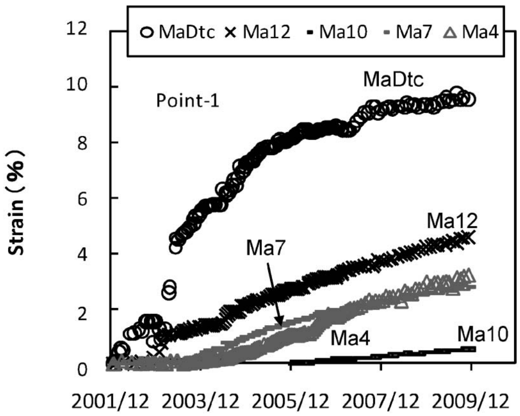


Fig. 15. Strain change of each layer at monitoring point 1

ures the pore water pressure by detecting the pressure value of the gas within the instrument and balancing it with the pressure of the water surrounding the instrument (Fig. 13). A pore water pressure gauge was placed in each of the Pleistocene clay and sand layers to enable the progress of consolidation in the clay layers and the permeability (drainage) of sand layers to be monitored. The changes of excess pore water pressure in the Pleistocene sand layers at monitoring point 2 are shown in Fig. 14. The earth pressure was also measured on the upper plane of the sand blanket at point 2 by an earth pressure gauge with a diameter of 1000 mm. The results show that not all sand layers function as permeable layers.

Figure 15 shows the change in the strain of each layer with time. The tendency varied with each layer. Settlement in the MaDtc near the permeable layers Ds1 appears to have almost completed. On the other hand, in Ma10 far from permeable layers there is a small strain.

OVERVIEW OF THE LAND DEVELOPMENT WORK

Ground Improvement Work

In the second phase project, as in the first phase, ground improvement by the sand drain method was applied to most parts of the seawall construction area and all of the reclamation area. In the beginning, a sand layer approximately 1.5 m-thick was spread evenly over the seabed, blanketing the entire construction site as a permeable layer for drained water. Once the sand blanket was laid, sand piles were driven into the seabed. Using eight large sand piling barges, each capable of driving 12 sand piles at a time, about 1.2 million sand piles of 40 cm diameter were driven into the Holocene clay layer, each with an average thickness of approximately 25 m. The spacing of the sand piles was set at 2.5 m × 2.5 m on a square grid, with which the settlement of the Holocene layer was expected to be completed during the construction period. In the area beneath the seawall, however, spacing was set at 1.6 m × 2.5 m rectangular grids to achieve the early strengthening of Holocene clay with accelerated consolidation. The spacing was decided after considering the seawall construction procedures.

Based on the experience from the first phase, we decided to drive the sand piles until they reached a sand layer below the Holocene layer so as to ensure effective drainage through it. We carefully controlled the sand pile driving operation to make sure that the pile driver shafts reached the sand layer. This was confirmed by monitoring the penetration resistance as estimated from the penetration speed or load. The criterion of the penetration resistance used by each sand piling barge was determined by penetration tests carried out at sites where the Holocene clay layer thickness had been obtained by boring. Since each sand pile was very long and thin, extending a length of 20 m or more, we prevented its discontinuity by vibrating the casing pipe using a vibro-hammer. Sand piles were driven into prescribed points by linking the position control systems of sand piling barges with the RTK-GPS.

Sea sands with a fine-grain fraction content of less than 10% were used for both the sand blanket and sand piles to facilitate drainage. As discussed above, the settlement of the second-phase airport island is progressing as predicted, which suggests that the ground improvement was successfully carried out.

Seawall Construction Work

A seawall surrounds the 13 km long perimeter of the

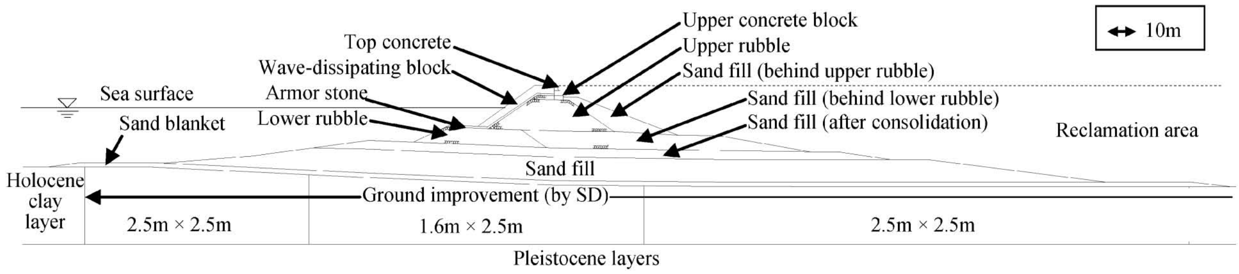


Fig. 16. Typical cross section of the rubble mound type seawall (with wave-dissipating block)

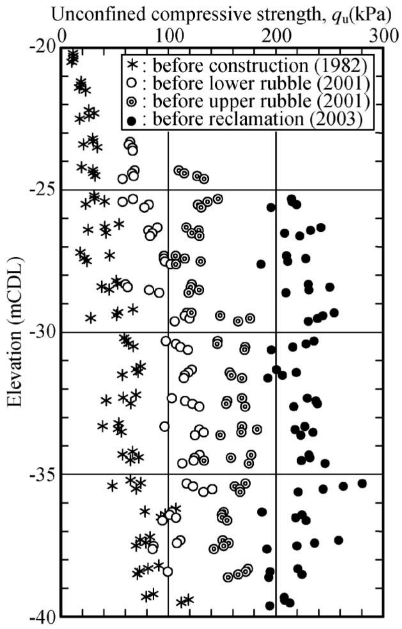


Fig. 17. Change of strength of ground improved by sand drain

second airport island. An embankment of about 35 m in height and about 250–300 m in base width, the seawall was constructed prior to reclamation to protect the reclaimed land from waves and to prevent turbidity caused by dumping.

As in the first phase, in the second phase more than 90% of the seawall was constructed as a rubble-mound seawall with gentle slope. This type of construction offers structural flexibility to the uneven settlement of the seabed, relatively low construction costs and is environmentally friendly.

Figure 16 shows a typical cross section of the rubble-mound seawall with gentle slope. This seawall was constructed by placing sand and stones, layer upon layer, to form a structural embankment with a gently sloping surface. In areas facing the open sea, wave dissipating blocks were placed in the front to protect the seawalls against larger waves, and to reduce wave overtopping onto the land and the reflection of waves to secure the safe navigation of small fishing boats.

The seawall was built in the early stage of the construction period after the ground improvement work. To achieve stability in the seawall during the construction stage, it was necessary to verify the increases in strength of the seabed as construction progressed. During seawall construction, a consolidation period of four months was allotted between two sand fill operations (one following another after consolidation), and also between lower rubble placement and upper rubble placement (*see* Fig. 15). Check borings were drilled at the end of each consolidation period, and unconfined compression tests were conducted using specimens from the boreholes (Fig. 17). The next stage was commenced only when it had been verified that the strength of the Holocene clay layer had sufficiently increased for the next stage. Depending on the check boring results, the work procedure and/or loading configuration were modified as required. Check borings were carried out for 19 sections of the seawalls.

Reclamation Work

During the reclamation work, the thickness of reclaimed soils was carefully managed to ensure the determined thickness. As an additional effort to minimize the uneven settlement that may occur after the completion of the airport island, the reclamation work was conducted as evenly as possible to minimize differences in the load history. Furthermore, information pertaining to the layer thickness and load history during reclamation was stored for later use for improving the accuracy of settlement prediction. The following subsections describe different stages of the reclamation work.

Direct Soil Dumping

As the first stage of reclamation work, the direct soil dumping operation by hopper barges into each section of the reclamation area was carried out until the layers of dumped soil came up to a depth of about 6 m below sea level. The 6 m-depth limit allows for the draft of pusher barges. The load produced from the layers of dumped soil caused consolidation and an increase in the strength of the improved seabed. Precise settlement measurements and bathymetric surveys were carried out to confirm the predicted settlement and thickness of direct dumping under the sand drain method. To confirm the uniformity of layer thickness, the entire reclamation area was divided into blocks of 200 m × 200 m, and strict site management and measurement were carried out on each block. The soil dumping work was carried out in such a way as to produce eight layers. The lower layers were designed to be thinner than the upper layers so as not to disturb the improved seabed. Furthermore, soil dumping was conduct-

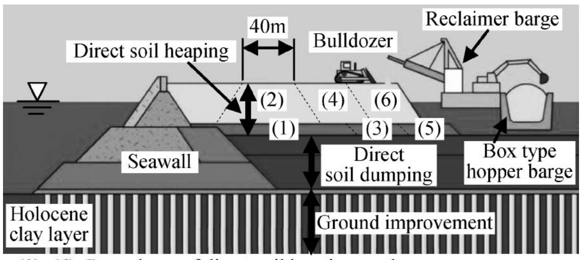


Fig. 18. Procedures of the soil dumping and heaping

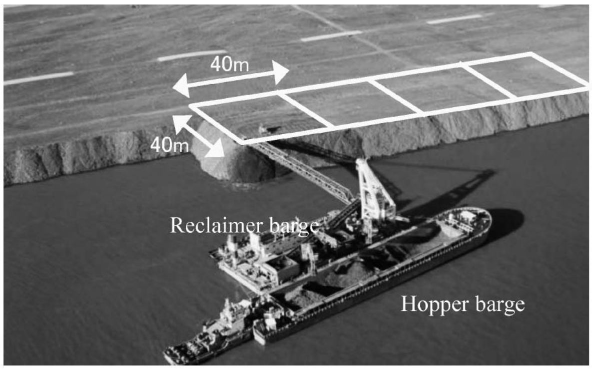


Fig. 19. Direct soil heaping

ed in a manner that would reduce differences in the load history among adjacent blocks. The direct soil dumping work produced a reclamation layer with the thickness of about 14–15 m.

Direct Soil Heaping

Since the hopper barges could not navigate in areas that are shallower than 6 m, the continuation of soil dumping operation in this uniform manner had to be terminated at this depth. Subsequently, therefore, direct soil heaping was conducted, which meant heaping soil up to a certain height that allowed for stability in the reclamation layer. To reduce the cost, a further direct soil dumping operation using smaller hopper barges was carried out until the dumped soil came up to a depth of 3 m below sea level, which was the depth limit for the draft of reclaimer barges. Then, the direct heaping operation was carried out on the ground reclaimed up to a depth of 3 m. Soils were transshipped using large backhoes from box type sand barges into hoppers of reclaimer barges, and then transported to a prescribed area by means of spreader machinery equipped with belt conveyers. Figure 18 shows the procedures of the soil dumping and heaping work typical in the reclamation process. In soil heaping work, the site management blocks of 200 m × 200 m were further divided into smaller management blocks of 40 m × 40 m in consideration of the length of the spreader installed on the reclamation barges (Fig. 19). Information pertaining to the layer thickness and load history was gathered in units of these smaller blocks. The direct soil dumping operation by small hopper barges and the soil heaping operation by reclaimer barges were conducted one after another for each 40 m × 40 m block.

Direct soil dumping by small hopper barges and soil heaping by reclaimer barges increased the reclamation layer thickness by 17–19 m. This thickness was determined in consideration of the lift capability of reclaimer barges and of the required stability of the improved subsoil. For the prescribed layer thickness to be achieved, the crest height of the reclamation ground was determined taking into consideration the increase of strength up to the period of soil heaping operation, with reference to the load history for each 40 m × 40 m block. Since the heaped soil produced a large load, load history differences in the soil heaping operation caused uneven settlement. Thus, the soil heaping operation was started at sites for key facilities, such as the runway and access taxiway, and this initial stage of soil heaping was conducted intensively by eleven reclaimer barges in these areas in order to minimize the difference of load history among blocks adjacent to each other.

Since the increments of the load produced by soil heaping were larger than increments in earlier work stages, it was necessary to ensure that the strength of the subsoil was stable enough to allow the soil heaping work to begin. The direct soil dumping by small hopper barges was carried out for one block ahead of soil heaping by reclaimer barges, as shown in Fig. 17. The dumped soil acted as a counterweight to the heaped soil. Check borings were carried out to confirm soil strength before the soil heaping. Stability analyses were carried out on each 40 m × 40 m block.

Indirect Soil Heaping

In the process of reclamation work, direct soil heaping was continued until the emerged ground reached a height limit that was determined in consideration of the ground stability and the lift capacity of reclaimer barges. Indirect heaping began after the direct heaping. The indirect heaping work consists of heaping soils from reclaimer barges along seawalls, transporting them to work areas by large dump trucks, spreading them by bulldozers and compacting them by heavy vibration rollers. Since the top layer of the reclamation soil serves as the foundation for airport facilities, it had to be improved. In the first airport island project, improvement of the ground by the SCP method, weight-dropping method, and MVT (Mammoth Vibro-Tamper) method were carried out after indirect heaping. For the second island, estimated settlement was very large (18 m), and so the thickness of indirect heaping was larger than that of the first island. Ground improvement by heavy vibration rollers was performed in parallel with transport and spreading work by dumps and bulldozers, which was the optimum method in view of the improvement of soils, cost and work efficiency. This ground compaction work in the final stage of land development was commenced on the runway and the access taxiway of the second phase construction site.

The ground compaction work was carried out by spreading and compacting soil, layer upon layer. The soil for reclamation included gravels, the maximum size of which was as large as 300 mm in diameter. To ensure effective compaction of this type of soil, test works were carried out to determine the effective work process. Tests included radioisotope measurements of soil density, in-situ soil density measurement by water replacement, and the plate-loading test. The ground density, water content, sub-grade reaction modulus and elastic modulus obtained through these tests were used to confirm the effectiveness of compacting the ground with vibration rollers. The test data were referred to when determining the layer thickness, types of heavy machineries to be used, the number of rolling operations, and the site management methodology for quality control. Specifically, each compacted layer was set as 60 cm-thick, with each layer compacted by vibration rollers eight times before adding another layer.

In order to give each layer exactly the prescribed thickness and stiffness of soils to serve sufficiently as the foundation, the position and elevation of the soil-spreading bulldozers and soil-compacting vibration rollers were controlled with GPS equipment installed on them (Fig. 20). Furthermore, in order to develop a stiff and highly uniform ground, site management focused on the careful management of the water content and the ground stiffness. The water content of soil transported from excavation sites was measured and controlled as follows: if the water content was too low, water was added from reclamation barges when heaping the soil at storage locations; if the water content was too high, the soil was exposed to air.

After compacting the ground by vibration rollers, the density and water content of the soils were measured using a radioisotope device on each 40 m × 40 m block. Figures 21(a) and (b) show the measured dry density and water content of the final layer produced by the compaction work. While the process control criterion for the dry density was set to 90% of the maximum dry density, all measurements exceeded this level, suggesting that the ground was well compacted. In addition, the stiffness distribution of surface layers was monitored using accelerometers installed on vibration rollers. Figure 21(c) shows the relative frequency distribution of the acceleration-based stiffness measured on 2 m-size meshes. Even though the measurements show a large distribution between 40 and 60 MN/m^2, this is attributable to differences or variations in the water content, grain size and the stiffness of lower layers.

The reclamation work on sites for facilities scheduled to open in 2007 was completed in the summer of 2005. For the remaining sites, ground compaction work was almost completed in March 2009.

Fig. 20. Indirect soil heaping

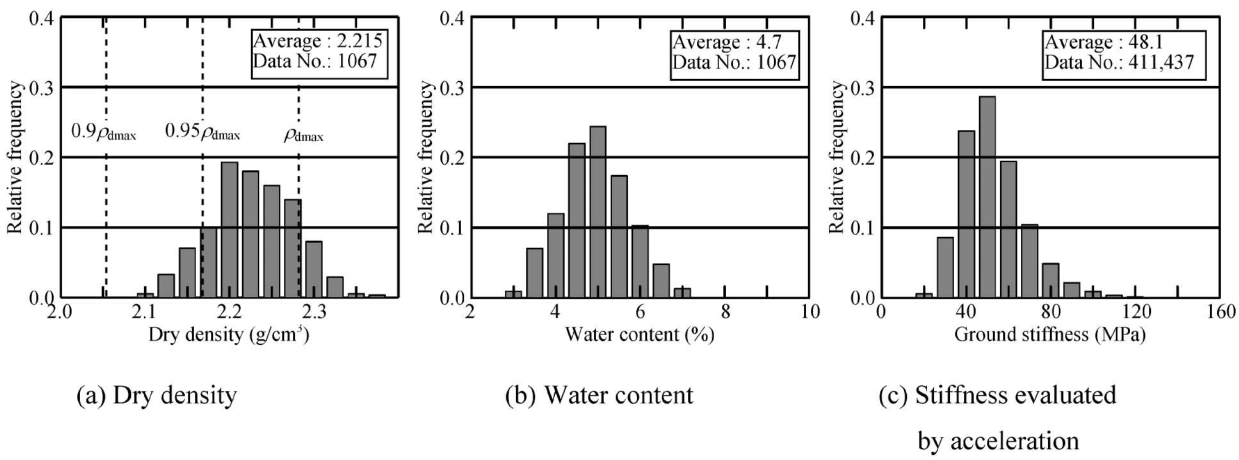
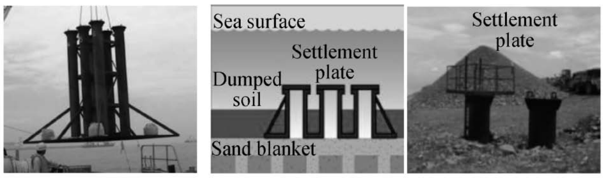


Fig. 21. Quality control in indirect soil heaping



(a) Settlement plate (b) During dumping work (c) After heaping work

Fig. 22. Typical settlement plate

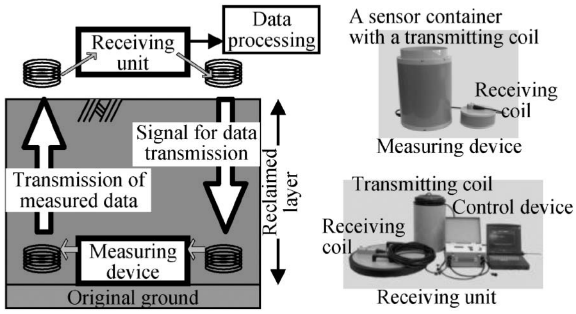


Fig. 23. Hydraulic pressure gauge with a magnetic transmitter

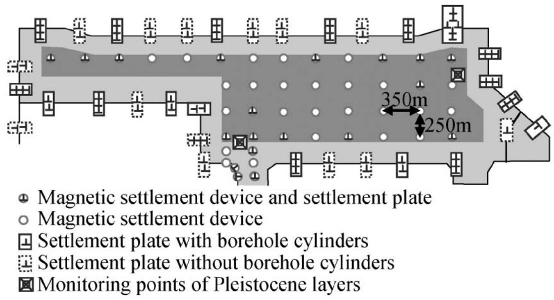


Fig. 24. Arrangement plan of settlement measuring devices

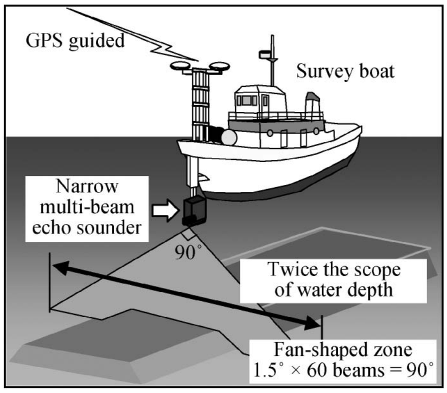


Fig. 25. Conceptual scheme of bathymetric survey

NEW TECHNOLOGIES FOR CONSTRUCTION MANAGEMENT

When conducting large-scale, rapid reclamation work on soft ground, it is important that the settlement of the seabed during the construction period is monitored accurately. Two types of settlement measuring devices—settlement plates and hydraulic pressure gauges with magnetic transmitters—were installed at many points on the second phase construction site.

The settlement plate is one of the most basic settlement measuring devices. Figure 22 shows a typical settlement plate. It consists of a bottom plate and an upright cylinder. Settlement plates are placed on the sand blanket immediately following ground improvement. The settlement of the seabed is determined through the water depth (or height) on the top of the cylinder. The settlement plates, besides being used for measuring the settlement, served as a guide for check borings. Settlement plates were used extensively in the past, but have some disadvantages; for example, they were obstacles to the navigation of hopper barges and other work vessels.

In view of these disadvantages of settlement plates, we made extensive use of hydraulic pressure gauges with magnetic transmitters (referred to here as *magnetic settlement devices*). These devices automatically measure the water pressure at two-hour intervals and transmit data via a magnetic transmitter to a receiving unit (Fig. 23). This method has several advantages: the device can be placed anywhere because it is not an obstacle to work vessels; and the device can measure settlement continuously because it stores data in its internal memory. Since the magnetic settlement devices had not been used extensively in a large scale project before, the functions and applicability of the device were confirmed in the seawall area, and then the device was used on the seabed of the reclamation area. To monitor the distribution of settlement of the second island, the magnetic settlement devices were placed in a rectangular grid of 350 m × 250 m (Fig. 24).

One of the technical challenges for the second phase construction was to conduct land reclamation as evenly and uniformly as possible in order to minimize any uneven settlement that might occur both during and after construction. The spatial distribution of the thickness of dumped soils was obtained in order to estimate settlement with more accuracy. In order to achieve the evenness and uniformity of the reclamation, advanced technologies superior to those employed during the first phase construction, such as GPS, a narrow multi-beam echo sounder and the earlier-mentioned magnetic settlement devices, were used extensively in the second phase construction.

The monitoring of the layer of dumped soils was carried out by a narrow multi-beam echo sounder before and after dumping operation by a hopper barge. This sounder simultaneously emits 60 beams of ultrasonic waves with an acute directive angle of 1.5°, like a fan with an opening angle of 90°, and conducts a bathymetric survey in a plane, as shown in Fig. 25 (in the past, only a linear survey was performed). The positions were determined using the GPS: compensation for the pitching and rolling motions of ships was possible.

The information database stores the shape patterns of the soil dumped on the seabed at various depths from

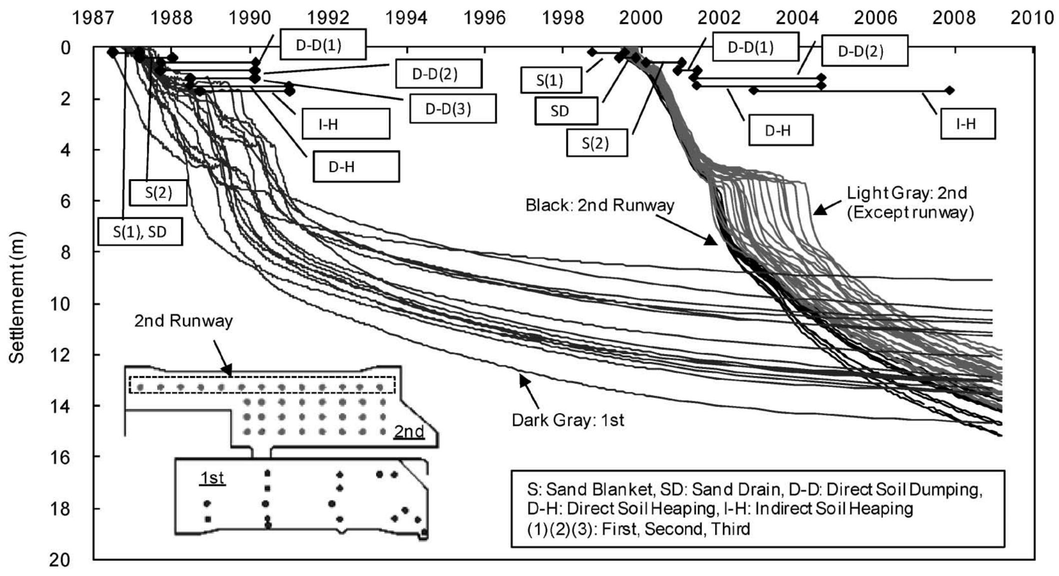


Fig. 26. Settlement of 1st and 2nd islands

each of the hopper barges, as well as the results from pre- and after-dumping bathymetric surveys. It became possible to predict the configuration of dumped soils through preliminary simulation using the database, which helped to achieve the evenness and uniformity of dumped soils by setting position of dumping in advance.

The optimum dumping location, the water depth, the direction of the bow and so on could be determined through preliminary dumping simulations. Each hopper barge was led to designated dumping positions by a GPS-based navigation management support system.

PRESENT CONDITION OF SETTLEMENT

As mentioned earlier, the construction of the 2nd island was carried out after making highly accurate predictions of settlement and by the careful management of layer thickness, and the production of a strong surface layer. In this section, we introduce the present condition of settlement by comparing the first phase island with the second phase island.

Figure 26 shows the change in settlement of the first and second phase islands with time. The settlement in the first island much varied with the location. On the other hand, the settlement in the second phase island showed almost same tendency regardless of location, except where direct soil heaping had taken place. This indicates that the construction of the second phase island resulted in very little difference in the construction history. In addition, it is important that the peak volume of soil was transported to avoid the discontinuity of the soil supply from the initial stage of construction.

Black line in Fig. 26 shows the settlement around the second runway. The construction of the second runway was carried out to avoid differences in the construction history. Therefore, the performance of settlement was almost the same. The difference of settlement after direct soil heaping was attributable to the difference of the thickness and the geological properties.

SUMMARY

The construction of the second phase island was carried out based on a precise construction plan for the management of the settlement of the seabed, the stability of the ground and reclamation thickness, considering the experiences accumulated during the first phase construction. As a result of the management, settlement has been observed to be within the predicted values, and the construction of the land development except the superstructure was completed smoothy, without problems, which is believed to contribute to advance of issues related to reclamation on soft deposit in future.

The second phase construction of the land development is almost completed. However, we will be connected with the settlement for a long time. We consider it responsibility to accumulate the observational data and to solve the unknown and academic problems in Kansai International Airport.

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