Construction of D-Runway at Tokyo International Airport

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ABSTRACT

In August 2010, the fourth runway called D-Runway (2,500-m long) was completed off the shore of the Tokyo International Airport (Haneda Airport), which is the fourth largest airport in the world in terms of passengers handled. Since the D-Runway is situated at the mouth of the Tamagawa River, a hybrid structure combining a piled-elevated platform and land reclamation was planned so that it would not obstruct the flow of the river. In the reclamation area, a slope-type rubble mound seawall was constructed on the soft clay seabed, which was improved using the sand compaction pile method, with a total of 52 million cubic meters of soil, sand and rock used in the land reclamation. The piled-elevated platform is a massive 520,000-m² structure in which 198 prefabricated steel jackets supported by 1,165 steel-pipe piles were connected together by welding on site. This project required huge quantities of construction materials, including 470,000 tons of steel and 450,000 m³ of concrete. The D-Runway was successfully constructed in just three and a half years thanks to the rapid construction method and a continuous 24-hours-a-day work schedule. This paper outlines the D-Runway project and its main structures.

Keywords: Haneda Airport, runway, hybrid structure, reclamation, steel jacket

1 INTRODUCTION

Tokyo International Airport (Haneda Airport) with four runways is one of the world’s leading international airports following Atlanta (the United States), Beijing (China) and London (the United Kingdom), handling 68 million passengers per year. A Re-expansion Project for Haneda Airport to increase its takeoff/landing capacity was carried out to build a fourth runway and a new passenger terminal for international flights. This expansion project was designed to increase the capacity for takeoffs and landings from the previous limit of 296,000 to 407,000, and was expected to further enhance the convenience of the airport.

Existing offshore airports in Japan, such as Kansai International Airport and Chubu International Airport, were all constructed on reclaimed land, however the D-Runway, as it is located partly across the mouth of the Tamagawa River, was built as a combination of land reclamation and a piled-elevated platform so as not to obstruct the flow of the river (Fig. 1.). Furthermore, the D-Runway is located under the obstacle limitation surfaces of A- and C-Runways that are in service. Consequently, structures and construction methods applicable to construction work at nighttime were required to facilitate nocturnal operation of the existing runways.

The D-Runway Project was ordered as an all-inclusive design-and-build contract, and a joint venture consisting of 15 companies, represented by Kajima Corporation, was awarded the contract in March 2005 as the successful bidder. After the contract was signed, the detailed design work followed, and on-site construction started at the end of March 2007. The project was executed in a remarkably short period of about three and a half years, with completion scheduled by the end of August 2010 and the new runway was brought into service in October 2010 (Matsunaga et al. 2010).

Fig. 1. Bird’s eye view of D-Runway at Haneda Airport.
At Haneda Airport, functional enhancements are being implemented such as the improvement of aprons and implementation of earthquake prevention measures in preparation for the 2020 Tokyo Olympic and Paralympic Games. D-Runway, the fourth runway at the airport, will be maintained appropriately while continually grasping settlements and other changes. This paper describes the respective structural sections and records of construction work of D-runway at Haneda Airport (Ishihara et al. 2010).

2 OUTLINE OF D-RUNWAY CONSTRUCTION PROJECT

2.1 Project outline

An outline of the D-Runway project follows.

Owner: Kanto Regional Development Bureau, Ministry of Land, Infrastructure, Transport and Tourism

Type of contract: Design-and-build contract

Design and construction: Joint venture involving Kajima, Aomi, Obayashi, Penta-Ocean, Shimizu, Nippon Steel Engineering, JFE

Fig. 2. Layout of the 4th runway at Haneda International Airport.

Fig. 3. Plan and section of D-Runway.
2.2 Structural Data

D-Runway, as shown in Fig. 3, consists of four main structures including the reclamation area, the piled-elevated platform, the junction structure linking the reclaimed area and piled-elevated platform, and the connecting taxiway bridge for access from the existing airport to D-Runway. Data for the main structures is as follows:

1) Reclaimed land
   - Dimensions: 2,020 × 424 [m]
   - Total volume of soil, sand and rocks: 52 million cubic meters
   - Ground improvement: Sand compaction pile method (SCP) and sand drainage method (SD)

2) Pile-elevated platform
   - Dimensions: 1,100 × 524 [m] (approximately 52 ha)
   - Substructure: Steel jackets (n=198) and steel-pipe piles (n=1,165)
   - Superstructure: Composite structure of steel I-girders and concrete deck
   - Decks: Precast pre-stressed concrete slabs (6.6 × 3.3 [m], approx. 10,700 slabs) and UFC slabs (7.8 × 3.6 [m], approx. 6,900 slabs)

3) Connecting structure between reclamation area and piled-elevated platform
   - Dimensions: 428.7 × 14.4 [m]
   - Substructure: Steel-pipe sheet-pile cellular foundation
   - Superstructure: Slit-type wave-breaking seawall

4) Connecting taxiway bridge
   - Dimensions: length 620 m, width 63 m
   - Substructure: Steel-jacket foundation and steel-pipe piles
   - Superstructure: Four-span continuous steel I-girder and precast concrete deck composite structure (bridge section)
   - Precast pre-stressed concrete girder-slab structure (pier section)

2.3 Performance requirements in design

A performance-prescribed design approach was adopted for D-Runway, and the contractors proposed structural forms, materials and construction methods to satisfy the performance requirements specified by the client. Major challenges in the design were 1) soft clay ground beneath very deep water, 2) airplane loads, especially fatigue caused by repeated actions, 3) corrosion and deterioration in the marine environment, and 4) safety in the event of a large earthquake.

Since the designed durability of D-Runway in the contract is specified as 100 years and the contract included maintenance for 30 years by the bidder, thorough considerations for long-term durability were required at the design and construction stages.

2.4 Soil conditions

An image of the soil profile along the center of the runway (Line 4) is shown in Fig. 4. Ac-1 and Ac-2
(alluvial clay) existing to a depth of 33 meters below the elevation of the seabed is a soft clay stratum with an SPT N value of zero. Below this depth are alternate layers of sand and clay (Ds or Dc) of diluvial origin with SPT N values of 20 to 40. The sand and gravel layers deeper than 55 to 60 meters are firm with the SPT N values exceeding 50 (Noguchi et al. 2007).

In view of the subsurface stratification conditions as above, major problems related to soil deposits affected the consolidation and stability of slopes in various phases of the construction process (Kawamura et al., 2006).

(1) Consolidation of the soft clays or silts

Of grave concern were the consolidation characteristics of the 20- to 30-meter thick alluvial clay strata below the seabed. In the reclaimed lands on the northeast, water to depths of 15 to 20 meters had to be filled with sand and rubble. Above this, soil had to be piled up to a level 13 to 17 meters above sea level. Then, the total surcharge applied to the clay layer was as much as 500~580 kN/m², an unprecedented value that had never been applied in the Tokyo Bay area.

The soil parameters necessary to estimate the settlement progress were determined through a series of laboratory tests on undisturbed clay samples recovered on various occasions. Fig. 5 presents a summary of the distribution of compression index Cc per depth, showing a rather large Cc value in the order of 1.1~1.7 for the alluvial clay layers Ac-1 and Ac-2, while smaller values in the order of 0.4~0.5 were found for diluvial deposits below 35 meters. The average Cc value used for the consolidation analyses is indicated by the thick lines in Fig. 5.

The value of the over-consolidation ratio, OCR, obtained in the laboratory tests is plotted in Fig. 6, together with the average value adopted in the analyses. It is well known that choosing smaller OCR values tends to yield larger settlement values, because applied loads reach a state of normal consolidation more readily from an early stage of load application. It can be seen that the OCR value ranges between 1.2 and 4.0, but smaller values were chosen for the analyses.

(2) Slope stability

The stability of slopes at various stages of the construction of the embankment was another issue of concern for the successful execution of the filling work. Safety factors were checked at various places in the cross sections of the main seawall embankment as well as in the inner banks. The shear strength of soils was estimated based on data from cone penetration tests. If the safety factor was known to be less than the pre-specified value at a certain stage and at a place of filling, construction was stopped until it was certified that the safety factor was large enough as a result of consolidation, which was detected using cone penetration tests. This kind of on-the-spot decision and modification was essential for construction to proceed. Tests details will be described in the next section.

3 RECLAMATION AREA

3.1 Structure of reclamation area

The reclamation area is an artificial ground of with a maximum height 17.1 meters above sea level, constructed in waters about 20 meters deep. The seabed has a soft clay layer about 40 meters thick, whose settlement due to consolidation from the surcharge of reclamation was calculated to be 8 meters at most. Therefore the maximum thickness of reclaimed
soil was as large as 45 meters. A sectional view of the reclamation area is shown in Fig. 7. The embankment structure was a slope-type rubble mound seawall, which flexibly could allow settlement of ground, and a sand compaction pile (SCP) method was used to improve the soft clay layer. Since the reclamation height behind the rubble mound seawall was huge, lightweight improved soil made using dredged clay mixed with cement, known as the in-pipe mixed cement-treated soil method, was cast behind the mound to improve ground stability. Soil improvement of the reclaimed area inside of the rubble mound seawall was accomplished using the sand drainage (SD) method (Watabe et al. 2010).

3.2 Construction of reclaimed area
Ground improvement using the SCP and SD methods was accomplished with 24-hours-a-day work using 18 SCP and SD barges. After building the slope-type rubble mound seawall, in-pipe mixed cement-treated soil was cast behind the rubble mound seawall. The solidified soil was produced from dredged clay soil mixed with cement. Dredged soil and cement were mixed during conveyance of the materials through the compressed air-mixture pipeline floated on the sea. Cement was injected in quantities of 80 to 110 kg/m³ into the pipeline, in which plug flow generated by the conveyance pump pressure helped to uniformly mix the materials.

During the reclamation work, four reclamation barges were used to unload 80,000 m³/day of sand, which was carried by dump trucks and compacted with vibration rollers. The compacting thickness per layer was set as 90 cm, and the vibration rollers were mounted with GPS equipment to control the position and frequency of compaction. In order to control the density after compaction, small dynamic plate bearing testers (FWD testers) that enabled quick and easy measurements were used.

Since the reclamation area was built in a rapid and large-volume construction with a maximum filling height of 45 meters, controlling the stability and settlement was extremely important. In each work phase, the strength of the ground and other relevant factors were measured using RI cone penetration tests.
(RI-CPT), and reclamation was carried out while checking stability using slip circle calculations. Regarding determination of the crown height, consolidation settlement for the whole airport island was forecast by settlement analysis, taking into account the three-dimensionality of the work history, and the crown height of the reclaimed land was determined with a surplus reclamation for settlement so as to achieve the required design height once D-Runway was brought into service (Toyota et al. 2006).

3.3 Observations during reclamation
Because time was limited to just 41 months, there was not enough time to wait for 80–90% consolidation to take place in the thick alluvial clay stratum. Thus, the placement of fills was forced to proceed from the stage where about 50% consolidation had occurred. Thus, it was mandatory to employ the observational method to control or adjust the progress of filling work from time to time to reflect data from the multiple series of in-situ monitors by means of various devices or equipment. The locations of the monitoring equipment are shown in Fig. 8. There were six major sections about 200 to 500 meters apart along the sea wall where settlement gauges, piezometers, earth pressure meters and inclinometers were installed for intensive monitoring. The locations of observations in the transverse cross section are shown in Fig. 9.

3.4 Measurement of settlements
In each of the six transverse cross sections, settlements were monitored using CB settlement gauges. In the center of the seawall, a 60-cm-wide steel pipe was installed vertically on top of the sand compaction pile (SCP) before the fill materials were placed. Inside the steel pipe a rod to measure settlement was placed vertically with a plate fixed at the bottom; that is, on top of the compacted original seabed. As the fill was raised, sinking of the rod inside the pipe indicated the settlement of the seabed deposit. The aim in using the pipe was to enable additional boring or sounding to be made later inside the pipe underneath the rubble mound seawall. This is referred to as a CB settlement gauge.

In order to monitor settlements as the filling went on under water, devices based on water pressure measurements were buried in two locations. The settlements in the underlying alluvial deposits were monitored using conventional layer-by-layer gauges.
This type of device permits compression of each layer to be assessed. These three types of mutually complementary devices permit the overall settlement that occurs in the deposits below the elevation at which the device is placed to be monitored (Sakaiya et al. 2008).

3.5 Monitoring of changes in soil properties
In order to monitor changes in void ratios and shear strength of the alluvial clay layers, cone penetration testers equipped with radioisotope devices (RI-cone, Fig. 10) were utilized extensively. In addition to probes to monitor tip resistance, skin friction and pore water pressures, a device for emitting and receiving gamma (γ) rays is attached to the cone rod about 50 cm above the tip which is used to monitor the density and water content in the soil around the probes (Takahashi et al. 2008).

3.6 Estimate of settlements
Because of the overall time constraints imposed on the construction, the filling works did not have the luxury of being able to stop and wait until consolidation was 80–90% complete. On average, the ratio of consolidation allowed in the present project was about 50%. As such, it was necessary to predict forthcoming settlements in advance and to determine the modified thickness of additional filling at the next stage.

In the observational procedure adopted in Haneda, settlements were monitored at several key locations as shown in Fig. 8. One of the data sets on the settlement progress observed at station SK (see Fig. 8) nearest the junction seawall is presented in Fig. 11 by a chain of circles, together with records of increasing thickness of the filling. Also, there was an updated version of HASP (Haneda Airport Settlement Prediction) software, newly elaborated to calculate the progress of consolidation by integrating all conceivable factors relevant to local soil conditions.

Fig. 10. Operation of RI-cone.

Fig. 11. Comparison of computed settlement curve with actually measured settlement.
conditions in the Haneda area. Using this program, theoretical curves were obtained as shown by a solid line in Fig. 11. It is to be noted that the parameters, Cv, at different clay layers were modified several times so that the settlement-time curves could fit the measurement data. In the soil-filling record shown at the top of Fig. 11, underwater filling is known to have completed in February 2009, after which land reclamation was performed.

The adjustment was made in August 2009, so that it becomes coincident with the measured settlement value at that point of time. It is noted that there was a sharp increase in observed settlement on April 2009, due to the large increase in surcharge load, which is also reflected in the HASP-calculated curve. It is noted that the HASP prediction, as modified midway, describes the settlement curve after August 2009 with a reasonable degree of accuracy. If the HASP results were to deviate again, further adjustments would be made. It is evident from Fig. 11 that on completion of construction, total settlement was 6.8 meters. If HASP is assumed to be sufficiently accurate, long-term settlement after August 2010 was anticipated to be about 90 cm including sinking due to secondary consolidation.

3.7 Stability checkup for the seawall

The stability of the seawall was of major concern as the height was lifted by dumping rubble and also by piling up stones for the riprap. The key factor for governing the stability was the temporal change in the undrained shear strength of underlying clay deposits. As is well known, the driving force towards instability tends to increase while the shear strength remains unchanged at the time new fills are placed. Thus, the safety factor drops upon placing new fill. As the excess pore water pressure dissipates with time, the shear strength increases accompanied by an increase in the safety factor for potential slip surfaces. The temporal change in the undrained shear strength was assessed using the cone tip resistance qt and values of pore water pressure measured in-situ by the RI-cone.

The safety factor was calculated by assuming circular sliding surfaces for both the seawall and inner embankments. The target safety value required for stability was set at 1.3 for clayey deposits and 1.2 for sandy deposits. If the safety factor was known to be above the specified value, the filling operation continued, but if not, it was suspended until the pore water pressure had dissipated sufficiently with a resultant gain in strength.

4 PILED-ELEVATED PLATFORM

4.1 Structure of piled-elevated platform

The southwest part of the runway, 1,010 meters long and 524 meters wide, consists of a jacket-type framed structure with steel pipes used as bracing members. This structure is supported by steel piles 1.6 meters in diameter, which were driven to depths of 70–90 meters where the firm diluvial deposits exist. The jacket structure with a planned length of 63 meters and width of 45 meters as assembled is shown in Fig. 12. The jacket structure comprised of steel beam bracing members and six legs were assembled in a yard at Futsu and Soga across Tokyo Bay (Fig. 13) (Asada, et al. 2009). Each was towed on a flat barge to the targeted site, lifted by a 2,400-ton or 3,800-ton capacity floating crane and placed on top of the in-situ steel pipe piles (Fig. 14).

Due to the soft ground conditions at the site, 1,165 driven steel pipe piles with diameters ranging from 1.4 cm to 1.6 cm were installed.
to 1.6 meters and a length exceeding 70 meters are required to support the jacket structures. Rapid load tests and dynamic load tests were conducted on two test piles to confirm the performance of these piles (Fig. 15) (Noguchi, et al. 2012).

The diameters of the legs of the jacket were 20 cm larger than of the steel pipe pile to allow a clearance of ±10 cm for the set-in-place. A picture of a jacket unit about to be placed on top of the piles is presented in Fig. 14. 198 assembled jackets were used in total, and all of them were successfully positioned on the steel piles within a deviation of 5 cm. After the jacket was fixed to the steel pipe piles with grouting, precast concrete slabs were placed on top of the steel girders. There were two types of concrete slabs in Fig. 16, namely, the conventional type of pre-stressed reinforced concrete slab and a new type of slab called “Ultra-high-strength fiber-reinforced concrete, UFC”.

4.2 Ultra-high-strength fiber-reinforced concrete slab

A recently developed new type of concrete, UFC, had found limited avenues of utilization so far in small to medium-sized bridges or buildings. Nowhere in the world has the UFC beam been used more extensively or on a larger scale than in the jacket structure at Haneda Airport. In fact, 6,939 UFC slabs were used in the project in total. Each slab had dimensions of 7.82 by 3.61 meters, as shown in Fig. 17. It was reinforced in two directions by pre-tension cables underneath the UFC slab. It is amazing to know that a floor slab as thin as 25 cm can carry an aircraft with a working load of 400 tons. It is to be noted, however, that the UFC slabs were only laid down over the outer portion of the runway where heavy loads will seldom be applied except in emergencies (Kameda et al. 2009).

UFC concrete is composed of fine silica sand, fine
cement and some powdered additives mixed with tiny steel needle-like fibers 15 mm long and 0.2 mm in diameter. The ratio of the composition was 180 kg of water, 2,240 kg of silica sand plus powered additives, and 157 kg of fine steel fibers per cubic meter. Thus, the unit weight of the UFC was 2,588 kg/m³. When these ingredients are thoroughly mixed, a highly viscous and pourable material can be produced facilitating its spread throughout the prefabricated form without agitation where the tendons were already set in place with tension. When it solidifies, a highly competent concrete slab is produced with a compressive strength as high as 180 N/mm² (Takahashi et al. 2006, Kono et al. 2008).

The use of the UFC concrete slabs offered multiple advantages for the design of the jacket structure. Firstly, because of its high strength and no requirement for reinforcement within the concrete, it reduced the weight of the superstructure by 60%, thereby decreasing the horizontal force due to seismic load, resulting in cost savings in the construction of the jacket structure. Secondly, the permeability of the UFC was smaller by a factor of 100 compared to normal concrete. Thus, penetration by salt water will be reduced drastically, thereby increasing the durability of the concrete floor slabs, which must last over one hundred years.

### 4.3 Precast concrete slab

Out of 500,000 m² of jacket structure, the inner part covering 310,000 m² of runway and taxiway was covered with conventional pre-stressed concrete slabs (PCa slabs). 13,000 slabs of different thicknesses and reinforcements were used. The design strength of the concrete was 50 N/mm².

Each floor slab was 6.585 meters by 3.32 meters by 40 cm, as shown in Fig. 19, and weighed 25 tons (Morii et al. 2010). The slabs were carried to the site on flat-bottomed barges. The PCa slabs were laid on the steel girders with some space left between neighboring slabs. Reinforcements were placed in the space and concrete poured into it to firmly connect all of the PCa floor slabs. It is to be noted that the vast area of the airfield constructed in this way was designed to behave as if it were a huge flat plate without any joints (Minami et al. 2006).

### 4.4 Corrosion protection for steel structure

As mentioned above, the new 50-hectare airfield was constructed with a steel-framed jacket structure. Since the steel frame is subjected to salt water, steel corrosion was of primary concern for its sustained use over a hundred years. To cope with this problem, three kinds of countermeasures were implemented.

1. **Anti-rust cathodic protection**
   This method has been used extensively particularly for steel structures in salt-water environments. The section of the piles between the seabed and a level 1.5 meters below the surface were designed to resist rusting by means of electrode bars attached to the piles.

![Precast PC slab (PCa slab)](image1)

![Corrosion protection for steel jacket](image2)

![Inside of titanium steel-coated cover plate](image3)

Fig. 19. Structure of PCa-slab.

Fig. 20. Corrosion protection for steel jacket.
(2) Stainless covered plates
The top portions of the legs and the bracing members of the steel jackets are subjected to splashing by seawater. All of these steel members in the splash zone were covered by specially prepared stainless steel lining as thin as 0.4 mm.

(3) Use of titanium steel-coated cover plates
At the top of the jacket structure, the steel girders and their connections to the legs are exposed to salt-containing moisture. To protect against corrosion of these portions, it was decided to install a number of titanium steel-coated cover plates below the steel girders, covering the entire area of the jacket structure. Thus, a number of room-like spaces 3 meters high, 4 meters wide and 7 meters long were created between the floor slabs and the titanium steel cover plates sectioned by the steel beams. Facilities were also installed to send dry air to each of these rooms through a network of air tubes, and to keep the humidity in these rooms at no more than 50%. Considering the fact that thousands of small rooms are going to be maintained at 50% humidity over the hundred-year lifespan of such a large-scale facility, the installation of the titanium steel cover plates should be cited as one of the new features of the technological challenges in the Haneda Airport construction.

5 JUNCTION STRUCTURE BETWEEN RECLAMATION AND PLATFORM

5.1 Steel pipe sheet-pile cellular seawall
A major challenge in the new runway construction was the choice of structure at the junction between the reclaimed land and the piled-elevated platform. After considerable contemplation, a steel pipe sheet-pile cellular seawall was adopted, as illustrated in the longitudinal cross section of Fig. 21. A plan view of the seawall is shown in Fig. 22. An illustrative oblique view is shown in Fig. 23. It can be seen that the seawall, 428.4 meters long, consists of 24 square cells. The steel pipes, 1.6 meters in diameter, were driven down to a depth of 75 meters in two rows in a direction perpendicular to the runway. Steel piles with the same diameter were also driven in a direction parallel to the runway. The partition wall was positioned to transfer the earth pressure from the reclaimed land to the seaward row of pipe piles.

Coming back to the longitudinal section in Fig. 21, one observes that a massive earth pressure is expected to act on the junction seawall due to its height of 14 meters.
meters above sea level. To minimize the lateral displacement of the seawall towards the sea, the degree of the sand compaction pile was increased locally from 30% to 78% at the frontal toe of the seawall. In front of the seawall, a stone mound was placed from seabed level to depth of 8 meters below the surface for a distance of 54.5 meters forward from the seawall (Sunasaka et al. 2009).

5.2 Use of lightweight materials as backfills
Since the earth pressure on the junction seawall is expected to be considerable, a lightweight material was utilized to backfill the area behind the wall for a distance of 200 meters towards the reclaimed land as shown in the cross section of Fig. 21. The lightweight material for this purpose was produced by adding foam-generating agent to the cement-mixed slurry. When the cement-mixed slurry was produced from the dredged clayey seabed mud in the plant equipment on the ship, the foam-generating agent was added in a predetermined proportion and the mixed slurry was transported by air pressure to the respective sites through pipes. This artificial material is called “Super Geo-Material (SGM)”.

By changing the amount of foam-generating additives, the density of the soil can be adjusted. At the back of the junction seawall, the in-pipe mixed cement-treated soil with a density of 1.4 ton/m$^3$ was placed in the deep portion under the seawater, but for the backfill above sea level, lightweight SGM at 1.1 to 1.2 ton/m$^3$ was placed as illustrated in Fig. 21. The spreading of the SGM at sites was made in multiple layers with a height of 0.25~0.5 meters per lift.

The strength of the SGM was about 200 kPa and a requirement for its effectiveness is that the soils should have clay content in excess of 90%. The new technology for the SGM was developed in Japan and first utilized in 1995 to retrofit the quay walls at Kobe port after the earthquake in 1995. During the 15 years since then, 520,000 m$^3$ of SGM was used in various small-scale works, whereas in this Haneda D-Runway project alone, 900,000 m$^3$ was used.

5.3 Wave-absorbing seawall
The superstructure of the junction seawall is a slit-type wave-breaking seawall as shown in Fig. 24, which was constructed on a steel-pipe sheet-pile cellular foundation to prevent reflection waves during storms from affecting the superstructure of the steel jackets. Precast pre-stressed concrete columns (f$'_c$=80 N/mm$^2$) of 1,200-mm diameter were arranged with 600-mm slits between columns. Reflection waves were reduced as their energy was absorbed as they passed through the slits. Construction of the precast PC columns for the wave-breaking seawall is shown in Fig. 23.

5.4 Expansion joint
Expansion joints were installed on the junction seawall to absorb the relative movement between two different structures of the reclamation area and the piled-elevated platform caused by temperature changes or seismic actions.

The expansion joint used in the runway is a roller shutter type joint as shown in Fig. 25, which can absorb relative displacement of ±60 cm in directions parallel and perpendicular to the runway. In designing and fabricating the expansion joint, a full-scale model test with the same specifications as the actual expansion joint was carried out including dynamic movement tests, load tests, and fatigue tests, to make sure that the performance requirements would be satisfied (Marioni et al. 2006).
6 CONCLUSIONS

Runway D at Haneda Airport was completed in a short construction period of three and half years owing to the work 24 hours a day and 365 days a year. Service was commenced in October 2010. The runway has been making great contributions to Japan’s international competitiveness together with the facilities in the new international airline facilities that started service in the same period (e.g. international passenger terminal, cargo terminal, aprons and new stations of Tokyo Monorail and Keikyu). Further efforts will be made toward the enhancement of international competitive basis and the functional refinements at Haneda airport for ensuring aviation safety and security.

REFERENCES


