The 2011 Tohoku Region Pacific Coast Earthquake struck off Tohoku and Kanto regions of Japan on 11 March 2011. We conducted investigations of structural and surface damage to pavement in the runway, taxiway and apron at Sendai Airport for the purpose of re-opening the airport for commercial flights. Many cracks were confirmed in the asphalt pavement in the runway and taxiway. However, it was clarified that these cracks except the one in the taxiway were not fatal structural damage that would hinder the provisional use of the airport. Large settlement was confirmed in a part of the asphalt pavement in the taxiway and concrete pavement in the apron due to liquefaction. It was confirmed that these settlement areas needed to be reconstructed for the re-opening of the airport. Furthermore, the effect of the void underneath the cement concrete slab on FWD deflection was clarified.

**Key Words**: earthquake, asphalt pavement, concrete pavement, liquefaction, airport

1. **INTRODUCTION**

The 2011 Tohoku Region Pacific Coast Earthquake struck off Tohoku and Kanto regions of Japan at 14:46 JST on 11 March 2011 with a moment magnitude of 9.0 and epicenter 170km east of Sendai Airport as shown in Fig.1. Sendai Airport was severely damaged due to both the earthquake and the tsunami.

Table 1 shows a summary of recovery works from 11 March 2011 to 13 April 2011, for the provisional re-opening of the airport. According to the recovery works by the airport operator (the Japan Civil Aviation Bureau (JCAB)) in cooperation with the Japan Self-Defense Force (JSDF) and the United States Armed Forces (USAF), Sendai Airport was
Table 1  Summary of recovery works.

| Date      | Elapsed days | Activity                                                                                                                                 |
|-----------|--------------|------------------------------------------------------------------------------------------------------------------------------------------|
| 11 Mar.   | 0            | 14:46 Earthquake occurred.  
            |              | 14:49 Tsunami Warning was issued (expected tsunami height: 6m).  
            |              | 15:14 Tsunami Warning was increased (expected tsunami height: greater than 10m).  
            |              | 15:59 Tsunami struck Sendai Airport.  
            |              | Tsunami height was 5.7m at terminal building2).  |
| 13 Mar.   | 2            | 07:30 Tsunami Warning was decreased to Tsunami Advisory.  
            |              | 17:58 Tsunami Advisory was cleared.  |
| 14 Mar.   | 3            | The Japan Civil Aviation Bureau began removal of debris and recovery works of airport facilities.  |
| 15 Mar.   | 4            | A part of Apron and Runway B (600m eastwards) were re-opened for helicopters (limited to emergency transportation).  |
| 16 Mar.   | 5            | Runway B (1,500m eastwards) was re-opened for aircrafts (limited to emergency transportation).  |
| 20 Mar.   | 9            | The Japan River Bureau began drainage works of tsunami water in the airport (part of the works had been started from 13 Mar.).  |
| 29 Mar.   | 18           | Runway B (3,000m) was re-opened for aircrafts (limited to emergency transportation).  |
| 13 Apr.   | 33           | Airport was re-opened provisionally.  
            |              | Temporary commercial domestic flights started.  |

(a) Apron

(b) Parking Area

Photo 1 Sendai Airport soon after tsunami struck.

Fig. 2 Strong-motion record of earthquake at Iwanuma City3).
Photo 2 Sendai Airport (photographed by Geospatial Information Authority of Japan).

Fig. 3 Damage to airport pavement.
re-opened provisionally and commercial domestic flights re-started temporarily on 13 April, 33 days after the earthquake.

The authors investigated the asphalt pavement (runway and taxiway) and the concrete pavement (apron) between 21 to 27 March, 2011 to evaluate the structural and surface damage to the airport pavement for the purpose of re-opening the airport for commercial flights.

This paper describes the summaries on structural and surface damage to the airport pavements due to the earthquake. Furthermore, the effect of the void underneath the cement concrete slab on deflection measured by FWD (Falling Weight Deflectometer) is clarified.

2. SUMMARY OF DAMAGE TO SENDAI AIRPORT

Sendai Airport is located 16 km south of Sendai City whose population is about one million. There are two runways in the airport. Runway A with a length of 1,200 m and a width of 45 m is used for small aircraft and Runway B with a length of 3,000 m and a width of 45 m is the main runway in the airport. There are twelve aircraft stands at an apron in front of the passenger terminal building.

Since the tsunami struck off the airport as shown in Photo 1, JCAB began recovery works on the airport facilities after the tsunami advisory was cleared.

Since about 370,000 m³ of debris and 2,000 or more vehicles were drifted to the airport, one of the most serious matters in the recovery works was to remove these debris and vehicles.

While 47 small aircrafts and 20 helicopters were flooded by the tsunami, fortunately there were no large passenger aircraft in the airport when the earthquake occurred and the tsunami struck the airport.

Fig.2 shows a strong-motion record of the earthquake measured at ground surface by the seismograph of K-NET at Iwanuma City where the airport is located. This earthquake was characterized by a long duration of the earthquake motion.

Photo 2 and Fig.3 show a plan of the airport and damage to the airport pavement. It was confirmed that there were 12 cracks in Runway B, 11 cracks in the taxiway and large settlement due to liquefaction occurred in the taxiway and in the apron.

3. DAMAGE TO ASPHALT PAVEMENT

The runway and taxiway in the airport were asphalt pavements. The CBR value of subgrade used for the asphalt pavement thickness design was 10% (16% in part). The thicknesses of each layer are shown in Table 2.

(1) Visual inspection

Visual inspection of Runway B and the taxiway showed 12 cracks on Runway B and 11 cracks on the taxiway. All the cracks except one were in transverse direction and throughout the width of runway and taxiway as shown in Photo 3. The crack width was about 1-3 mm and 5 mm maximum. Though there was 5 mm faulting across crack R10 as shown in Fig.3, there was no faulting across other cracks.

| Layers                  | Runway | Taxiway |
|-------------------------|--------|---------|
| Asphalt concrete        | 28-42  | 27-39   |
| Surface course          | 5      | 4       |
| Binder course           | 10-11  | 10-11   |
| Asphalt stabilized base course | 13-26  | 13-24   |
| Granular material       | 18-56  | 18-56   |

(a) crack R9 (b) crack R10

Photo 3 Transverse crack in runway.

Bottom of asphalt concrete layer

Surface of asphalt concrete layer

Photo 4 Core sample of asphalt concrete layer at crack R1.
As a result of core boring at cracks R1, R10, and R11 shown in Fig.3, it was confirmed that the cracks extended from the surface to the bottom of the asphalt concrete layer as shown in Photo 4.

Highway and river cross under the runway and the taxiway as shown in Fig.3. Since liquefaction due to earthquake had been forecasted at four crossing points (runway-highway, runway-river, taxiway-highway and taxiway-river) before the earthquake, soil improvement works had been completed by 2010 at three crossing points and were being prepared at one residual at taxiway-highway crossing point in 2011.

As a result of visual inspection of Runway B and the taxiway, large settlement due to liquefaction was confirmed at the un-improved taxiway-highway crossing point as shown in Photo 5 though no large settlement occurred at the other three points, which had already been improved. The width of the settlement was about 5 m at both sides of the box culvert of the highway and the settlement was about a few decimeters in the taxiway shoulder.

Fig.4 shows a cross-section of this point. Since the settlement was confirmed at both sides of the box culvert and along the highway, the cause of the liquefaction seemed to be the low compaction of back filling soil beside the box culvert.

(2) Structural inspection by using FWD

To evaluate the bearing capacity of pavement, structural inspection was conducted by using FWD (maximum load was 200 kN and diameter of loading plate was 300 mm) at Runway B and the taxiway.

FWD loading was conducted at the cracks, 10 m apart from the cracks, runway-highway crossing point and taxiway-highway crossing point. Fig. 5 shows the loading positions at Runway B and the taxiway.

To evaluate the bearing capacity of asphalt pavement, an evaluation method described in the “Manual and guideline of Airport Pavement Maintenance and Rehabilitation” issued by JCAB was used. In this method, deflection ratio calculated as equation (1) is used to evaluate the bearing capacity of asphalt pavement.

\[ R = \frac{D_0}{D_{0\text{-cri}}} \]  

where \( R \) is deflection ratio; \( D_0 (\mu m) \) is deflection measured at the center of loading plate with a load of 200 kN; and \( D_{0\text{-cri}} (\mu m) \) is critical deflection at the center of loading plate with a load of 200 kN. This manual indicates that there could be some structural problems and other structural inspections such as an excavation survey may be needed in case the deflection ratio is larger than 1.0.

The critical deflection \( D_{0\text{-cri}} \) was calculated by multi-layered elastic theory in the actual conditions of FWD load 200 kN, 300 mm diameter of loading plate, thickness of each layer at loading position and in the common condition of elastic modulus of each layer as shown in Table 3, which was the same as in the manual. Since the elastic moduli of asphalt concrete layer (surface course, binder course and asphalt stabilized base course) vary depending on temperature and have a great influence on the result, they were estimated by surface temperature of pavement.
measured at loading positions\textsuperscript{5}). The box culvert was ignored for calculation of the critical deflection at the pavement on the box culvert since the box culvert was located at the ground much deeper than the subgrade of the pavement.

Fig.6 shows the deflection ratio in Runway B. Though the deflection ratios exceeded 1.0 at a few cracks, generally the deflection ratios were less than 1.0. In particular, the deflection ratio at the runway-highway crossing point was not much larger than those at other points of the runway. This indicates that soil improvement works as liquefaction countermeasure were effective.

Fig.7 shows the deflection ratio in the taxiway. The deflection ratio exceeded 1.0 at T8 due to crack and at the taxiway-highway crossing point due to liquefaction. In particular, the deflection ratio at a distance from the box culvert at the taxiway-highway crossing point was not large. This indicates that the effect of liquefaction on the bearing capacity of the pavement was limited in the narrow width beside the box culvert.

(3) Recovery works

Based on the results of these inspection, JCAB reconstructed the pavement at the taxiway-highway crossing point S2 where large settlement due to liquefaction occured and the pavement around crack T8 in the taxiway where the deflection ratio was relatively large by the time of airport re-opening on 13 April 2011.
4. DAMAGE TO CONCRETE PAVEMENT

The apron in the airport was jointed concrete pavement. Design thickness of slab was 42 cm and design $K_{75}$ value of granular base was 70 MN/m$^3$.

(1) Visual inspection

As a result of visual inspection of the apron, large settlement and many cracks about 1-3 mm wide were confirmed in the cement concrete slabs at aircraft stands No.1, No.2, and No.3 as shown in Fig.8 and Photo 6. In particular, maximum settlement was about 18 cm between aircraft stands No.2 and No.3. Since the cracks occurred around uneven settlement area, the cause of these cracks seemed to be uneven large settlement due to liquefaction.

New cracks were confirmed in May as shown in Fig.8. These cracks might have occurred because groundwater level in May became lower than that in March due to the drainage work.
(2) Structural inspection by using FWD

To evaluate the bearing capacity of pavement, structural inspection was conducted by using FWD (200 kN load and 300 mm diameter of loading plate) at the center of each cement concrete slab.

Fig. 9 shows FWD deflection $D_0$ at the center of each slab. However, $D_0$ at the settlement area between aircraft stands No.2 and No.3 tends to be larger than that at the non-settlement area around aircraft stand No.1, the relationship between $D_0$ and settlement is not clear as shown in Fig.10.

(3) Recovery works

Based on the results of inspections, JCAB decided to close aircraft stands No.1, No.2, and No.3 temporarily as it would be difficult to repair these slabs in time for the airport re-opening. Reconstruction of the concrete pavement and soil improvement for liquefaction in this area were conducted after the airport was re-opened provisionally.

In recovery works, voids with a depth of 7 cm and 20 cm underneath the cement concrete slabs were confirmed by the removal of two cement concrete slabs as shown in Photo 7.

5. EFFECT OF VOID UNDERNEATH CEMENT CONCRETE SLAB ON FWD DEFLECTION

As mentioned in Sections 3 and 4, the relationship between FWD deflection $D_0$ and surface settlement due to liquefaction was not clear in the concrete pavement though the deflection ratio was very large at the taxiway-highway crossing point in asphalt pavement where settlement was large. To determine the effect of the void underneath the cement concrete slab on FWD deflection, we compared the deflections at the settlement area and the non-settlement area in the apron.

The precise position of the void underneath the cement concrete slab in the apron was unknown since we did not conduct sufficient inspection to confirm the position of the void. Thus, we estimated and defined “slab with void” where both the settlement and the deflection were large around aircraft stands No.2 and No.3, and “slab without void” where both the settlement and the deflection were small around aircraft stand No.1. To examine the effect of the void underneath the cement concrete slabs on FWD deflection, deflection data of 20 slabs in each area were used.
(1) Normalized deflection

The shape of the deflection basin was examined. Fig. 11 shows a normalized deflection \( ND_x \) calculated as shown in equation (2).

\[
ND_x = \frac{D_x}{D_0}
\]  

(2)

where \( ND_x \) is normalized deflection at \( x \) (mm) apart from the center of the loading plate; \( D_x \) (\( \mu \)m) is deflection measured at \( x \) (mm) apart from the center of the loading plate; \( D_0 \) (\( \mu \)m) is deflection measured at the center of the loading plate.

The normalized deflection of the slab with the void tended to be larger than that of the slab without the void. Furthermore, it was also observed that the normalized deflection of some slabs with the voids exceeded 1.0, which meant that \( D_x \) was larger than \( D_0 \). These phenomena indicated that FWD deflection in a distance from the loading plate of the slab with the void could be close to the deflection at the center of the loading plate because the slab was not supported by a base layer due to the void underneath the slab.

(2) Peak time difference of deflection

The deflection data in time series were examined. Fig. 12 shows an example of deflection data in time series. In this examination, we defined “peak time difference, \( \Delta t_x \)” as shown in equation (3) and Fig. 12.

\[
\Delta t_x = t_x - t_0
\]  

(3)

where \( \Delta t_x \) (ms) is peak time difference of deflection at \( x \) (mm) apart from the center of the loading plate; \( t_x \) (ms) is time when deflection \( D_x \) becomes maximum in time series; and \( t_0 \) (ms) is time when deflection \( D_0 \) becomes maximum in time series.

Fig. 13 shows the peak time difference in the slabs with and without the void. The peak time difference in the slab with the void tends to be smaller than that of the slab without the void. This indicates that not only the center of loading plate but also portions in a distance from the loading plate tend to deform at almost the same time in the case of the slab with the void.

(3) Presumption of void

Fig. 14 and Fig. 15 show the relationship between normalized deflection and the peak time difference. As shown in these figures, the difference between the slab with the void and the slab without the void can be clearly verified by using these two indexes.

We try to presume the possibility of the void underneath the slabs in column N shown in Fig. 9 by using this method since deflections at the center of the slabs in column N are somewhat small in the apron; nevertheless, these slabs are very close to the settlement area.

Fig. 16 shows the results. There may be a void underneath the slab of N7 since this slab is plotted in the group of the slab with the void. On the other hand, there may not be a void underneath the slab of N14 though the deflection at the center of this slab is 331 \( \mu \)m, which is one of the largest deflections among the slabs in column N.

As a result, it is possible that the void underneath the cement concrete slab is easily detected by using the normalized deflection and the peak time difference measured by using FWD.
6. CONCLUSION

(1) There were several cracks in the asphalt pavement and large settlement due to the earthquake. Cracks in Runway B and the taxiway were not fatal damage. Settlement at the taxiway-highway crossing point due to liquefaction was too large for aircraft to use the taxiway. However, no settlement due to liquefaction occurred at the other three crossing points (runway-highway, runway-river and taxiway-river) at which soil improvement had already been conducted before the earthquake.

(2) Damage to the concrete pavement consisted of many cracks and large settlement in the apron due to the earthquake. The cause of many cracks seemed to be the large settlement due to liquefaction of the ground. As a result of inspection, large void due to settlement was confirmed underneath the cement concrete slabs.

(3) FWD deflection in a distance from loading plate of the slab with the void could be close to the deflection at the center of the loading plate because the slab with the void was not supported by a base layer due to the void underneath the slab. Thus, the normalized deflection of the slab with the void tended to be larger than that in the slab without the void.

(4) The peak time difference in the slab with the void tended to be smaller than that in the slab without the void. This indicated that not only the center of loading plate but also portions in a distance from the loading plate tended to deform at almost the same time as in the case of the slab with the void. It was possible that the void underneath the cement concrete slab was easily detected by using normalized deflection and the peak time difference measured by using FWD.

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