Analysis of coastal structure damaged by the 2011 Off the Pacific Coast of Tohoku Earthquake - Field investigation and numerical simulation -

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ABSTRACT

Site investigations were performed on a damaged river dike, which was located in the central and southern part of Iwate prefecture. The field investigation results show that subsidence related failure probably had taken place in the dike body with low penetration resistance and high water table. Further investigations also revealed that the dike body had low liquefaction resistance and the volume change after liquefaction was rather large. Numerical investigation through two dimensional effective stress analyses show that due to main shock and aftershock, there is a likelihood of liquefaction in dike body and the re-liquefaction possibilities are high even under small ground motion.

Keywords: coastal structures, river dike, numerical analysis, site investigations, liquefaction

1 INTRODUCTION

The 2011 Off the Pacific Coast of Tohoku Earthquake and the resulting tsunami caused irreparable damage to many coastal structures. In some cases, damage to structures was attributable to more than 1 m of tectonic subsidence caused by the strong ground motion and the record breaking tsunami that easily overtopped. The report of central disaster mitigation council of the ministry of Japan (Central Disaster Mitigation Council, 2003), states that about 2m of tectonic subsidence is expected in the Kochi area of Shikoku island, Japan, by that earth-quake. In order to mitigate the damage from such future devastating earthquakes, it is necessary to take appropriate measures that can protect the infrastructures from the compound disasters instigated by the combined effect of events such as earthquake, liquefaction, and tsunami.

This paper deals with field investigation and numerical simulation focusing on the structural and geotechnical aspects of a damaged river (Yoshihama river) dike due to the 2011 Off the Pacific Coast of Tohoku Earthquake. The river dike was located in the central and southern part of Iwate prefecture. The authors conducted field surveys of the river dike, using in-situ density test, dynamic cone penetration test, micro tremor measurement and surface wave exploration. Two dimensional effective stress analyses were also conducted to make clear the mechanism of the damage.

2 THE DISASTER IN THE YOSHIHAMA AREA

Yoshihama area repeatedly suffered tsunami damage during the past big earthquakes (Iwate Nippo, 2011). Due to complete collapse of the sea wall and the river dike (Fig. 1), the tsunami easily entered and inundated the plain area (Fig. 2). Due to strong shaking of the earthquake and tsunami that followed, the coastal dike at the mouth of Yoshihama river was completely damaged over a wide range with complete collapse of the concrete slabs. In some parts, more than 30 m displacement of the concrete slab was observed. As compared to many other sea walls destroyed due to the tsunami located along the coastal area of the Tohoku region (Hara et al., 2012; Hazarika et al., 2011) the damage to this dike in Yoshihama was not due to the scouring or erosion. It is worth mentioning here that during the first phase of our investigation (May, 2011),

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the paddy fields behind the sea wall were completely covered with deposited sands from the seashore, and the inundated waters were seen over a wide area. Our survey indicated the crown height of non-damaged river dike was +5.2 m. Fig. 1 indicates the tsunami inundation height as +16-17 m, which implies that the tsunami easily overtopped the river dike.

Fig. 1 Damage in Yoshihama area (Arrow shows tsunami direction)

Fig. 2 Tsunami inundation in the surveyed area

3 CHARACTERISTIC OF EARTHQUAKE

Furumura et al. (2011) showed the rupture process of the main shock of the 2011 off the pacific coast of Tohoku Earthquake using the acceleration record obtained from K-NET and KiK-net (K-NET, 2011). According to their study, the first rupture occurred off Miyagi prefecture, and strong seismic waves were released all over Tohoku (phase1). After several tens of seconds, another massive rupture occurred and strong seismic waves were released (phase2). The third rupture occurred at the offshore near the northern Ibaraki, and strong seismic waves were radiated towards Ibaraki prefecture (phase3). Fig. 3(a) shows the acceleration records of the main shock recorded at K-NET IWT007 station (at Kamaishi), which recorded maximum acceleration 741.6 Gal. Fig. 3(b) shows the acceleration records of the main shock recorded at K-NET IWT008 station (at Ohfunato), which recorded maximum acceleration 387.0 Gal. In both the acceleration records, continuation time was over 220 seconds.

4 FIELD INVESTIGATION ON YOSHIHAMA RIVER DIKE

4.1 Types of Investigations

As a part of the site investigation, GPS surveying, portable dynamic cone penetration test (PDCP), surface wave exploration and micro tremor measurement were conducted at two locations of Iwate prefecture. Disturbed soil samples were also collected from the sites and laboratory investigations were carried out.

PDCP is recognized widely as a standard method for obtaining dynamic characteristics of soils at the site by
the Japanese Geotechnical Society (JGS 1433). In PDCP, a drop hammer weighing 5 kg is allowed to fall through a rod from 50 cm height, which enables the cone attached at the toe of the rod to penetrate into the ground. The number of blows (Nd) to penetrate every 10 cm of the ground measured. Nd is related to the N-value of the standard penetration test. In this study, using the relationship proposed by Okada (1992) for sandy soils, Nd values were converted to N-values. The location of the ground water table was judged from the wet condition of the rod immediately after termination of the test.

The surface wave exploration is a convenient method to obtain S-wave velocity distribution (within the ground up to a depth of 10 m), which can measure and analyze the transmission of the surface wave (Rayleigh wave) that transmits near the ground surface. In this method, a wave is generated by striking the ground surface with a hammer. The generated wave propagates according to the surface and subsurface material conditions. During our investigation, in order to obtain the characteristics of the ground layer indirectly from the surface, the surface wave exploration was carried out together with the PDCP. The locations of the investigations are shown in Fig. 4.

4.2 Results and discussion

Fig. 5(a) shows an estimated standard cross section of the dike before the earthquake. The dike is having the standard shape typically used in Japan with 5.15 m in height, 2.0 m in width at the crown, and 1:1.2 in gradient at the back. The structure of the dike consists of a concrete wall with counterfort on the river side, and a dike body with filled soil covered with concrete blocks on the back side of the slope. Fig. 5(b) shows the N-value converted from Nd obtained from the PDCP test conducted on the soils of the dike body. The N-values of the fill soils lie within the range of 1 to 4. This implies that the soil was in very loose state. However, the N-value increases along the bottom of the body, implying a dense state. From the PDCP test the ground water level was confirmed to be at 2 m below the ground surface. Therefore, it can be said that the fill soil was almost at the saturated state.

Fig. 6(b) shows the S-wave velocity distribution analyzed from the surface wave exploration data of the ground near the dike (Fig. 6(a)). The S-wave velocity ranges from 200 to 250 m/s near the surface, and the converted N-value was about 20 (Imai & Tonouchi, 1982), which implies that the surface soil is very dense. Near the end of the measured zone, the S-wave velocity was found to be low. Based on the observations of the topography and the deposited sand due to tsunami, it can be said that the lower velocity was due to the reclaimed soils used in construction.

The results of the microtremor measurements are shown in Figs. 7(a) to (c). The amplitude ratio calculated in this study was based on the method proposed by Nakamura (1989). Measurements were made at the top as well as at the bottom of the dike. On the surface of dike body, the predominant frequency of the soil deposits is 3.1 Hz (Fig. 7(a)). Near the toe of the dike body, the predominant frequency of soil deposits is between 4.0 to 6.0 Hz (Fig. 7(b)). Spectral ratio between the top and the bottom of the sloping side was found to be 2 Hz (Fig. 7(c)). As observed in Fig. 6(b), the shear wave velocity is approximately 200 m/sec within 5 m from the surface layer. The predominant frequency of the site is 10 Hz. In the downstream, the shear wave velocity of the surface wave within 5 m from the surface is below 100 m/sec. Thus, the result of the micro tremor measurements and the surface wave exploration shows a good agreement.
In order to analyze the cause of damage of the embankment due to the strong ground motion, numerical analyses were conducted for the damaged parts of the dike. The numerical model is shown in Fig. 8. The parameters used in the analyses were determined based on the in-situ and laboratory tests as described in Hazarika et al. 2013. Simulations were performed by considering the history of the main shock, aftershocks.

The target structure is the damaged part of the Yoshihama dike shown in Fig. 8. The bottom of the analysis model was set in the location, where the measured shear wave velocity ($V_s$) exceeded 350 m/s. Regarding the boundary conditions, the bottom was taken as viscous boundary and the sides were set as equal displacement boundary.

The input parameters are shown in Table 1. The input parameters were determined based on the undrained cyclic test of the materials collected from the surveyed site (Ueno et al. 2012), and using the simulations of the element tests. Embankment soils and foundations soils were modeled as elasto-pastic materials proposed in Oka et al. 1994. The retaining wall, concrete covers at the top and the back of the embankment were modeled as linear elastic materials. The input parameter of the concrete was determined that is equal to the design strength of 18 N/mm². The interface between the wall and embankment was simulated using join elements. The contact and separation between the wall and the embankment were also simulated. The ground water level was considered to be at T.P. +1.78 that was observed when it had passed enough time after the earthquake. Fig. 9 shows...
the earthquake record of KiK-net that was recorded near the surveyed area, which was adopted as the input ground motion in the analyses. It is to be noted that the area around our case study experienced the main shock at 14:46 hours and the two aftershocks at 15:06 hours and 15:09 hours (JMA 2011). Ground motion observed at KiK-net Kamaishi (IWTH23), which has almost the same Vₜ as the base of analysis model, was used in the analyses. The ground motions (NS direction and EW direction) were synthesized by changing to direction normal to the dike. The deformation of the dike is shown in Fig. 10. Due to the main shock and two aftershocks, the embankment subsided by about 0.67 m and the back slope suffered displacement and peeling off of the concrete covers as observed in the in-situ survey. However, near the toe of the embankment no large deformations were observed due to the ground motions.

5.2 Results of Analyses

Fig. 11 shows the development and dissipation of the excess pore water pressure due to the ground motions in the element A within the embankment located below the ground water level. It can be seen that in spite of the sound bed rock that supports the dike, due to the maximum ground motion of about 400 Gal, almost whole of the bottom part of the embankment reached the excess pore water ratio of 1.0, and as result many locally liquefied zones were observed. However, within 20 minutes of the main shock (before the aftershock), the excess pore water pressure almost dissipated. During the first aftershock when the maximum acceleration was about 200 Gal, the excess pore water pressure within the embankment built up again, and the Δu/σ’ value reached 1.0. The pore water pressure dissipated before the second after shock (maximum acceleration 70 Gal) and Δu/σ’ values came down to 0.4. However, the second aftershock brought the Δu/σ’ value again to 0.9 , and as a result a part of the embankment experienced re-liquefaction. The vertical displacements of the embankment at two points (node 1 and node 2 of Fig 8) due to dissipation of the excess pore water pressure after the main shock and the aftershocks are shown in Fig.12. The vertical displacement δₑ in the surface of the embankment were calculated to be 0.52 m after the mainshock, 0.60 m after the first aftershock, and 0.67 m after the second aftershock. Therefore, it can be inferred that the subsidence of the embankment progressed due to repeated dissipation of the pore water pressures, and as a result even before the arrival of the tsunami, the embankment suffered huge deformation. It is to be noted that the subsidence at the end of the second aftershock calculated from the simulation agrees with the value recorded in the field survey.

![Fig 9 Input ground motion records of Main shock and Aftershocks at KiK-net Kamaishi (IWTH23)](image_url)
Fig. 10 Deformation of the dike due to the main shock and aftershocks

Fig. 11 Time history of the excess pore water pressure

Fig. 12 Subsidence of the embankment due to the ground motions

6. CONCLUSIONS

Based on the field and numerical investigations, the following conclusions could be drawn.

1. The Yoshihama river dike was damaged by subsidence related failure in the dike body with low soil density and high water table. Furthermore, the tsunami and the backrush led to further reduction of the strength of the levee body in spite of the existing concrete blocks at the back.

2. When several ground motions from the main shock and the aftershocks act on the dike, in the saturated zone of the foundation soils a large change of the excess pore water pressure develops. As a result even under small aftershocks, the foundation soils undergo re-liquefaction.

3. The subsidence of the dike body due to dissipation of the excess pore water pressure after liquefaction increases due to repeated loads under the main shock and aftershocks.

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