Case Studies of Geotechnical Damage by the 2011 off the Pacific Coast of Tohoku Earthquake and Tsunami in Japan

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SUMMARY
The strong quake and the associated tsunami due to the 2011 off the Pacific coast of Tohoku earthquake resulted in huge damage to many geotechnical structures. A lot of coastal protection structures suffered heavy casualties due mainly to the scouring and erosion by the tsunami. The main objective of this research was to collect basic information related to the seismic resistant and tsunami resistant characteristics of the earth structures in the tsunami affected areas. To that end, site investigations and laboratory investigations were performed on two earth structures (a damaged coastal dike and a non-damaged retaining wall) in the southern central part of Iwate prefecture. The results show that subsidence related failure probably had taken place in the dike body with low soil density and high water table. In the case of non-damaged retaining wall made of tires, due to flexibility and ductility of the tires and the hoop tension that prevails within each individual tire prevented the wall from any damage due to the impact forces of debris and tsunami.

Keywords: Dynamic deformation, Earth structure, Flexibility, Scouring, Site investigation

1. INTRODUCTION

The March 11, 2011 Tsunami triggered by the 2011 off the pacific coast of Tohoku Earthquake resulted in the greatest natural disaster (the so-called East Japan Disaster) to have ever struck in Japan in recent decades. The gigantic earthquake of magnitude Mw = 9.0 was the most powerful known earthquake to have hit Japan, and one of the five most powerful earthquakes in the world overall since record-keeping by strong motion seismograph began in 1900. Several compound disasters (Hazarika, 2011) followed the earthquake including widespread liquefaction in a vastly wide area covering the prefectures of Aomori, Iwate, Miyagi, Fukushima, Ibaraki, Chiba, Kanagawa, Tochigi and in some parts of Tokyo metropolis. Although the immediate recovery from the disaster is a major issue now, however, for many years to come the long-term solutions and recoveries are the challenging issues facing the geotechnical engineering community.

The earthquake resulted in the subsidence of the ground, and the subsequent tsunami inundation led to the collapse of coastal protection structures. Especially, the tsunami caused damage to many river banks and railway embankments. According to investigation by the Ministry of Land, Infrastructure, Transport and Tourism (MILIT), Japan, there were more than 1195 damage in the river bank that

Tohoku district maintenance office directly manages (MILIT, 2011). In the river mouth, the damage were mostly by the tsunami. In the other parts of the river banks, the damage were mostly due to the subsidence by either earthquake motion or the liquefaction of soils in the levee body. According to investigation conducted by Hara et al. (2012) in southern central part of Iwate prefecture, the level of damage by the tsunami on river banks varies according to the structural forms, such as existence of surface covering, materials and topographical features. According to Hara (2011) the overtopping tsunami caused sliding failures in many land development sites. On the other hand, according to the investigation conducted by Hazarika et al. (2012) in Aomori prefecture and northern part of Iwate prefecture, most of the damage of the river banks or coastal dikes were mainly due to scouring at the back of the structures. According to the observation made by Hazarika et al (2012), scouring was caused not only by the overtopping tsunami itself, but also the force of the backrush of the tsunami. Extent of damage by the tsunami was high, especially near the coastal area, due to more than 1 m ground sinking caused by the strong earthquake as well as the tsunami that easily overtopped many coastal structures. According to the central disaster mitigation council of the ministry of Japan (Central Disaster Mitigation Council, 2003), about 2m ground sinking is expected in the Kochi area of Shikoku island, Japan, by the Nankai earthquake, which is predicted to occur any time in the near future. 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 an earthquake, liquefaction, and tsunami. Therefore, it is necessary to investigate the cause of damage to various coastal protection structures which were affected during the disaster.

This paper is a report of the site investigations focusing on the structural and geotechnical aspects of a damaged river levee with concrete covering and a non-damaged retaining wall made of tires due to the 2011 off the pacific coast of Tohoku Earthquake and the associated tsunami. The report contains information based on the investigations conducted by the authors covering seismological, geotechnical and structural aspects of the two structures due to the tsunami using both the field survey and laboratory testing. As a part of the field survey, in-situ density test, dynamic cone penetration test, micro tremor measurement and surface wave exploration were conducted. Laboratory investigations were conducted on soils sampled from the sites to determine their earthquake resistant characteristics.

2. OUTLINE OF THE TSUNAMI AND THE EARTHQUAKE

2.1 Tsunami and the Damage

Field surveys were conducted on two geotechnical structures located in Yoshihama area and Okirai area of Ofunato city, Iwate prefecture. The surveys were conducted in two phases: the first phase was on May 3 and 4, 2011, and the second phase was six month after the disaster (September 12 and 13). Yoshihama area repeatedly suffered tsunami damage during the past big earthquakes. Due to complete collapse of the sea wall and the river levee by the tsunami this time, the tsunami easily entered and inundated the plain area. 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 and concrete blocks (Fig. 1a). In some parts, more than 30 m displacement of the sea wall was observed (Fig. 1b). 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., 2012) the damage to this sea wall 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), 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. However, in the next phase of our investigation (September, 2011) almost all the flood waters receded.

On the other hand, the whole of Okirai area was inundated completely by the tsunami. The causalities were greater in this area with 66 deaths and 30 missing people. As shown in Fig. 2(a), the concrete sea wall along the coastal line completely collapsed. As a result, the tsunami run-up washed away most of the wooden buildings in the area (Fig. 2b). The tsunami run-up was up to the third floor of the Okirai elementary school, which is located 200 m from the coastline. The tsunami run-up height in this location was found to be 16 m by the RTK-GPS survey conducted by the authors. According to previous records, the tsunami run-up height in Sugishita area of Okirai was 11.6 m during the Showa Sanriku tsunami in 1933, and was 7.8 m during the Meiji Sanriku tsunami in 1896 (Shuto, 2011). Based on our surveying data using the total station, the tsunami run-up height this time was estimated to be 16.79 m, which is much higher than past tsunamis that inundated this area.

(a) Collapse of seawall (b) Collapsed seawall and flood in paddy fields

Figure 1. Damage situation in Yoshihama

(a) Collapse of seawall (b) Collapse of houses in area protected by levee

Figure 2. Tsunami damage in Okirai

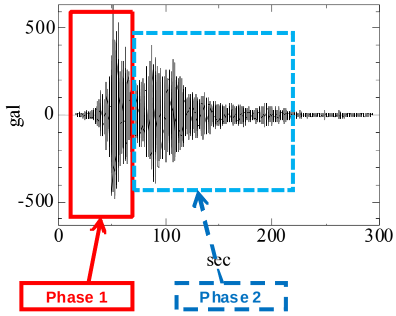
Figure 3 shows a tire retaining wall that miraculously survived the disaster due to the 2011 Great East Japan Earthquake. This tire retaining wall is located about 150 m away (towards the land) from the collapsed sea wall of Fig. 2a. The building situated on the backfill ground was damaged by the tsunami, and a natural slope nearby this tire retaining wall was eroded by the tsunami. However, this tire retaining wall was neither damaged by the earthquake nor by the inundation and erosion due to the tsunami.

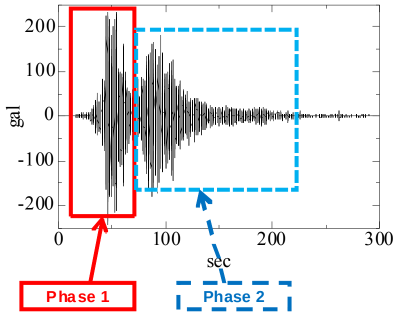
Figure 3. Tire retaining wall in Okirai (Photo taken after the compound disaster)

2.2 Earthquake Characteristics

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). The rupture property and the radiation characteristics of the third slip were different from those of the others.

Figure 4(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. 4(b) shows the acceleration records of the main shock recorded at K-NET IWT008 station (at Ofunato), which recorded maximum acceleration 387.0 Gal. In both the acceleration records, continuation time was over 220 seconds. Such long continuation time, which is the characteristics of the earthquake this time, was one of the reasons for the damage to more than 2000 river dikes and coastal dikes in the Tohoku region.





(a) IWT007 (Kamaishi) (b) IWT008 (Ofunato)

Figure 4. Acceleration records of the main shock recorded at K-NET

3. ANALYSIS OF DAMAGE TO GEOTECHNECAL STRUCTURES

3.1 Damage Investigations

As a part of the site investigation, 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 5kg is allowed to fall through a rod from 50cm height, which enables the cone attached at the toe of the rod to penetrate into the ground. The number of blows (N_d) to penetrate every 10 cm of the ground measured. N_d is related to the N-value of the standard penetration test. In this study, using the relationship proposed by Okada (1992) for sandy soils, N_d values were converted to N-values. The location of the ground water table was judged from the wet condition of the rod immediately after the finishing 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. This method measures and analyses 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 this 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.

Micro tremor measurement has become a powerful tool for engineers to estimate the ground motion characteristics, amplification of ground motion in the soil deposits, microzonation and dynamic behavior of existing service structures. The micro tremor observations described in this study were carried out by portable micro tremor equipment (type New PIC). Measurements (2 horizontal components and 1 vertical component) were conducted in velocity mode, which were recorded by the sensors. H/V Ratio is then calculated based on the smoothened ratio of horizontal to vertical Fourier spectra of the micro tremor data. The amplitude ratio calculated in this study was based on the method proposed by Nakamura (1989). The value corresponding to the peak represents the predominant frequency of the motion.

3.2 Site Investigation at Yoshihama River Dike

3.2.1 Field survey

Figure 5 shows the plane view of the river dike damaged by the tsunami after the earthquake at Yoshihama area. PDCP (one location), the surface wave exploration (over a distance of 24 m) and the micro tremor (two locations) were carried out on the levee body.

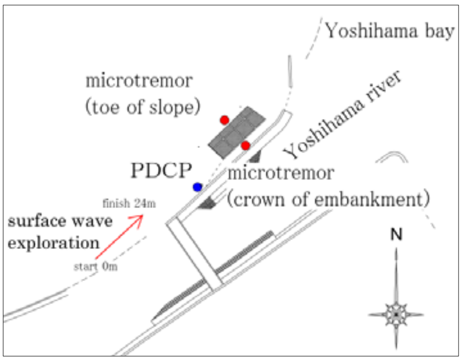
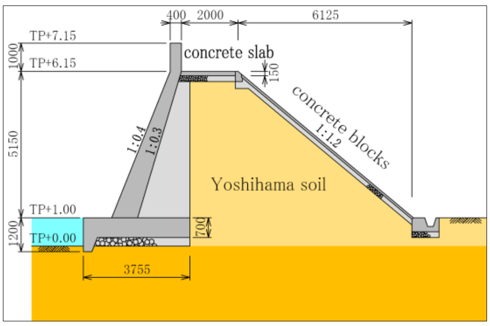
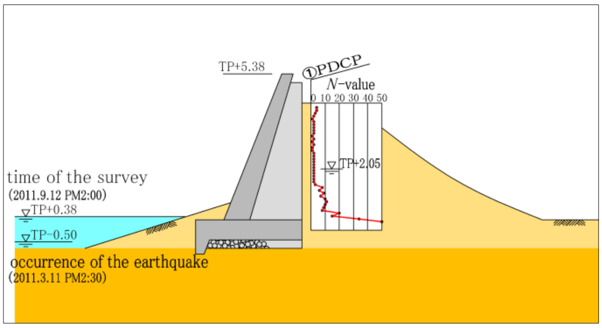


Figure 5. River dike damaged by the tsunami after the earthquake at Yoshihama area





(a) Before the earthquake (b) After the earthquake

Figure 6. Estimated cross section of the sea wall before the earthquake

Figure 6(a) shows an estimated standard cross section of the embankment before the earthquake. The embankment 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 sea wall consists of a concrete wall with counterfort on the river side, and a levee body with filled soil covered with concrete blocks on the back side of the slope. Figure 6(b) shows the N-value converted from Nd obtained from the PDCP test conducted on the soils of the levee body. The N-values of the fill soils ranged between 1 to 4, implying that the soil is very loose. 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 is almost saturated.

Figure 7 shows the S-wave velocity distribution analyzed from the surface wave exploration data of the ground near the embankment. 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 of restoration work.

Figures 8(a)～(c) show the results of the micro tremor measurements. Measurements were made at the top as well as at the bottom of the embankment. On the surface of levee body, the predominant frequency of the soil deposits is 3.1 Hz (Fig. 8a). On the bottom of the levee body, the predominant frequency of soil deposits is between 4.0 to 6.0 Hz (Fig. 8b). Spectral ratio between the top and the bottom of the sloping side was found to be 2 Hz (Fig. 8c). As seen from Fig. 7, 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. At the lower stream, the shear wave velocity of the surface wave within 5 m from the surface is blow 100 m/sec. Thus, the result of the micro tremor measurements and the surface wave exploration shows a good agreement.

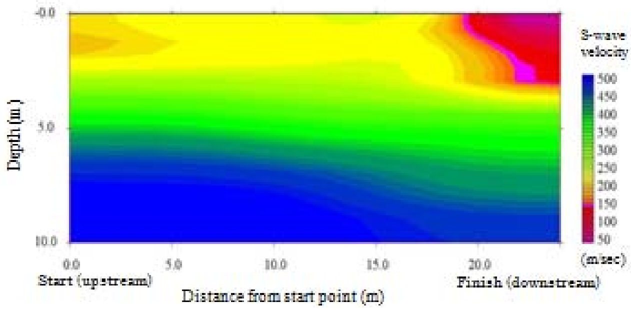
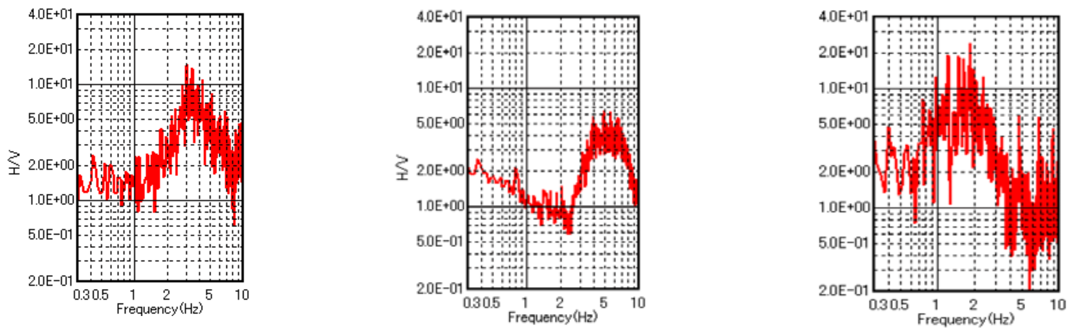


Figure 7. Distribution of S-wave velocity (River levee in Yoshihama area)



(a) H/V-ratio at the surface (b) H/V-ratio at the bottom (c) Transfer function

Figure 8. Micro tremor observation (River levee in Yoshihama area)

3.2.2 Laboratory investigations

Physical testing and shear testing of soil samples collected at the investigation site in Yoshihama area (called Yoshihama soil hereafter). Test samples were found to be composed of gravelly soil that included gravel with a maximum grain diameter of 9.5 mm. The grain size distribution revealed that the samples contain gravel fraction of 12% and fine fraction of 20%. Soils passing through 0.425 mm sieve had a plasticity index I_P of 4.7, indicating some level of non-plasticity in the samples. In-situ wet density test of the soils at the site conducted using the core cut method (JGS 1613-2003) was obtained to be 1.46 g/cm^3, and the degree of saturation was found to be 34.8%. The relative density of the embankment soils calculated using the JIS A1224 method was found to be 60%. Thus, there is a likelihood of liquefaction of the soils by the strong shaking during the earthquake. The shear strengths of the soils were determined using the cyclic triaxial shear test. In the triaxial apparatus used in this research, the specimen size was 50 mm in diameter and 100 mm in height. The soil specimens were prepared by the wet tamping method, because the other preparation methods such as air-pluviation or water-pluviation tend to intensify the segregation of soil particles for a well-graded granular soil. The relative density was adjusted by tamping to approximate target values at the site (D_r = 60%). The specimen was saturated fully by supplying CO_2 gas and de-aired water into the specimen. The specimen was isotropically consolidated by applying an effective stress of 49 kPa and maintaining the back-pressure at 98 kPa. The Skempton's B-value measured was greater than 0.96 in all the tests.

Figure 9 shows the relation between the cyclic stress ratio $\sigma_d/2\sigma_c'$ and the number of cycles N_c from the undrained cyclic triaxial test when the double amplitude of the axial strain (DA) reached 2%, 5%, and 10% at the excess pore water pressure ratio of 0.95. It can be seen that the liquefaction strength has changed significantly with the increase in the number of cycles. Fig. 9 also shows liquefaction strength curves for various other alluvial sands and decomposed granite soil without fines (Hara et al., 2004). The liquefaction strength, R_{L20} of Yoshihama soil is found to be low (0.18) at N_c = 20. Comparing the R_{L20} values, one sees that Yoshihama soil has a lower strength than alluvial sand having the same relative density. The internal friction angle of Yoshihama soils was found to be 36.6 degree, which is close to value proposed by Nishida and Aoyama (1984) for undisturbed soils.

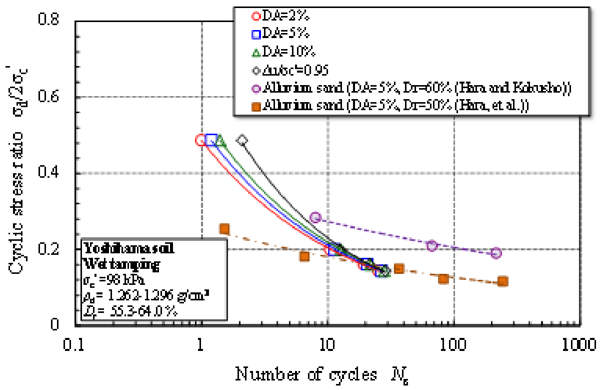


Figure 9. Cyclic triaxial test results

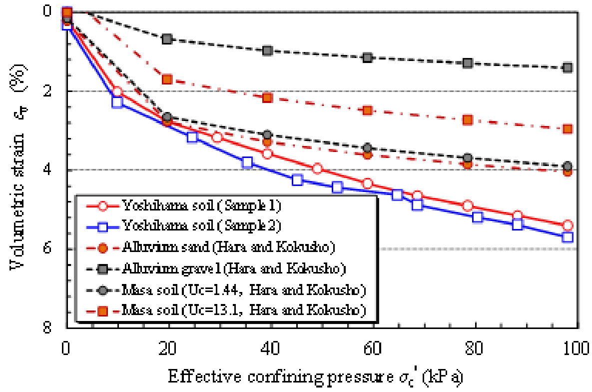
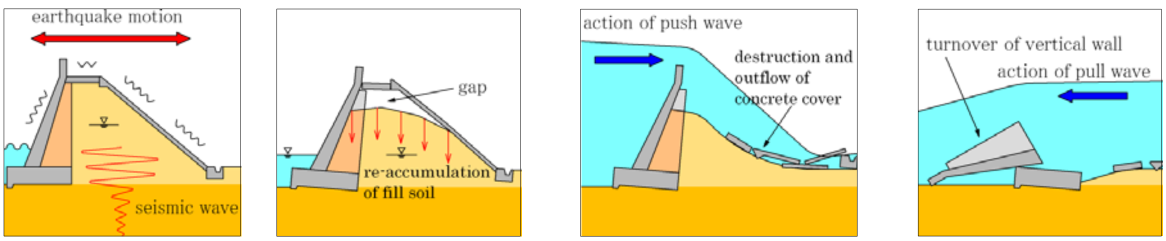


Figure 10. Consolidation test results

Figure 10 shows volumetric strain ε_v versus effective confining pressure σ_c' relationships obtained from the consolidation tests carried out after the undrained cyclic triaxial tests. The volumetric strain was found immediately after removing the load when DA reached 10%, based on the amount of drained water in a burette when specimens were returned to the drained state at the point of completion of the initial consolidation. Figure 10 also shows the same relation at D_r = 50% for alluvial sand, alluvial gravel, and decomposed granite soil without fines. The change in volume for the Yoshihama soil after liquefaction was greater than for the alluvial sand and gravel containing hard grains or decomposed granite soil containing soft grains.

3.2.3 Mechanism of the collapse of the levee

From the results of the site investigation on the coastal dike at Yoshihama area, it could be confirmed that the depth of dense layer in the embankment was shallow; the fill soil was not compacted enough and was in loose state when the earthquake and the tsunami struck. Also, the water level within the levee body was high. On the other hand from the laboratory investigation on the sampled soil, it was found that the pore water pressure rose within the embankment due to the strong earthquake motion with long continuation time and the fill soils liquefied. Due to huge volume change resulting from liquefaction, the levee body subsided and this has led to the collapse of the structure. Furthermore, the tsunami, which followed the earthquake, led to further reduction of the strength of the levee body in spite of the existing concrete blocks at the back.



(a) Before the EQ. (b) Just After the EQ. (c) Effect of the tsunami (d) Effect of backrush

Figure 11. Mechanism of the collapse of the embankment

Figures 11 (a) ~ (d) illustrate the mechanism of the collapse. The top layer of the fill soils subsided largely by the action of cyclic loading during the earthquake as shown in Fig 11(a). This has resulted in the generation of voids and differential settlement between the levee crown and the concrete blocks as shown in Fig 11(b). Furthermore, as shown in Fig. 11(c), the vertical wall tilled due to extra force of the tsunami wave, and the inundated water entered into the levee body resulting in the scouring of the sides and the toe of the levee. Finally, as shown in Fig. 11(d), the backrush of the tsunami further deteriorated the strength of the levee due to scouring and the force due to backrush led to the complete collapse of the wall.

3.3 Site Investigation in Okirai Area

3.3.1 Field survey and laboratory investigation

Figure 12 shows plane view of the tire retaining wall in Okirai. Fig.12 also shows the locations of the various field surveys (in-situ density, PDCP test, surface wave exploration, and micro tremor measurement) that were conducted. Soils samples were also collected from the three locations (No.1, No.2, soil within tire). Fig.13 shows distribution of S-wave velocity by surface wave exploration method conducted on the backfill soils. As shown in Fig.13, beyond the depth of 10 m, S-wave velocity is greater than 220m/s implying a hard stratum near the sloping side. Within the depth of less than 10 m, stratum with 150~200 m/s of S-wave velocity exists. Since the average height of the retaining wall was 3.2 m with a maximum height of about 4 m, it can be said that as a whole, the backfill soil was in the loose state. Fig.13 also shows converted N-value obtained from the PDCP test. Converted N-value and S-wave velocity in general are showing the similar trend. The converted N-values to the depth of 0~70 cm are high, so it can be said that near the surface the backfill soil has a very high density. The higher density near the surface may be the result of influence of cyclic load experienced by the backfill soils due to parked cars, since the yard was used as a parking lot. On the other hand, converted N-value within the depth of 1~1.5 m is about 5. Therefore, if we consider that the backfill consists only of sandy soil, it can be said that in general the backfill soil was in loose state.

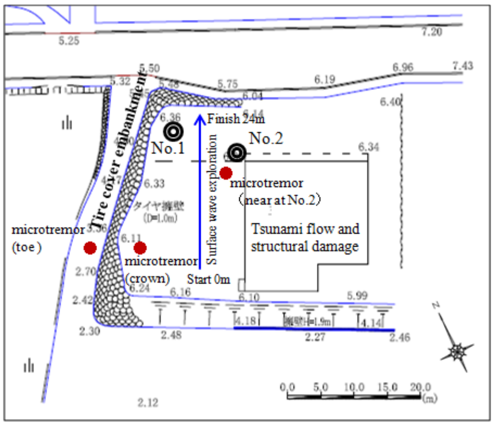


Figure 12. Tire retaining wall in Okirai

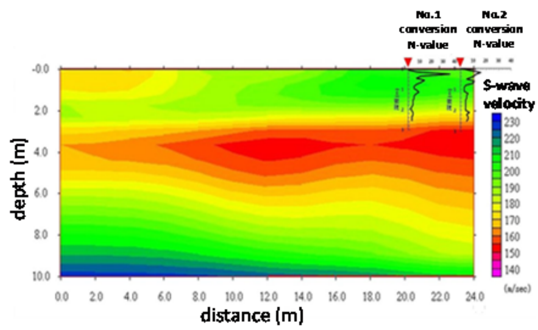
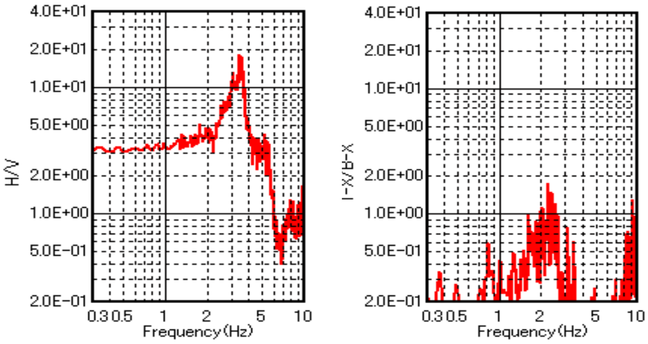


Figure 13. Distribution of S-wave velocity

Figure 14 shows the results of the micro tremor measurements made at the top as well as at the bottom of the embankment. At location No. 2, the predominant frequency of H/V ratio of the soil deposits is 3.1 Hz (Fig. 14a). Spectral ratio between the top and the bottom of the sloping side was found to be 2.1 Hz (shown in Fig. 14b). The shear wave velocity from Fig. 13 was found to be approximately 200 m/sec beyond 10 m depth. The predominant frequency of site is 3.8 Hz. Therefore, it can be said that the result of the micro tremor measurements and the surface wave exploration is in good agreement. Laboratory test was conducted for soils samples that were collected at the sites. The in-situ density of the backfill soil was measured using the core-cut method (JGS1613-2003).The values of in-situ wet density ρ_t at the two locations (No.1 and No.2) are almost the same (a little more than 1.5 g/cm^3). From the laboratory testing the void ratio of the soil was found to be less than 1.1. The grain size distribution showed that the soils were well-grained. It can also be said that the soils are well compacted. On the other hand, it is found that the filled soils inside the tires are found to be of higher density, and thus can be said that they were compacted better. The angle of internal friction of the sample soil (collected within 30 cm from the surface) of the backfill ground is 45.7°, while it is 48.2° for the soil within the tires. Therefore, it can be said that near the surface both soils have high value of internal friction, because the wet density ρ_t is quite high.

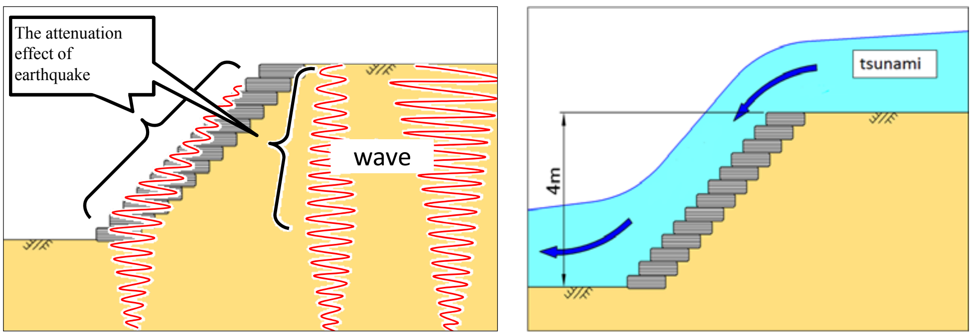


(a) H/V ratio (b) Transfer function

Figure 14. Micro tremor observations in Okirai

3.3.2 Reason for the non-damage

Figure 15 shows the state of the tire retaining wall at the time of earthquake and tsunami. As shown in Fig. 15(a), the tire retaining wall has the confining effect (Fukutake & Horiuchi, 2007) which made the wall strong against earthquake, and this could prevent any sliding failure or surface failure of the backfill soils. On the other hand, permeable and flexible structures like the tire retaining wall can reduce the earth pressures and water pressures during earthquakes and tsunamis. In addition, the earthquake motion could be attenuated because of the seismic isolation characteristics of the tire and that prevented the failure of the backfill ground. On the other hand, as shown in Fig. 15(b), tsunami entering from the backfill side damaged the building situated on the backfill ground and also eroded the natural slope opposite to the road parallel to the building. However, the tire wall could prevent damage to the backfill as well as the sloping side of the backfill from erosion. The flexible structure also could prevent any scouring of the wall at the bottom, a phenomenon which was prevalent in almost any sea walls, sea dikes, breakwaters and quay walls in many parts of Tohoku area due to the tsunami this time. The fact that tires are strong against scouring is evident from the fact that even during the backrush, the tsunami could not do any damage to the retaining wall. The hoop tension due to confinement, isolation effect and the anti-scouring effect is due to the high strength attributable to the hoop stress in each individual tire, and the high flexibility of the tire retaining wall.



(a) Earthquake (b) Tsunami

Figure 15. Behavior of the tire retaining wall at the time of earthquake and tsunami

4. CONCLUSIONS

In this research, field investigations and the laboratory investigations were carried out on two civil engineering structures, which show contradictory behavior in their seismic resistant and tsunami resistant characteristics as far as the level of damage is concerned. Based on the investigations, the

following conclusions could be drawn.
1) Dikes covered with concrete block in Yoshihama were damaged completely. The results show that subsidence related failure probably had taken place in the dike body with low soil density and high water table. Due to huge volume change resulting from liquefaction, the levee body subsided and this has led to the collapse of the structure. 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) The tire retaining wall was not damaged in spite of the loose state of the backfill. Damage to the tire retaining wall could be prevented by the earthquake due to confinement effect. In addition, the flexibility inherent in tires made the earth pressure and water pressure reduce during the earthquake and the tsunami. The seismic isolation effect was the reason behind such good performance.

After the great east Japan disaster brought by the gigantic earthquake in recent years, there has been a lot of attention on tsunami resistant characteristics of geotechnical structures. For example, development of ductile structures that can protect the structures without collapse even if a tsunami of such scale attacks the structure. The present research was limited only to two typical structures. However, the results will be useful in the development of new materials and techniques, which could prevent scouring, erosion and collapse of civil engineering structures in the future devastating earthquakes and tsunamis.

REFERENCES

Central Disaster Mitigation Council. (2003). Report of the 16th Committee Meeting, *Investigation Committee on Tonankai Earthquake and Nankai Earthquake*, Cabinet Office, Government of Japan (in Japanese).

Fukutake, K., and Horiuchi, S. (2007). Stacks of Tires for Earth Reinforcement Using Their Resistance to Hoop Tension and Land Reclamation Methods, *Scrap Tire Derived Geomaterials, Hazarika & Yasuhara (eds)*, Taylor and Francis, London, 205-214.

Furumura, T., Takemura, S., Noguchi, T., Takemoto, T., Maeda, K., and Padhy, S. (2011). Strong Ground Motions from the 2011 Off the Pacific Coast of Tohoku, Japan (Mw=9.0) Earthquake Obtained from a Dense Nation-wide Seismic Network, *Landslides.* **8:** 333-338.

Hara, T. (2011). Relationship between geotechnical disaster and building, Wood Industry, **66:11**, 492-497 (in Japanese).

Hara, T., Kokusho, T. and Hiraoka, R. (2004). Undrained Strength of Gravelly Soils with Different Particle Gradations, Proc. of the 13th World Conference on Earthquake Engineering, Paper No.144, 2004, 1-9.

Hara, T., Okamura, M., Uzuoka, R., Ishihara Y. and Ueno, K. (2012). Damages to River Dikes Due to Tsunami in South-central Coastal Area of Iwate Prefecture in 2011 off the Pacific Coast of Tohoku Earthquake, *Japanese Geotechnical Journal*, Japanese Geotechnical Society, **7:1**, 25-36 (in Japanese).

Hazarika, H. (2011). Infrastructural Damage and Lessons Learnt – A Review on the Aftermath of the March 11, 2011 Earthquake, *Keynote Lecture, Proc. of the 4th AUN/SeedNet Regional Conference on Geohazard Mitigation in ASEAN*, Phuket, Thailand, CD-ROM.

Hazarika, H., Kataoka, S., Kasama, K., Kaneko, K., and Suetsugu, D. (2012). Compound Geotechnical Disaster in Aomori Prefecture and Northern Iwate Prefecture due to the Earthquake and Tsunami, *Japanese Geotechnical Journal*, Japanese Geotechnical Society, **7:11**, 13-23 (in Japanese).

Imai, T., and Tonouchi, K. (1982). Correlation of N-Value with S-Wave Velocity and Shear Modulus, *Proc. of the 2nd ESPT*, 67-72.

Kyoshin Network K-NET (2011). National Research Institute for Earth Science and Disaster Prevention, *http://www.k-net.bosai.go.jp/*

MILIT (2011). Damage and Restoration of River and Coastal Structures due to the East Japan Disaster, *Water and Disaster Management Division, Tohoku Regional Bureau*, Ministry of Land, Infrastructure, Transport and Tourism, Sendai, Japan (in Japanese).

Nakamura, Y. (1989). A Method for Dynamic Characteristics Estimation of Subsurface Using Micro tremor on the Ground Surface, *Quarterly Report of RTRI*, **30:1**, 25-33.

Nishida. K., and Aoyama, C. (1984). Classification System of Decompose Granite soil in Undisturbed State, *Journal of Japan Society of Civil Engineers*. **352:III-50**, 159-168 (in Japanese).

Okada, K., Sugiyama, T., Noguchi, T., and Muraishi, N. (1992). A Correlation of Soil Strength between Different Sounding Tests on Embankment Surface, *Tsuchi to kiso*, **40:4**, 11-16 (in Japanese).

Shuto, N. (2011). History of Tsunami in Sanriku Area (Part 1): Information on Tsunami due to 2011 off the Pacific Coast of Tohoku Earthquake, *Joint Investigation Group of 2011 off the Pacific Coast of Tohoku Earthquake*, *http://www.coastal.jp/ttjt/index.php* (in Japanese).