Liquefaction-induced river levee failure during 2011 Great East Japan Earthquake: case history with Swedish weight sounding tests

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ABSTRACT
A case history of river levee failure observed at Sekiyado of Noda city, Chiba, during the 2011 Great East Japan Earthquake, is introduced. From the first look at the micro landforms of the area, the portion of the failed river levee is found located at an old river channel, and the other portions of the non-failed river levee are located at stable terraces. Therefore, it is expected that the portion of the failed river levee was built upon loose liquefiable sand deposits prevailing along an old river channel, and so a liquefaction-induced slip failure has occurred consequently. A series of Swedish weight sounding (SWS) tests were carried out, and soil sampling was also carried out for acquiring samples from the layer of liquefied natural sand. Based on the results of SWS tests and soil sampling, the slip failure observed along the river levee is discussed in detail.

1 INTRODUCTION
In the event of 2011 Great East Japan Earthquake that struck the eastern areas of Japanese archipelago on March 11, soil liquefaction effects were eminent especially in the reclaimed lands along Tokyo Bay as well as along Tonegawa River in Kanto region. Numerous numbers of infrastructures and private houses consequently suffered from severe damages, and river levees were also not the exceptions. From the report of Kanto Regional Development Bureau of MLIT, river levee failures were observed at more than 700 locations in Kanto region after the main shock, and they increased up to more than 900 locations after about 45 days of experiencing 6 major aftershocks.

The authors’ group has been performing geotechnical earthquake reconnaissance investigations with a useful tool, Swedish weight sounding (SWS) tests, at a number of sites including Itako city, Kamisu city and Naka-minato of Ibaraki, Katori city and Asahi city of Chiba (Kawabe et al. 2012, Tsukamoto et al. 2012a, 2012b, 2012c, Kanemitsu et al. 2013, Tsukamoto et al. 2013, Tsukamoto et al. 2015). The authors’ group is currently engaged in investigating liquefaction-induced river levee failures. In the present study, a case history of river levee failures observed at Sekiyado of Noda city, Chiba, is introduced, where a series of Swedish weight sounding tests and soil sampling are conducted.

2 GENERAL OUTLOOK OF RIVER LEVEE FAILURE AT SEKIYADO OF NODA CITY
A river levee failure was observed after the main shock of 2011 earthquake on the left bank of Edogawa River at Sekiyado of Noda city, Chiba. It is certain that this particular failure took place during the main shock based on local eye-witness accounts. A large major crack, as well as minor cracks, was induced over a length of 200 metres as shown in Fig. 1. The large crack was found associated with liquefaction-induced sand boils observed at the foot of the river levee.

Fig. 2 shows the micro landforms observed around this area. The micro landforms of this area appear to have developed under the influence of the two main rivers of Tonegawa and Edogawa. The diverging point of these two rivers is located north of the failed river levee. In between the geotechnically stable terraces, pairs of natural levees and flood plains prevailed. Some old river channels are also present, along which loose liquefiable sand deposits may be present. It was found that the particular location of the failed river levee lies on an old river channel, and most of the other non-failed river levees lie on stable terraces. Therefore, from the viewpoint of micro landforms, it is certainly expected that this particular river levee failure would have been induced by liquefaction of a liquefiable sand layer accumulated along an old river channel underlying a river levee itself.

Fig. 3 shows the plan and elevation views of the failed portion of the river levee. The large crack seems to form a top scarp of a liquefaction-induced slide, beneath which there are subsided and heaved portions. The liquefaction-induced sand boils were found around the boundary of the subsided and heaved portions.

3 SWEDISH WEIGHT SOUNDING TESTS AND SOIL SAMPLING
The authors’ group has been using Swedish weight sounding (SWS) tests for earthquake reconnaissance investigations. The testing procedure and the interpretation of test results are described by Tsukamoto et al. (2004). The testing procedure basically consists of two phases; static penetration and rotational penetration. The static penetration is the phase where the
screw-shaped point attached to the tip of the rod is statically penetrated by dead weights amounting up to 980 N (100 kgf) without any forcible rotation of the rod, and the value of $W_{sw}$ (kN) denotes the total loads taking values of 0.15, 0.25, 0.5, 0.75 and 1 kN. When the static penetration comes to a halt, the rotational penetration is resumed. The horizontal handle attached to the top of the rod is rotated, the value of $N_a$ is counted, which is the number of half a turn necessary to penetrate the rod through 25 cm. The value of $N_a$ is then converted to the value of $N_{sw}$ (ht/m), by multiplying $N_a$ by 4.

Some drawbacks of SWS tests mainly lie in the following two points. One is the difficulty in determining precisely the depth of groundwater, and the other is the difficulty in classifying the soils and soil strata in which the tip is in during the SWS tests. In the present study, in order to overcome these drawbacks, soil sampling was carried out with appropriate equipment.

Swedish weight sounding tests and soil sampling were carried out at the location of the river levee failure on September 30, and November 18 and 19, 2015. Along section A – A’ shown in Fig. 3, three series of SWS tests were carried out. Fig. 4 shows the results of SWS tests along with the estimated soil profile based on the existing borehole data. It is known that fill was placed on top of the river levee recently in 2007, as shown in Fig. 4. The old fill dated back to the period from 1958 to 1980. The deposits of natural sand underlying these fills are most likely to have liquefied, as evident by the sand boils at the foot of the river levee. Due to the liquefaction of the underlying natural sand deposits, the overlying reclaimed fills have lost stability and have slid along the slip failure planes as shown in Fig. 4. There would have been two possible scenarios of sliding failures, assuming two different slip failure planes. One is certainly the slip plane passing through the liquefied natural sand deposits, and the other is the slip plane passing through the boundary between the most recent and old fills.

In the present study, in order to overcome one of the drawbacks of SWS tests, soil sampling was carried out. From the viewpoint of operational robustness and reliability of sampling, the method of soil sampling adopted in the present study was found most relevant for use with SWS tests. Fig. 5 shows the procedure of soil sampling. The sampler of 30 cm long and 2.5 cm in inner diameter was pushed into the ground until the desired soil deposit was met. The inner rod encased in the sampler was then pulled up and fixed. The sampler fixed with the inner rod was slowly pushed into the desired soil deposit, and a soil sample is captured into the sampler. The sampler fixed with the inner rod is then pulled up to the ground surface. The grain size distributions of the two soil samples retrieved from the deposits indicated in Fig. 4 are shown in Fig. 6. The natural sand deposits that have liquefied were found to contain some amount of fines less than 0.075 mm.

From the grain size distribution of the soil sample obtained from the location of Sw-2, the amount of fines
content less than 0.075 mm $F_c$ was 25.1 %. The mean particle diameter $D_{50}$ was 0.155 mm. It is then possible to estimate the value of void ratio range, based on the empirical relation between $D_{50}$ and $e_{max} - e_{min}$ (Cubrinovski and Ishihara 2004). The void ratio range was estimated as $e_{max} - e_{min} = 0.65$. The values of relative density $D_r$ can then be estimated as follows, (Tsukamoto et al. 2004).

\[
D_r = \sqrt{\frac{(N_{sw} + 40)(e_{max} - e_{min})^{2.2}}{90}} \frac{98}{\sigma'_v} \quad [1]
\]

From the recent development of use of SWS tests, it is also possible to estimate the liquefaction resistance of soil deposits from the results of SWS tests, by using the following empirical equations (Tsukamoto et al. 2016),

\[
R_I = 0.016 \sqrt{N_{sw1}} = 0.016 \sqrt{(N_{sw} + 40 \times W_{sw})} \frac{98}{\sigma'_v} \quad [2]
\]

\[
R_I = 0.02 \sqrt{N_{sw1}} = 0.02 \sqrt{(N_{sw} + 40 \times W_{sw})} \frac{98}{\sigma'_v} \quad [3]
\]
where Eq. 2 is for clean sand and Eq. 3 is for sand with silt and silty sand, and $\sigma'_v$ is in kPa.

The distributions of relative density $D_r$ and liquefaction resistance $R_l$ estimated for the liquefied soil layer are shown in Fig. 7.

From the acceleration records observed at Noda station of K-Net which is the closest to the site of the river levee failure, the earthquake motion lasted for over 90 seconds, and the maximum accelerations in the N-S, E-W and UD components are 321, 374 and 141 Gals, respectively. K-Net is the strong-motion seismograph networks operated by National Research Institute for Earth Science and Disaster Resilience in Japan. It is quite certain that the corresponding layer of natural sand has liquefied based on the empirical procedure of evaluating liquefaction triggering from the liquefaction resistance of soils and earthquake accelerations, by calculating the cyclic resistance ratio from Eq. 3 and also by calculating the cyclic stress ratio assuming the ground surface acceleration of 374 Gals, for example. It is then found that the factor of safety against liquefaction should have been less than $F_l = 0.4$.

Herein, another series of SWS tests was carried out at the location where collapse and crack had not been observed during 2011 earthquake, as shown in Fig. 8. It was found that there were some loose soil layers, which exhibited low values of SWS penetration resistance almost equivalent to the values of $N_{sw}$ observed at the collapsed portion of the river levee shown in Fig. 4.

The key difference between the collapsed portion and non-collapsed portions of the river levee can be found on their corresponding micro landforms, where the collapsed portion is located on the old river channel and the non-collapsed portion is located on the terraces, though their values of SWS penetration resistance were almost identical. The ageing effects might have come into effect on the liquefaction triggering as well as the mobilisation of undrained shear strength of soils, which needs to be further resolved in future study.

4 CONCLUSIONS

The river levee failure observed at Sekiyado of Noda city,
Chiba, during the 2011 Great East Japan Earthquake was presented as a case history. From the viewpoint of the micro landforms, since the portion of the failed river levee was located at an old river channel while the other portions of the non-failed river levee were located at stable terraces, the portion of the failed river levee appeared to have built upon loose liquefiable sand deposits prevailing along an old river channel. Hence, the liquefaction-induced slip failure has occurred consequently. The ageing effects might have come into effect on the liquefaction triggering as well as the mobilisation of undrained shear strength of soils, which needs further study to be resolved. Based on the results of Swedish weight sounding tests and soil sampling, the slip failure planes were assumed and the liquefaction triggering of natural sand deposits were examined. It should be emphasized that the soil sampling technology used in the present study was found quite reliable and suitable for use in combination with Swedish weight sounding tests in retrieving soil samples from liquefiable soil layers. The results of Swedish weight sounding tests can be analysed together with the grain size distribution data to estimate the liquefiability of soil deposits.

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