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# BEHAVIOR OF LIQUEFIED SOIL NEAR BOUNDARY OF BACKFILLED BASIN DUE TO MAIN AND FOLLOWING MUXIMUM AFTERSHOCK

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**ABSTRACT**: Reclaimed lands backfilled by the remainder which collected iron content in the iron ore sand deposits were liquefied in Asahi City, Chiba Prefecture due to the 2011 off the Pacific coast of Tohoku Earthquake. Based on the site investigations and the numerical analysis, the behaviors of the liquefied soil at the brinks of the backfill basins were classified into three types, namely heave, cave-in and landslide. The mechanism of these behaviors was attributed to the relationship between the predominant direction of displacement of liquefied soil and the location of the excavation boundary.

**Key Words**: Great East Japan earthquake, iron ore sand, liquefaction, upheaval, excavation boundary, interview, damage, numerical analysis

## INTRODUCTION

Foundations liquefied at various places in the Kanto district due to the 2011 off the Pacific coast of Tohoku Earthquake (MLIT and JGS 2011). Most of them are the lands reclaimed after 1965 along the coast of Tokyo Bay, old river channels and flood plains of the Tone River, developed lands at old ponds and marshes reclaimed by dredge sand. In addition, severe liquefaction occurred here and there at reclaimed lands backfilled by discarded sand from iron ore production at the iron ore deposits in Asahi-city, Chiba prefecture.

Though it is well known that the typical soil behaviors due to liquefaction are the land slide at a slope, the lateral flow near quays, subsidence of horizontal ground, sink of heavy structures and floating up of light underground structures such as a sewage tank (Onoue et. al. 2011), rise and cave-in of the ground and wave-like undulations were also conspicuous in addition to those in Asahi-city during the present earthquake. This paper reports investigation results about behaviors of foundation due to liquefaction observed at old iron ore deposits in Asahi-city and new geotechinical findings obtained through a numerical analysis on the liquefaction.

## OUTLINE OF INVESTIGATION

### Locations of iron ore sand deposit and liquefied areas

The areas where liquefaction damage was remarkable in Asahi-city are shown by the ellipses in Fig.1. The iron ore sand deposits located in old Iioka-town are described in the History of Iioka-town (Iioka-town 1981). Although all the locations of these iron ore sand deposits were indicated in Fig. 1 with broken lines, some of them have not been excavated in the past as a mine. Moreover, although there are many places where a liquefied area does not overlap with the position of the ore deposit on a map, at least all the liquefied lands were reclaimed by iron ore sand according to the reports of the earthquake victims. The areas where iron sand has been excavated are, therefore, more than those indicated in Fig. 1 including ellipses indicated in old Asahi-city (left half of Fig. 1).

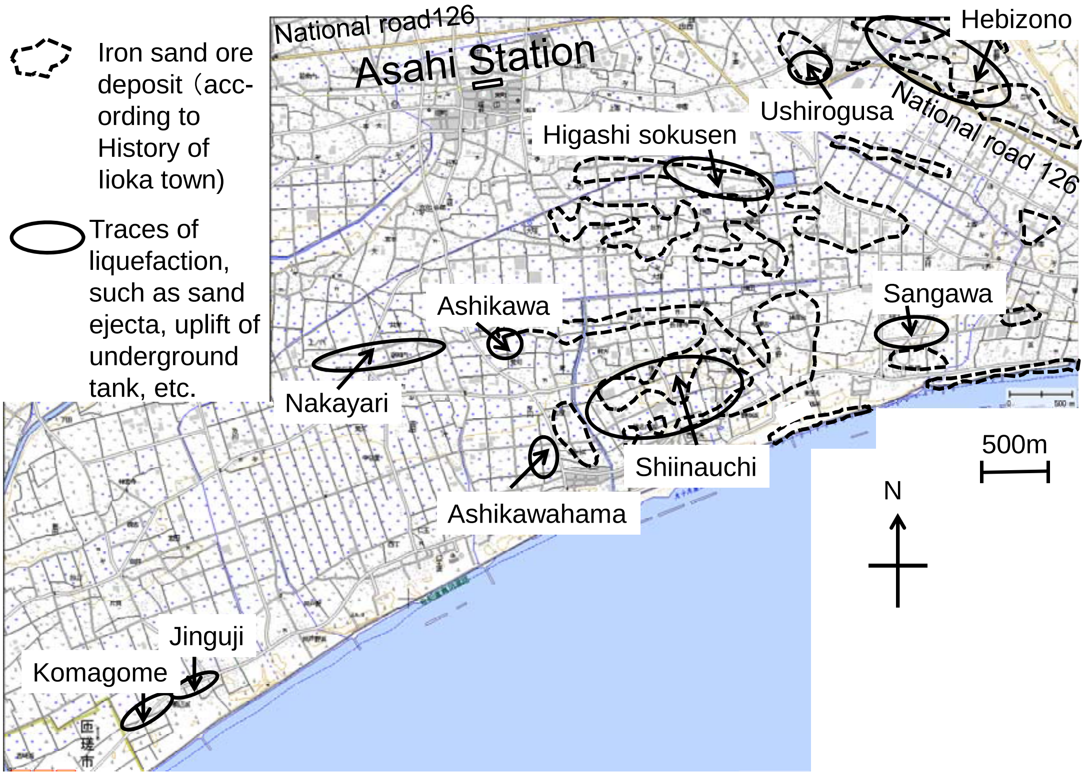



Fig.1 Liquefied areas and distribution of iron ore sand deposits in Asahi city

### Summary of reconnaissance and following investigation

Deformations and cracks of foundation, subsidence and tilting of buildings and telegraph poles, uplift of underground structure, sand ejection, etc. were observed on-site. Interview with victims, land surveys of deformed foundation, in-situ soil investigation through Swedish Weight Sounding (SWS), and laboratory tests on the intact samples taken on-site were performed. The time when liquefaction occurred was inspected through a two dimensional liquefaction analysis based on the laboratory test results.

## RESULTS OF INVESTIGATION

Out of many liquefied areas shown in Fig.1, a close view of the Ushirogusa and Hebizono areas where quite a few houses suffered great damage due to liquefaction are shown in Fig. 2. The Nakasone zone in the Hebizono area shown in the region surrounded by the solid line was a residential area developed on the land backfilled after iron ore mining. This zone suffered such devastating damage that 40 houses out of totally 48 houses were judged to be totally destroyed. According to explanations given by both owners of excavated land and unexcavated land, the boundary of the excavated and backfilled area is the solid lines in Fig. 2.

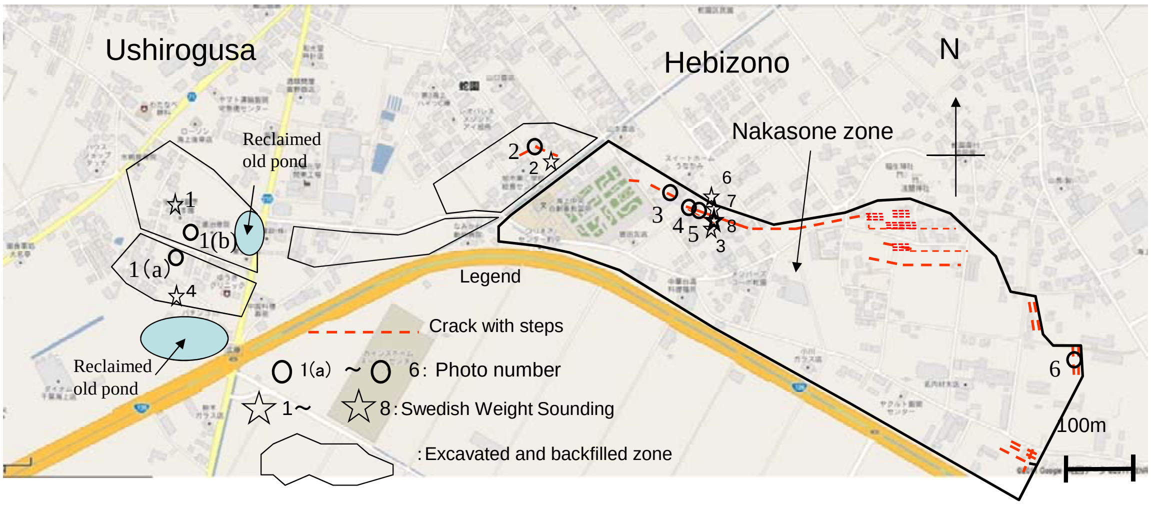



Fig.2 Location of soil investigations, photos and remarkably damaged foundations

### Ushirogusa area

Photo 1(a) shows an example of vast sand ejected due to the main shock and Photo 1(b) shows a trace of ejected sand remaining on the wall. The trace indicates that the thickness of the ejected sand was 60 cm here. The grain size distributions of liquefied sand sampled at Ushirogusa and Hebizono are shown in Fig.3. It shows markedly inferior distribution of fine sand having a uniformity coefficient of 2. Figure 4 shows equivalent SPT N-value converted from the results of SWS conducted at the Ushirogusa and Hebizono areas. According to this figure, the depths of water table in this area are about 1.3 m and the excavation depths of iron sand ore ranges from 3.5 m to 4 m below the ground surface. The thickness of liquefied sand portion appears to be as thin as 2 to 3 m.


(a) Spouted sand (25 minutes after the main event, Photo: Ikeda)

(b) Trace of sand spout

Photo 1 Sand ejecta and its trace observed at Ushirogusa area

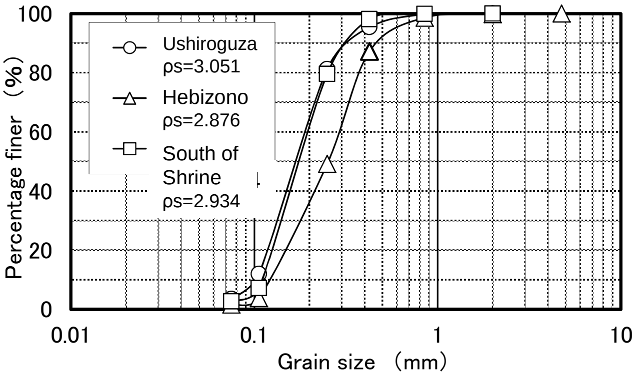



Fig.3 Grain size distribution and specific gravity of soil

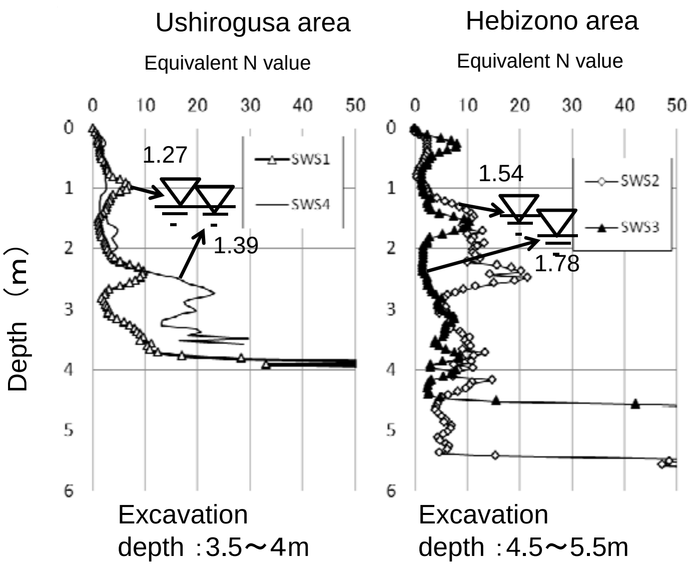



Fig.4 Equivalent N-values based on the result of SWS

**Nakasone zone in Hebizono area**

***Upheaval of land***

Photo 2 shows the site to the north of SWS No.2 location where sand spout was observed during the largest aftershock. Although no damage was seen regarding the ground in the near side of this photo, a crack with a level difference appeared along the line connecting the entrance terrace and the car, and the ground heaved up at far side of the line. This site is located at the upper left corner in Fig. 6 with its enlarged map. The results of level survey were shown along the three survey lines, A –A', B – B' and C – C' in Fig. 6. Photo 2 was taken at Point B viewing Point B'. According to the survey results along B – B' and C – C sections, the ground rose by 60 cm and by 38 cm, respectively, in the right side of the crack. The direction of the ridge of upheaval is almost the east - west. The ridge is parallel to the boundary between the excavated and the unexcavated lands, and the distance between them is about 30 m. The down-ward arrows, (↓), noted as "Excavation boundary" in Fig.6 show the estimated locations of the excavation boundary. The white arrows show the estimated predominant directions of liquefied sand motion. For instance, the excavation boundary is located in the direction of A'-B'-C' side for this residential land in Photo 2. Rise of the soil liquefied at the inside of the boundary means that liquefied sand at A-B-C side is considered to have moved mainly to the direction of the boundary. Cave-in means, to the contrary, that the liquefied sand moved mainly in the direction which keeps away from the boundary.


Photo 2 Upheaval of housing lot at Hebizono area

Photo 3 shows a ground deformation near the boundary. The waterway, the retaining wall on the left and the rice field on the left in far side in the figure were not damaged because they are located in unexcavated part. To the contrary, the ground heaved up and the waterway was collapsed and disappeared on near side which is located in the excavated and backfilled part. Photo 4 shows a house which was greatly bent into a mountain shape since the site heaved up near the middle of the house.


Photo 3 Boundary between unliquefied foundation and damaged foundation


Photo 4 Destroyed house due to ground upheaval at the center of the house

The cultivated field shown in Photo 5 is located in the excavated side and the excavation boundary is located on the left side of this photo. Although this field had been flat and no sand spout was observed during the main shock, it heaved up with ejects of sand during the largest aftershock at approximately 15:15. Photo 5 shows the middle and upper steps having a height difference of 60 cm. According to E – E' section in Fig. 6, elevations of the middle step and the upper step were 25 cm and 85 cm higher than that of Point E on the lower step which has not liquefied because of belonging to the unexcavated side. Figure 5 shows the equivalent N-value along E – E' section. The unexcavated ground is so stiff at depths deeper than 2 m that the equivalent N-values are larger than 40. To the contrary, the backfilled sand is so loose that the equivalent N-values are less than 10 in general to the depth of 6 m to 6.5 m, though they exceed 15 at some depths.

The mechanism of the ground upheaval all at once at the inside of the excavation boundary is considered to be as follows: The foundation had not reached near complete liquefaction with sand ejection, though the excess pore water pressure increased to considerably large. It liquefied almost completely and the sand was ejected during the following largest aftershock. When the liquefied soil moved toward the un-excavated foundation, it heaved up because it was blocked in the hard stratum and rose. It is imagined that the ground heaved up to the broken line indicated in Fig.5 just after rising and subsided to the solid line with increasing elapsed time. Along the survey line D – D', the maximum value of upheaval was 80cm here.

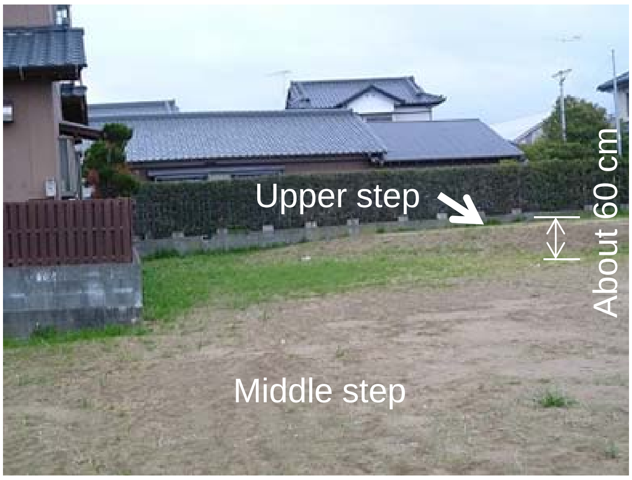



Photo 5 Up-heaved field due to liquefaction of soil

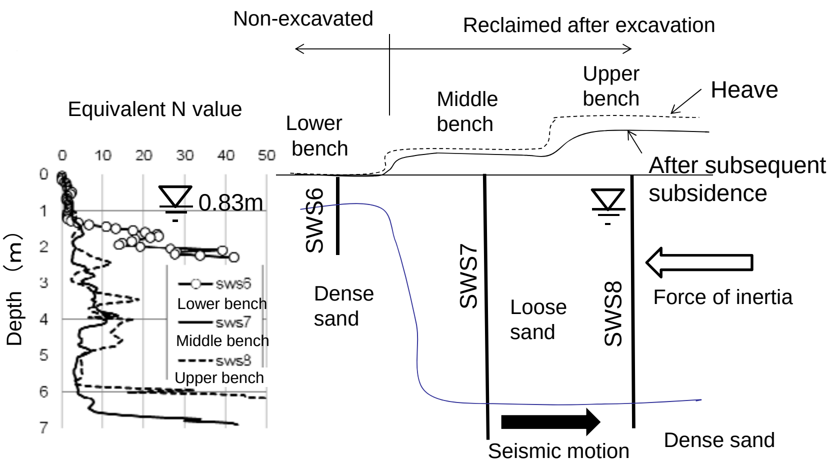
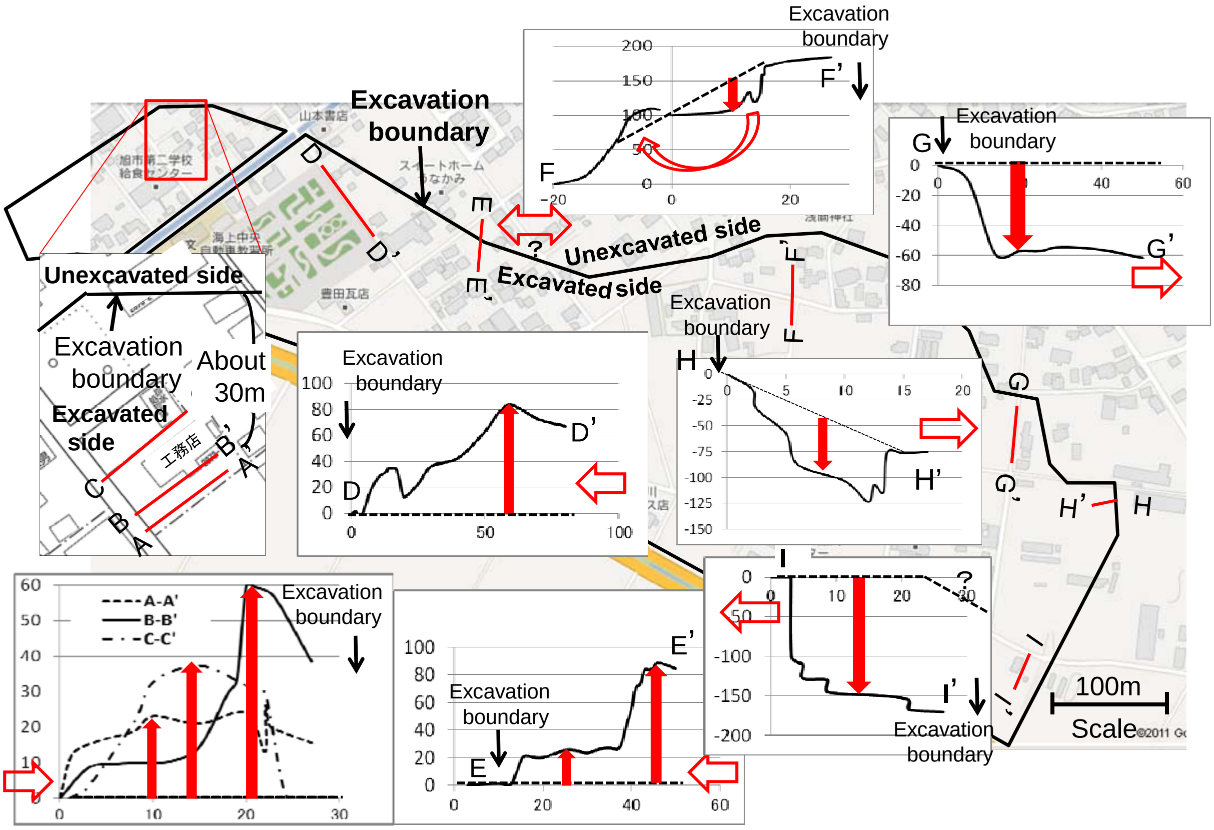



Figure 5 Schematic picture of upheaval mechanism




Unit: vertical axis [ cm ], horizontal axis [ m ] : Predominant displacement direction of liquefied soil

Fig.6 Upheaval, cave-in and slope failure due to liquefaction at the periphery of backfilled discarded iron ore sand (Nakasone zone in Hebizono area)

***Landslide***

Along F – F’ survey line in Fig.6, the slope failure with the main scarps were observed as indicated in the schematic picture. A tremendous amount of ejected sand was observed in the field around Point F. Therefore, the slope failure is considered to be caused by the liquefaction of foundation.

***Cave-in 2***

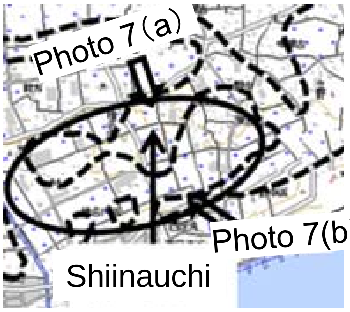
Although no damage to house or ground were observed outside the boundary along the survey line G – G’, the foundation settled by 60 cm at the inside of the boundary as indicated in Fig.6. This subsidence at the inside of the boundary was considered to have occurred because the liquefied layer at the inside of the boundary moved mainly in the opposite direction to the boundary.

Photo 6 shows cave-ins along the survey line H – H’. As this is the view from north side, H’ and H are located in the left and right sides of this figure, respectively. As seen in the figure, no foundation damage was observed at the left (east) end of this figure, because that area was located in the unexcavated side. However, the severe cave-ins with several openings are observed in the excavated side. This cave-in was considered to be caused by the movement of the liquefied soil in the excavated side toward the opposite direction of the boundary.

Along the survey line I – I’, a main scarp whose height difference was about 1 m appeared at Point I in Fig.6. Furthermore, several openings and gaps were created in the housing lot, resulting a total subsidence of 170 cm in height along the line I - I’. A purified water tank of the sewage heaved up and many traces of sand eject were observed in the neighboring yard. This cave-in was considered to be caused by the combined effect of landslide due to liquefaction of the foundation and movement of the housing lot toward the opposite direction to the excavation boundary.

**Shiinauchi area**

Figure 7 shows the excavation boundary and the liquefied zone at Shiinauchi area. The apartment house in Photo 7(a) was located at the inside of the excavated area and the road was located outside of it. The apartment house undulated and deformed, and the parking lot heaved up by 60 cm, though no damage was observed in the road. The dominant displacement direction of liquefied soil here is considered to be toward the road though the liquefied soil was hindered its movement by the stiff layer under the road and heaved up. To the contrary, Figure 7(b) shows a cave-in occurred at the opposite boundary in the Shiinauchi area. The foundation here was considered to have moved mainly toward the opposite direction to the excavation boundary.





(a) Upheaval at north boundary (b) Cave-in at south boundary

Fig.7 Map of Shiinauchi area. Photo 7 Upheaval and cave-in at the inside of the excavation boundary

**Time when liquefaction occurred**

According to the results of interview to 28 victims, there are 5 houses where the sand spout was observed during and just after the main event started 14:46 on March 11 and 9 houses where it was observed the largest aftershock started 15:15, excluding 2 cases which were located on the unexcavated ground and 12 cases where the victims were absent during the events. This result was different from those at Mihama-ku, Chiba-city where all victims observed sand spout prior to the largest aftershock, excluding unclear cases.

## NUMERICAL INVESTIGATION OF OCCURRED LIQUEFACTION

**Dynamic properties of liquefied sand**

The intact samples were taken at the point, SWS 7, by penetrating short thin-wall samplers with a length of 25 cm into the sand layer after removing the surface soil with a thickness of 80 cm at the middle step of the agricultural field shown in Fig.5. These samples are called “Hebizono”. Other intact samples were taken below a depth of 80 cm in the same manner at Point F’ shown in Fig.6. These samples are called “South of Shrine”. The liquefaction strength and the dynamic deformation properties were shown in Fig.8 and Fig.9, respectively. According to Fig.8, liquefaction resistance of the soil at Hebizono is larger than that at South of Shrine.

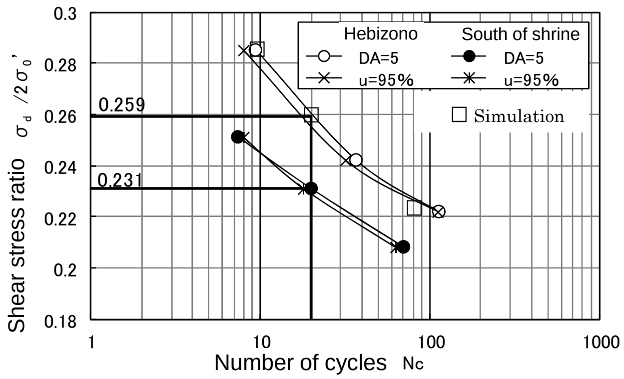



Fig.8 Liquefaction strength of excavated and backfilled iron sand (Hebizono zone)

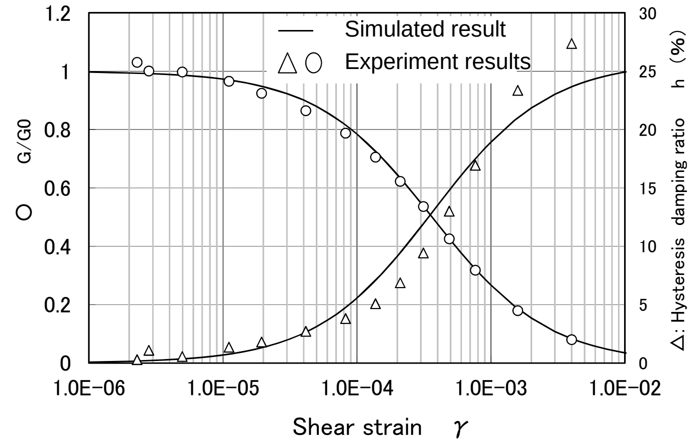



Fig.9 Dynamic deformation properties of excavated and backfilled iron sand (Hebizono zone)

**Outlines of two-dimensional fully-coupled dynamic effective stress analysis**

Two-dimensional fully-coupled dynamic effective stress analysis software UWLC (Forum 8 Co. Ltd. 2005) was used in this paper. The software UWLC consists of three parts: (1) a program used to identify the parameters of constitutive laws by simulating the laboratory tests, (2) a program to calculate the initial effective stress before the earthquake, and (3) a fully-coupled dynamic effective stress analysis program to calculate the seismic response. The derivation of finite element formulae on the fully-coupled effective stress analysis was detailed in the Manual of UWLC. Rayleigh damping with a damping coefficient of 0.02 was also introduced. The time increment of 0.001 s was used.

**Parameters for constitutive models**

In this paper, the generalized plasticity model for sand developed by Pastor et al. (1990) and modified in some aspects (Cai et al. 2002) was used for the excavated and backfilled iron sand, and Hardin-Drnevich model was used for the unexcavated iron sand. The laboratory test results were used to identify the parameters of the generalized plasticity model for the excavated and backfilled iron sand. The values of parameters were shown in the previous paper (Onoue et.al. 2012). As shown in Fig. 8, the simulated liquefaction strength using the identified parameters was consistent with the laboratory results. The parameters of Hardin-Drnevich model for unexcavated iron sand were determined using the dynamic deformation test results shown in Fig. 9, here initial shear stiffness $G_0$ was estimated using N-value of the standard penetration test and a statistic equation.

**Analyzed cross section and input motion**

Cross section E-E’ proximately in the north-south direction was selected in the numerical investigation, and Fig. 10 shows finite element mesh. The thickness of the analyzed ground was 8 m based on in-situ investigation. The width of the left and right un-excavated grounds were respectively 100 m, and the width of the excavated and backfilled iron sand ground in the middle of the mesh was 200 m, of which the upper soil layer 6m thick was excavated and backfilled iron sand and the lower soil layer 2m thick was unexcavated layer. Points A and B located at a depth of 3m from the ground surface, the horizontal distance from Point A to the excavation boundary was 8m, and Point B located at the middle of the excavated and backfilled iron sand ground. The groundwater level was at a depth of 1m below the ground surface.

Seismic wave at 14:26:00 of the main shock and at 15:15:00 of the largest aftershock in NS direction observed at K-Net Yokaichiba station (CHB010) was used as input motion. Fig.11 shows the epicenters of the main shock and aftershock as well as the location of Asahi city. Because of the time limit of numerical analysis, the time interval between the main shock and the largest aftershock was shortened as shown in Fig. 12. Moreover, preliminary analysis was carried out to adjust the input motion until the calculated maximum acceleration at the ground surface was consistent with the observation.

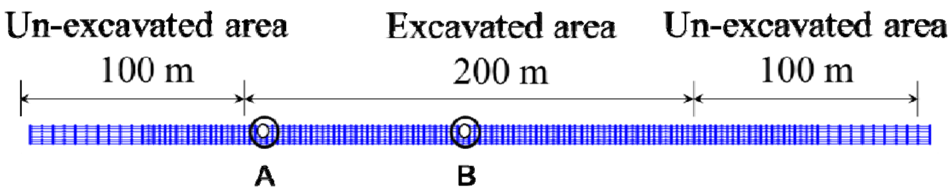



Fig.10 Finite element mesh with a thickness of 8m

**Numerical results**

Figure 13 shows the time histories of excess pore water pressure ratio at points A and B. At Point A, the excess pore water pressure ratio increased to 0.64 during the main shock and kept almost constant after the main shock. It reached then 0.95 during the largest aftershock. In contrast, at Point B, the maximum excess pore water pressure ratio during the main shock reached 0.95, indicating almost liquefaction. It lowered to 0.85 after the main shock and then increased once again to reach complete liquefaction. The numerical results support the testimonies of eyewitness that sand boiling occurred during the main shock at some places and during the largest aftershock at other places.

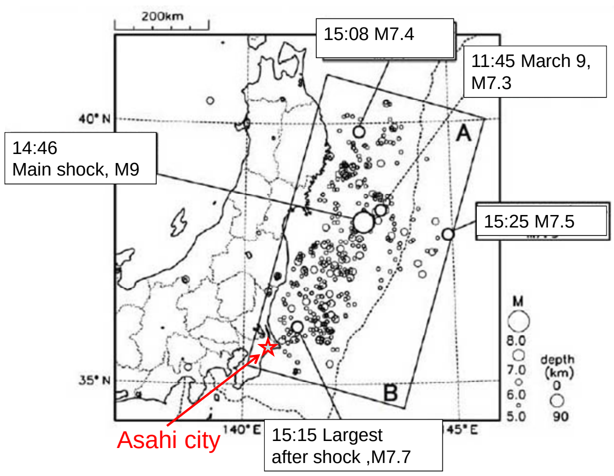



Fig.11 Location of Asahi-city and epicenters

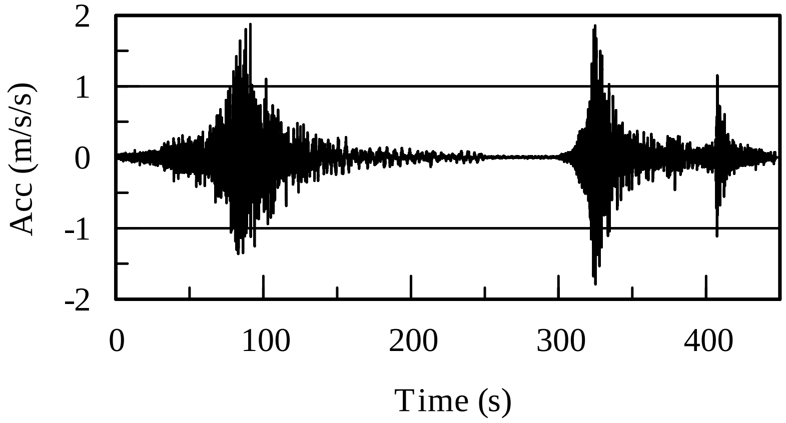



Fig.12 Input motion of main shock and the largest aftershock
(t=0s corresponding to 14:47:06, t=250s to 15:15:00, 11 March, 2011)

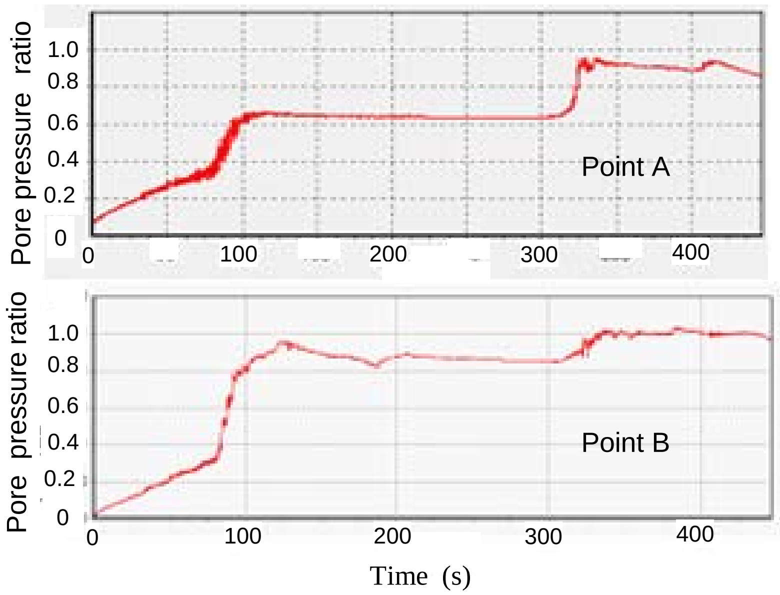



Fig.13 Time histories of excess pore water pressure ratio at points A and B shown in Fig.10

## CONCLUSIONS

The reclaimed lands which were backfilled with discarded iron ore sand were liquefied at various areas and many residential houses were severely damaged in Asahi-city, Chiba-prefecture, due to the 2011 off the Pacific coast of Tohoku Earthquake. Among these areas, the detail investigations, such as victim interview, soil investigation, land survey, etc., and the numerical analysis on the liquefaction were carried out at Nakasone zone in Hebizono area. As a result, following conclusions were obtained with regard to damage to foundation.

1) The backfilled sand having a thickness of 5 - 6 m was liquefied at the basin-like reclaimed land of about 250 m in width and about 1 km in length. Along the periphery of the reclaimed land, the liquefied soil up-heaved when it displaced toward the excavation boundary and caved-in when it displaced to the opposite direction to the boundary. In the case of slope, landslides often happen accompanied by such upheaval or cave-in.

2) According to the victim interviews, the number of witnesses who confirmed liquefaction by sand spout during and after the main event is larger than that of witness who confirmed it during and just after the largest aftershock starting 29 minutes later. This is a different feature from the time of liquefaction occurrence along the coast of Tokyo bay, such as Chiba city, where the liquefaction occurred during the main event.

3) It was confirmed analytically based on the wave record and the soil properties that there might be two cases that the liquefaction occurred during the main event and that it occurred during the largest aftershock.

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