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SUBJECTS OF THE LIQUEFACTION RESEARCH SEEN TO THE LIQUEFACTION DAMAGE OF THE GREAT EAST JAPAN EARTHQUAKE DISASTER

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ABSTRACT: This paper discusses the new subjects of the liquefaction research regarding the liquefaction damage of the Great East Japan Disaster. Specifically, the subject about the liquefaction potential assessment, the subject about the evaluation of ground subsidence caused by liquefaction and the subject seen to generate the liquefaction damage of a river levee, etc. are discussed based on the damage observed or the experimental results. It has been shown that understanding current damage on the extension of the present technology has a limit, and that the prediction and countermeasure technology of the liquefaction damage based on a new concept is necessary.

Key Words: Earthquake, liquefaction damage, ground subsidence, liquefaction potential assessment, fines content, duration of earthquake motion, river levee

1. INTRODUCTION

Looking back on the damage caused by the Great East Japan Earthquake, it gives an impression that the structural damage of the reinforced concrete (RC) constructions, ex. viaducts' pillars, was not so serious, whereas the extensive damage by liquefaction in the Tokyo Bay area was very severe due to its intensity of the earthquake motion. While one of the reasons of less damage on those RC structures might be that the intensity of the earthquake motion was weaker than that of the Great Hanshin-Awaji Earthquake in 1995, it might be evaluated as an achievement of promoting aseismic technology in contrast to the damage of the Great Hanshin-Awaji Earthquake. On the other hand, the reason of severe liquefaction damage cannot be found out only in the external forces resulting from the long duration of a gigantic earthquake. For example, in the case of the New Zealand Canterbury earthquake that occurred in February 2011, although the magnitude was small and the earthquake motion was short in duration, it caused liquefaction damage as wide as that of the Tokyo Bay area after the Great

East Japan Earthquake. This might show that technological innovation on liquefaction countermeasures after the Great Hanshin-Awaji Earthquake have not lead to visible damage reduction. The first author of this paper recalls that even immediately after the Great Hanshin-Awaji Earthquake in 1995, concerns arouse regarding that the long duration of the earthquake motion caused by the maritime earthquake of large magnitude would cause even greater damage by liquefaction.

Today there are no engineers and researchers involved in the construction industry, and geotechnical engineering especially, who are not aware of the liquefaction phenomena. In the wake of the Alaska earthquake and the Niigata earthquake in 1964, researchers from Japan and the United States studied it vigorously, and by the time of the Great Hanshin-Awaji Earthquake in 1995, knowledge of the liquefaction phenomenon, the technology for its prediction and various countermeasures have been incorporated into practice to some extent. However, compared with the earliest days, although somehow it has become possible to deal with practical problems, there are still a lot of issues that we do not understand about liquefaction and there are still many points to be improved in practice. The first author of this paper has written about development of liquefaction research after the Great Hanshin-Awaji Earthquake[1,2)]. Regarding liquefaction damage in the Great East Japan Earthquake, there is a necessity for further technology development, but in the present circumstance, we do not see a new approach emerging from the conventional techniques and frameworks. This paper, in order to develop technology for predicting and reducing liquefaction damage, points out some of the issues which cannot be handled on the extension of present technology, with examples of current liquefaction damages.

2. SUBJECTS ON THE LIQUEFACTION PREDICTION METHOD AND THE EVALUATION OF RESIDUAL DEFORMATION OF THE GROUND – LIQUEFACTION JUDGMENTS NOT BEING ABLE TO GET AWAY FROM THE WORLD OF SAFETY FACTOR

2.1 Necessity of evaluation of material’s ductility

The greatest lesson from the Great East Japan Earthquake is that nothing is absolutely safe. In the event of a gigantic tsunami, it is not possible to design a power plant structure with absolute safety. It is a common perception of engineers and researchers that a framework for aseismic design and tsunami-resistant design should be constructed on the premise that there are possible external forces which could go beyond what was expected. Nevertheless, today’s liquefaction potential assessment is still focusing on determining whether liquefaction will occur or not, by basing liquefaction safety factor (FL) on whether it surpasses unity or not. This method is not more than a simple primary screening which excludes those cases where liquefaction cannot occur; therefore, further development is nowadays required.

This recognition has also been pointed out at the time of the Great Hanshin-Awaji Earthquake, and it seems to be taken into account within the framework of performance-based design. This type of design makes safety evaluations by estimating the amount of deformation through numerical analysis. The accuracy of this numerical analysis depends greatly on the modeling of ground material; and the amount of deformation depends on the deformation characteristic settings of the soil material subjected to the cyclic load. In order to improve the accuracy of the high-grade analysis, it is necessary to make an evaluation which reflects the deformation characteristics of the soil. This evaluation cannot be performed using only the N-value and the liquefaction strength R. Unfortunately, as it was pointed out firstly, the performance-based design has not proved to be very effective, since the current undrained liquefaction strength analysis is unable to evaluate the process leading to liquefaction of the soil material, subsequent deformation characteristics, and ductility.

After the Great Hanshin-Awaji Earthquake, in the evaluation of the seismic performance of the RC column, the ultimate load and ductility of the structural materials are evaluated by a cyclic loading test with a strain which exceeds largely the elastic range. Several reinforcing methods for imparting ductility were developed and implemented. On the other hand, with regard to the evaluation method of liquefaction resistance of soil, improvements for active evaluation of ductility against liquefaction have been rarely seen in practice. Assuming a large shear stress ratio caused by a strong earthquake motion, the soil can easily cause shear failure, and liquefaction can occur even in the ground with an N value of 30. Therefore, it falls into a situation where the assumed seismic load has become too large to design any ground improvement. An N value of 30 is almost the maximum value that can be achieved by compaction of sandy soil. Without evaluating the ductility of soil, the ground cannot withstand external forces of level 2 earthquake motion. The problem is not whether liquefaction happens or not, but, whether the ground can sustain limited deformation and to what extent can the soil persist even after the liquefaction. In order to overcome such situation, first of all, a definition of liquefaction, which defines failure as the development of shear strain of 5% and the increase in excess pore water pressure, should be reconsidered at the element level. Not all of the soil will liquefy and fail at once. When the excess pore water pressure ratio reaches 100%; because of the high sand density, or high soil ductility, the soil recovers effective stress with a little deformation and regains stiffness. In some cases, numeral calculations have shown results that element level strain can exceed several 10% to 100%; however, it has been pointed out that the assessment of material properties has to comply to those strain level in order to claim the calculations' validity[5)].

In the example of evaluating the earthquake resistance of RC columns, ductility is evaluated based on the envelope curve of the bearing load from the displacement controlled test which applies cyclic deformation up to about 10 times the strain that exerts the yield point, and the energy absorption capacity obtained from the load-displacement relation. In the field of earthquake geotechnical engineering, this kind of ductility evaluation has been used before[6)], so it is possible to evaluate the ductility of the soil material against liquefaction if necessary[7,8)].

2.2 Liquefaction potential assessment methods need to be selectively used according to the purpose of liquefaction damage assessment

By inspecting ground earthquake performance against level 2 earthquake motion, there are issues need to be examined related to the importance of how to consider ductility in the assessment of the soil's liquefaction damage. Fig. 1 indicates the relationship between N1 value and cyclic stress ratio of in-situ samples. In this case, two major issues are hidden as follows. The curves in the figure are very useful in some cases, for example Specifications for Highway Bridges, of which purpose is serving as a design guideline for a large numbers of structures; and, as a government policy, aiming to conserve state properties by minimizing their damage ratio. In those cases, the focus is the direction of an arrow labeled as 'Evaluation on Damage Ratio' in the figure, and the objective is to distinguish the area which will be damaged or the area which will not be damaged in a wide variety of grounds based on the ratio of a certain number of data. That is to say, it is not the matter of predicting damage at individual points, but reducing the number of points where the prediction failed as a whole. The evaluation function like these curves are effective for simple evaluations across wide areas, for example, damage estimation map and F_L method, and also become effective when it uses other methods which evaluate a large number of conventional structures together.

On the other hand, to design important structures individually, the evaluation of liquefaction strength based on ground investigation should focus on the direction of an arrow labeled as 'Individual Evaluation' in the same figure. In such case, it is not negligible that the liquefaction strength around the N_1-value of 30, which needs detailed assessment for larger earthquakes (when the N_1-value is small, it is used to predict damage by gigantic earthquake, so, making the counterplan is more important than the detailed assessment,), varies greatly. The problem is that there are mixed data from various kinds of soils in this figure; therefore, it is highly possible that the data enclosed within the circle correspond to two kinds of soils with completely different nature, even their classification is the same as alluvial soil. Which means, those data should not be compared to the same line, for cases of individual evaluation. However, even in the design of the important structures, it seems that many designs use only standard penetration tests without any laboratory tests to derive liquefaction strength, and use that liquefaction strength for parameter setting of advanced numerical analysis, in order to reduce the costs.

Therefore it can be pointed as follows; (1) The data used to reduce the damage ratio and the data used for the design of individual structures must be clearly distinguished (2) Individual examination about the relationship between the type and condition of sand and liquefaction strength for soil with large N value is necessary, because it shows very large variations in liquefaction strength.

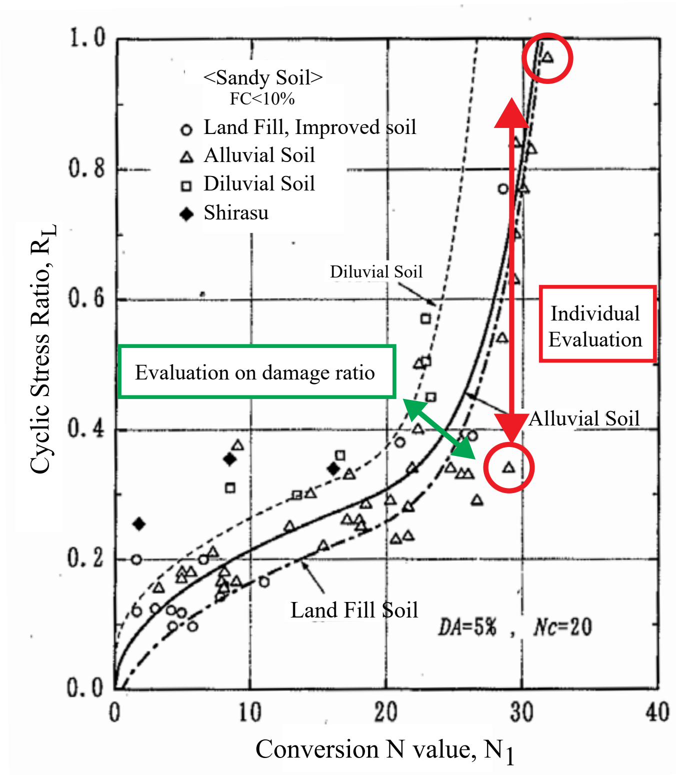

Fig. 1 Relationship between conversion N value and cyclic stress ratio (after Matuso[9])

2.3 Influence of long duration seismic motion and aftershock which cannot be explained by undrained cyclic shear characteristics of material

Regarding the liquefaction damage to the reclaimed land in the metropolitan gulf area caused by the Great East Japan Earthquake, the influence of long duration seismic motion and aftershock has become a topic[10,11]. Especially in Urayasu City, despite the seismic motion being not so large, it suffered from serious liquefaction damage.

It may appear one-sided trying to understand the influence of the duration and aftershock only from the aspect of the soil's material characteristics. Simply speaking, this influence is just a result of the increasing number of cycles of the seismic motion. Specifically, the soil, of which its undrained cyclic strength R_{L100} (cyclic stress ratio which cause failure by 100) is under 0.2, does not lead to liquefaction failure when cyclic stress ratio of under 0.2 acts in less than 100 repetitions; however, considering that pore water inflows during the cyclic shear and expands the pore volume, even if it is soil with a cyclic stress ratio of 0.2, it could lead to failure by liquefaction, when cyclic stress ratio of under 0.1 acts only 20 cycles[2]. A situation like the influence of seismic motion increasing during the latter half of the long duration or in the aftershock 30 minutes later can be explained by the redistribution of the void space due to the inflow and outflow of the pore water. The issue of pore water's redistribution should be treated as a boundary condition problem, along with material characteristics, since it is determined by ground composition, and permeability and volumetric compressibility during liquefaction. The influence of long duration seismic motion and aftershock could not be understood correctly by only considering undrained cyclic shear characteristics of the ground material.

3. THE PROBLEM IN EVALUATING THE AMOUNT OF SUBSIDENCE CAUSED BY LIQUEFACTION

3.1 What determines the volume contraction after undrained cyclic shear?

The liquefaction degree of soil in which the excess pore water pressure has risen once can be quantified by maximum volumetric strain, which is the amount of drainage from a state not causing residual shear strain. The amount is known to be dependent on the deformation strain history of the soil, in addition to material properties like its density. Currently, the degree of liquefaction was sorted out depending on the maximum shear strain amount generated during stress controlled undrained cyclic shear tests[12,13]; however, our research has found out that the cumulative shear strain amount is most accurately represented as a historical index changing the degree over time[14]. As a brief example, the maximum shear strain cannot explain the difference in the amount of drainage between the case with a cyclic deformation of several times and the case with that of several dozen times under the same strain amplitude. The important point is that, in case of examining volume changes in soil caused by shear, certain test conditions cannot be assured in the stress-controlled tests, which examines deformations due to loading. Material properties resulting from volume changes under cyclic shear can be compared only after applying the same deformation conditions. Specifically, it has been confirmed experimentally that when the cyclic shear strain history is the same, the volume changes due to the cyclic shearing in the dry state and the volume changes during the drainage after the undrained cyclic shearing in the saturated state, are almost the same[15]. This is understandable considering the idea that granular material's dilatancy is caused by mutual movement of grains. That is the reason why the strain-controlled test is superior for material testing.

3.2 Issues while predicting settlement of sandy ground that contain a large quantity of silt (fine grain)

(1) Model experiments[16]

The ground of Urayasu City, where significant liquefaction has been observed, contains 50% of fine fraction. The method currently used for predicting ground subsidence by liquefaction is using relative densities; however, in the first place, the sand containing fines more than 5% is excluded from maximum and minimum void ratios test methods of Japanese Geotechnical Society Standards. Therefore, we studied the settlement of sandy ground that contains a large quantity of fines, by using shaking table tests at a 30G centrifuge.

Fig. 2 shows the model section and sensor arrangement. We used three kinds of samples, liquefied sand from Urayasu (Urayasu Sand, FC approx. 50%), silica sand (silica No. 7, FC=0%), and fine-mixed silica sand resembling the grain size distribution of Urayasu sand (silica No. 5, 6, 7, 13% for each, quarts powder 61%, FC approx.60%). For the excitation, we input 200 Gal, 2 Hz sine wave for 300 seconds to the Urayasu sand ground and for 50 seconds to the silica sand ground and the fine-mixed silica sand silica sand ground.

Fig. 3 shows the relationships among the amount of subsidence from the centrifugal vibration experiment, the range of the maximum and minimum void ratios (determined by the maximum and minimum density tests of JGS, the test which is not applicable to those with FC > 5% originally), and the static consolidation test. In this figure, the fine-mixed silica sand, which started from relative density of 100% or more, showed larger subsidence on excitation than the silica sand under the same excitation condition. From the above, we consider it is difficult to evaluate the volumetric strain from the relative density obtained by the JGS's minimum density test to the sand containing fine grains.

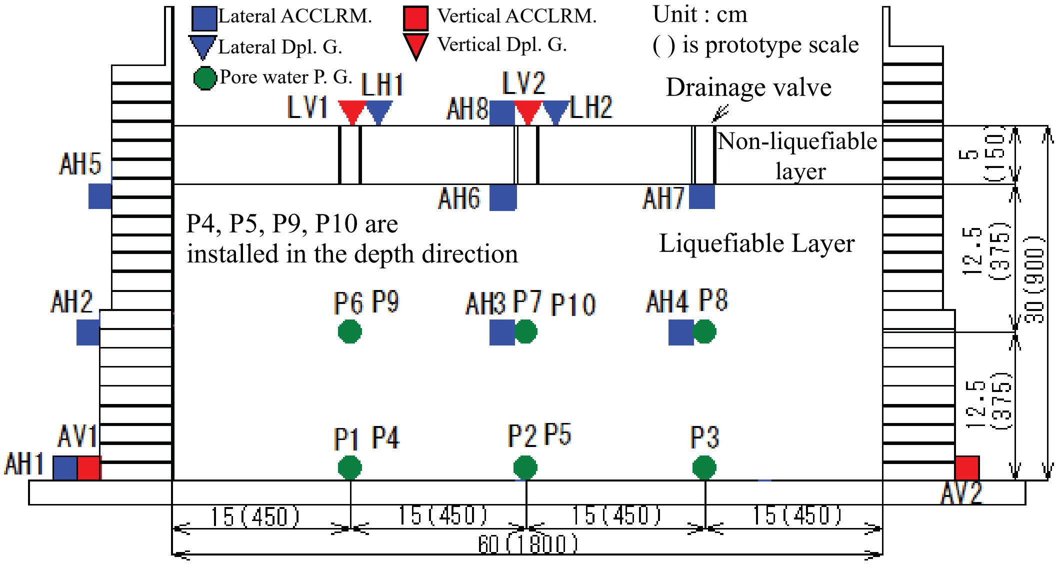

Fig. 2 Model cross section and sensor arrangement[16)]

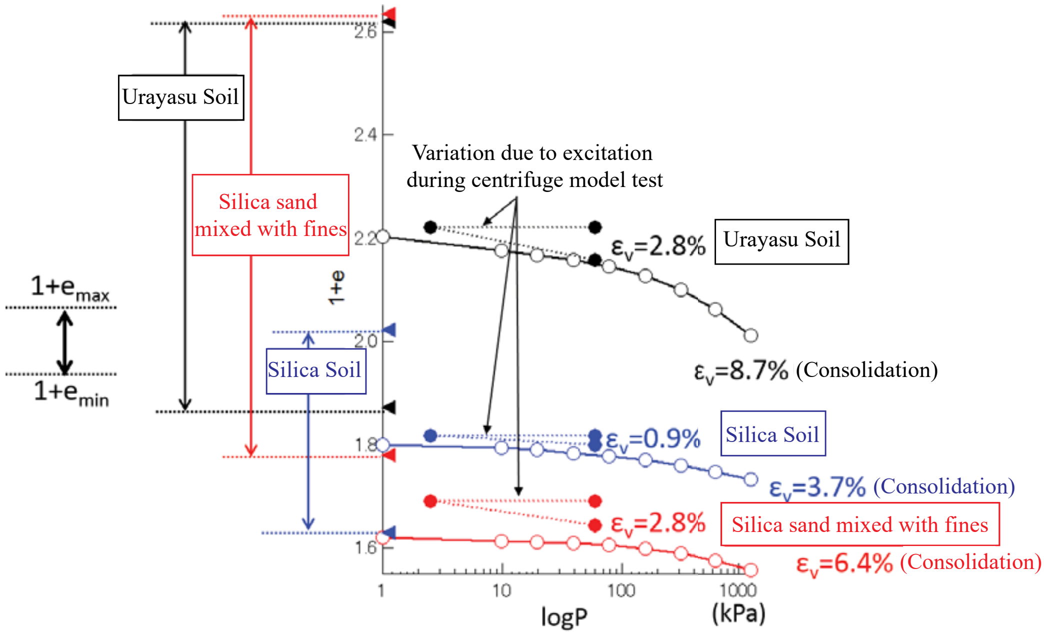

Fig. 3 Relationship between settlement resulted from centrifuge model test, maximum and minimum void ratio, and oedometer test

(2) Compressibility of sand containing large quantities of silt (fine content[17,18]) – the relative density is an indicator applicable only to clean sand

We have already seen that fine content, together with viscosity and plasticity, increase liquefaction strength. On the other hand, regarding the influence of non-plastic fine grain content on liquefaction strength and damage after liquefaction, it seems that there is no fixed opinion at present. The primary reason for this, besides the differences in materials, is that the index and the comparison method used for each researcher are different. The results of the centrifugal test mentioned in (1), clearly show this fact. Relative density is often used as an important index when evaluating liquefaction strength /

damage. The relative density is the index indicating where the soil in a certain state is located between the maximum and minimum void ratios. In the present situations, when using relative density, the knowledge obtained in an A soil is carelessly applied to a B soil. In order to figure out the relative density, it is necessary to calculate the maximum and minimum void ratios, which use the value determined by the method prescribed by the Geotechnical Society (JGS 1224). On the contrary, as described in the Japanese Geotechnical Society Standards, this test method is not aimed at determining the absolute minimum void ratio of soil, because the ratio obtained by this method varies depending on the number of strikes, striking force, and confining stress. The density of soil containing a large number of fine particles changes drastically influenced by those conditions, as the FC becomes larger. For example, in the case of a sample containing fines of 11.6%, there was a report which failed to obtain the maximum void ratio because moist contents became 2.3% after a single test with a sample in an absolutely dry state, thus it did not fall in the air[19]. On the contrary, there is the case of another report that the method overestimates minimum void ratio.

As mentioned above, the minimum void ratio of the soil greatly changes depending on the difference between the dry state and the water immersion state, in particular, the difference in confining pressure, the way to yield compaction energy, and so on. Soil containing a large amount of fine grains is greatly influenced by the amount of pore water and the way to yield energy. From the viewpoint of evaluating liquefaction strength and damage, we consider that volume changes due to cyclic shear correspond mostly to the seismic load. We introduce below the results of examining the compressibility of the soil at cyclic loading by laboratory tests.

We have used a sample containing clean sand and fines to struck it in absolute dry condition and water immersion condition to examine the influence of pore water on the minimum void ratio. In addition, we compared it with the minimum void ratio after cyclic shearing by the hollow torsional shear test. For details of the experimental method, see reference[18]. Fig. 4 shows the results of the examination.

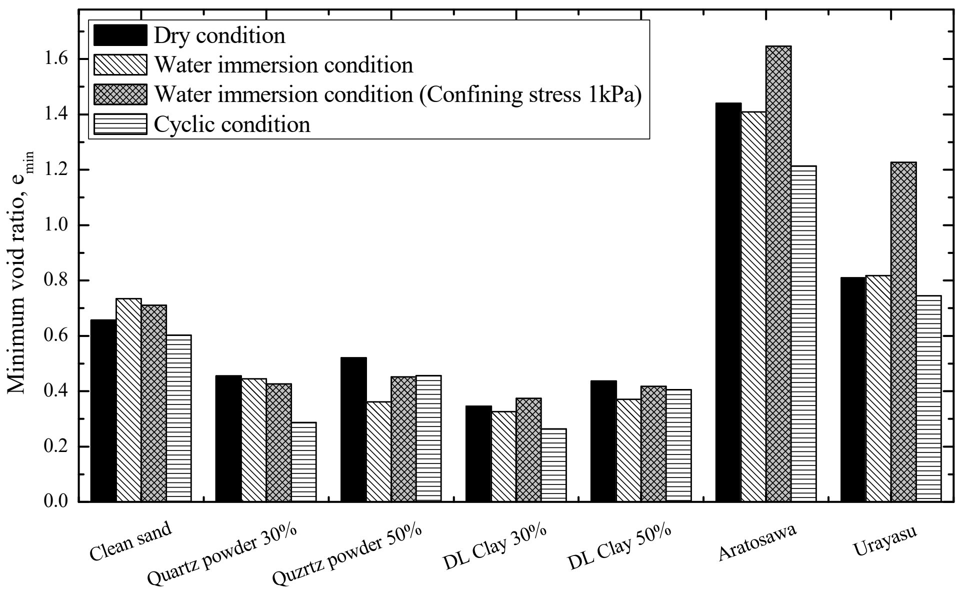

Fig. 4 Minimum void ratio resulted from diverse conditions and testing methods: The percentage shown after the material name indicates the fine content mixed with Iide silica sand. Quartz powder 30% implies the sample was prepared by mixing quartz powder of 30% to Iide silica sand. Fine content of each material was 0, 100, 100, 93, and 53.4 for Iide silica sand, quartz powder, DL Clay, Aratosawa, and Urayasu respectively.

We tested the minimum void ratio in the water-immersed state, without confining stress and with 1 kPa confining stress. The results shows that, in case of 100% of Iide-Silica-Sand No. 7, the void ratio in the dry state is smaller than that in the water immersion state, while in case of the sample containing fines, the void ratio is very similar or even smaller in the water immersion state. That means, the density can be higher in the water-immersed state than in the dry state, therefore, the JGS method overestimates minimum void ratio for cases where the sample contains fines. Fig. 4 also shows the influence of confining stress in the state of immersion in water. Other than the sample that contains 50% of DL clay, we can see almost no influence of this stress, neither the density becoming higher when it does not exist. Furthermore, the case of cyclic condition, which repeats undrained cyclic shear and drainage afterwards, results in the lowest density. This shows the separation between the experimental results and the evaluation obtained by the JGS method becoming greater when the fine fraction content FC is larger. The soil, which can be obtained based on the density test method regulated by JGS, is limited to practical clean sand; however, the serious problem is that the method is used beyond the scope of the application. Also like us, Tatsuoka et al.[21] pointed out, about embankment compaction, the deviation between the minimum void ratio on the site of embankment compaction management and JGS's minimum density test method for finding relative density has shown to increase when FC increased.

On the other hand, obviously the studied soil for liquefaction was not entirely clean sand, and the quality of soil, which is subjected to cyclic shearing, is varied from sandy soil, intermediate soil, to cohesive soil. It should be recognized that there are limits to the relative density, which only applies for clean sand, and to the liquefaction strength index using the N-value of which meaning differs depending on soil and confining pressure. Incidentally, researchers who have studied about the relationship between N-value and relative density[22, 23] have not stated that their result could apply to all kinds of soil in general.

3.3 The amount of settlement cannot be determined only by volume contraction.

Generally, the amount of ground surface settlement caused by liquefaction has been understood as to be determined by the amount of pore water, which is squeezed out by the contraction of the soil particles skeleton. That corresponds to, as mentioned in 3.1, the amount of drainage after undrained shear tests of a sand sample. However, liquefaction is different from the ideal consolidation phenomenon, which squeezes out only pore water, due to the ejection of soil particles together with the water. As left Fig. 5-left shows, the amount of sand ejected from house-grounds and load paved faces was bigger than ever. Due to sand being removed, the amount of ground settlement by liquefaction is naturally larger than the case in which only pore water is drained

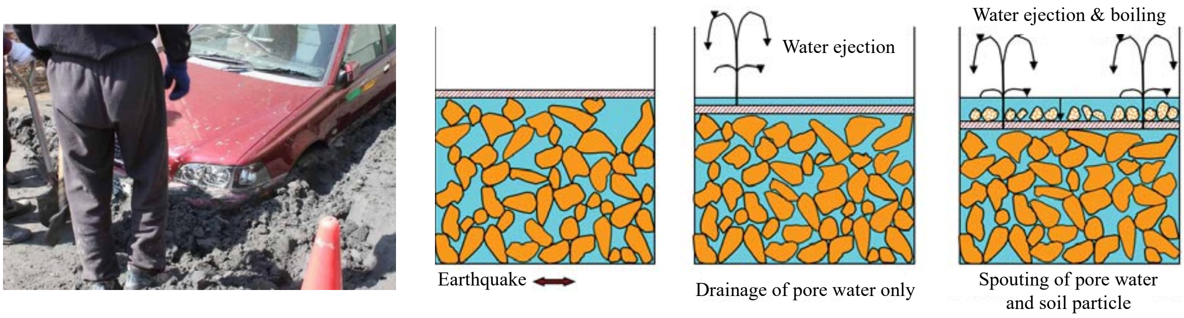

Fig. 5 (left) Sand boiling in Urayasu City : from Eight-Japan Engineering Consultants Inc.
(right) Scheme of sand boiling and water ejection : the amount of settlement is larger in case of water ejection accompanied by sand boiling compared to water ejection only

In order to predict the amount of ground settlement by liquefaction, we also need to predict how much sand will be ejected. So far, the term ejected-sand used to have meaning only as supporting evidence of liquefaction under the ground; however, it creates a new problem. If we try to predict the amount of ejected sand in response to the ground liquefaction, its prediction would be quite difficult. The material requirements for ejecting sand and the requirement as a boundary condition are unclear at present. Trying to reproduce the situation using clean sand in an experiment similar to Fig. 5, would not have been successful. Pore water would be drained easily from the soil particle skeleton and it would not keep its flowing condition. Moreover, in order to eject, an external force to push out is required for the ejection.

Taking these facts into consideration, the soil containing fine grains with low water permeability is a rather undesirable material compared to clean sand, even in the sense that generated excess pore water pressure cannot easily dissipate. In general, as the amount of FC increases, it can be said that the amount of liquefaction resistance gets larger; however, at least for the reclaimed ground using pump-dredging (hydraulic fill) should not be evaluated its liquefaction resistance by amount of FC.

4. ISSUES ON LIQUEFACTION DAMAGE OF RIVER DIKES

4.1 Shimonakanome-district, Naruse-river - liquefaction damage of river dikes caused by thin liquefied layer

As for the liquefaction damage of the Great East Japan Earthquake, a large number of liquefaction damage has occurred in Tohoku district[26]. Among them, there is a case of a characteristic river dike damage caused by a thin liquefied layer. Here, we introduce the case of the left bank of Nakanose-river, upstream of Shimonakanome –district.

In this district, a soft clayey ground is thickly deposited on the base ground. Its microtopography, based on the flood control topography classification, corresponds to the flooding area. The upper part of this clay layer, which is the foundation ground, had a concave shape centering on the lower part of the top end because of consolidation by embankment load. The levee body is based on the former bank of cohesive soil, which was widened and raised to the back side of the river with sandy material. Inside the levee body, the groundwater level, which is convex up to a position higher than the ground level, was confirmed. Therefore, it was considered that the saturated embankment sandy soil below that position liquefied. This district has no report of the damage history of the past earthquakes such as the earthquake of the Miyagi prefecture in 1978 and the earthquake at the northern part of the Miyagi prefecture in 2003.

(1) Overview of the embankment damage and open cut investigation[27]

Fig. 6 shows photographs of the damage and profiles from the open cut investigation. There are multiple longitudinal cracks that occurred on the levee crown and on the slope of the liver's back side; therefore, due to settlement of the levee crown and deformation to the back side of the river, the embankment after the disaster was almost horizontal.

Fig. 6 (left) the disaster area (the inside of the dike is shown on the left side, seen from downstream to upstream
(center) liquefaction trace verified by the open cut investigation (marker points indicate the trace of liquefied sand which penetrates into the inside of the dike
(right) cross-section of the open cut investigation : trace of squeeze of liquefied sand was confirmed

Researchers conducted an open cut investigation of the damaged embankment, in order to confirm the cause of the disaster and its mechanisms. The results are as follows;
- The foundation ground is concave in the lower part. There are no cracks or deformations which cross the cohesive soil layer.
- On the upper part of the foundation ground (the lower part of the levee body), there was a sand layer considered liquefied, and the subsidence and depression of the levee body material dug into the sand layer. Therefore, the thickness of the sand layer assumed to be liquefied was uneven.
- There were a large number of sand dikes extending upward from the sand layer of the foundation ground surface (in Fig. 6), along with the sand dike (Fig. 6 right side) flowing outward of the levee body along the foundation ground surface layer at the lower part of the slope. They indicate that liquefaction occurred in the sand layer at the bottom of the widening embankment on the back side of the river.

Based on the results of the open cut investigation, we estimated that the main factor of the embankment damage was liquefaction of the saturated levee body. In the bottom of the levee body, which widened (embanked) on soft clayey ground with sandy materials, the soft ground layer has become concave due to settlement resulting from consolidation. At the top of the ground layer, water, which penetrated the levee body, formed a saturated area with ground water, which we call the saturated levee body. As a result, rigidity and strength of the saturated levee body has declined, in addition to cracks and depressions forming in the levee body. Afterwards, liquefied sand has been blown out from the foot of the back slope, and lateral fluctuation of the levee body has occurred. Fig. 7. Shows assumed damage mechanism.

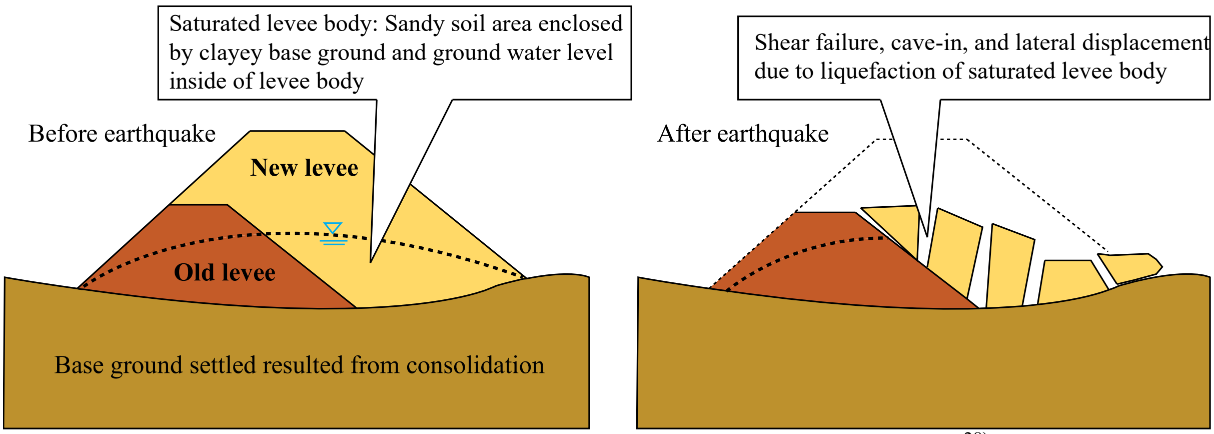

Fig. 7 Scheme of saturated levee body and damage mechanism[28)]

(2) Response to damage caused by liquefaction of closed saturation region of river bank

The characteristic feature we observed was the traces of liquefied sandy soil in embankment immediately above the foundation ground being composed of soft, viscous soil, and in many cases the liquefaction resulted in damage to the embankment. In this case, the sandy soil, which is the material of the levee body, is considered to be liquefied as it became saturated below the ground water level due to consolidation settlement of foundation ground, and at the same time it was loosened due to stress relaxation. For this particular case, there was a report on liquefaction of sandy soil in the levee body constructed on peat ground that occurred at the time of the 1993 Kushiro-oki earthquake[29)] and there was a warning issued[30)]; however, it did not caught enough attention in practice.

Looking back on those damages, the most important thing is that the liquefaction of a thin layer of about 50 cm to 1 m will cause a severe damage. As can be seen from those examples, when the foundation part of the foot of a slope, which exerts the shear resistance under static condition and resists the deformation of the levee, loses the resistance by liquefaction, it easily causes large damage due to shear fracture. When looking at the damage of liquefaction, we often regard the thickness of liquefaction layer as important (for example, PL value). It is effective for damage prediction that integral value works, for example, settlement prediction. On the other hand, in shear failure of river embankment, the question is how much shear resistance expected under static condition declines. The thickness of the liquefaction layer is not closely related to the damage amount prediction. In the numerical calculation of the damage prediction due to liquefaction of a river levee, it is important how to determine the physical properties around the foot of slope.

4.2 Liquefaction damage of the river dike at Shimonomyo-district, Abukuma-river – How much should we perform an investigation finely?

Here, we introduce the example that a suffering area was divided in two by the position relation between the distribution of the liquefaction layer and the bank body made by cohesive soil.

The damage example is Abukuma-river left bank side dike of the approximately 500m south side down a join of Abukuma-river and the Shiraishi-river. Lateral deformation of the embankment caused by liquefaction of the foundation sandy layer, the damage of residential houses inside, and sand boiling trace were observed. The damage area is located at the left bank side of Abukuma-river on natural levee. 13 places of sand boiling were confirmed around the dike (3 being river side and 10 being on the inside). Lateral deformation of the dike towards the inside caused the deformation of roads and houses nearby. Fig. 8 shows the cross section view based on a 3 bore-holes investigation and 25 Swedish sounding tests. A-A' section is the central section of damaged area.

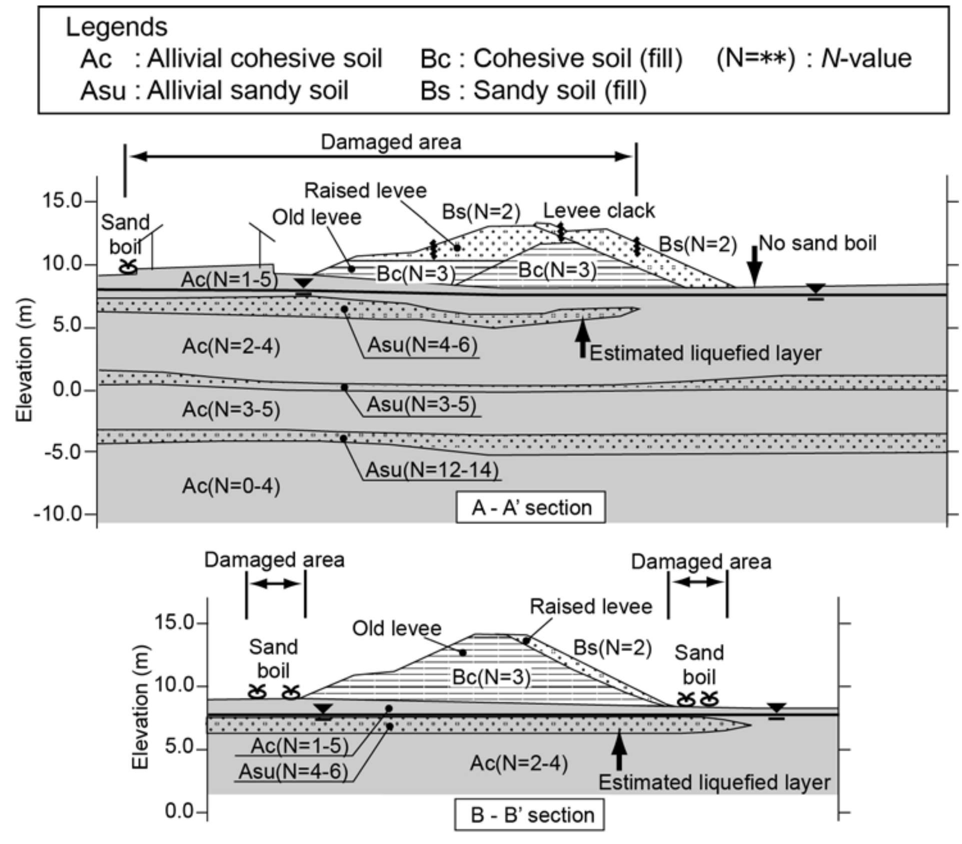

Fig. 8 Presumed cross-section of damaged area (Shimonomyou-district, Shibata-cho)[31)]

Foundation ground of the embankment is composed of 3 alluvial sand layers spanning up to 17m and 4 alluvial clay layers. Ground water level was observed at 1.0 m below the foundation ground. The old bank part showed viscous soil; the new bank part showed sandy soil among bank bodies; and the trace of the liquefaction of the bank body in itself was not found. From this, it was inferred that the shallowest alluvial sandy layer (SPT N-value = 4-6) located from 5 m to 7 m above sea level had liquefied.

This estimated liquefaction layer did not continue in the A-A' section to the foot of the slope and broke off under the bank body. On the contrary to this, in the B-B’ section, the liquefaction layer continued beyond the foot of the slope. In the part which the lateral displacement of the bank occurred, the estimated liquefaction layer breaks off under the bank body, and sand boiling occurs only in the inside. On the other hand, in the B-B' section where the estimated liquefaction layer continued to the foot of the slope, the lateral displacement of the bank body did not occurred, but sand boiling occurred at the both ends of the bank body.

It is thought that this kind of damage contrast was generated by the positional difference of liquefiable layers in the plain and the underwater drainage condition, this, regarding the dissipation of excess pore water pressure[32)]. With respect to the earthquake-resistant evaluation of the river bank and the countermeasures, how much can we grasp the difference in such situation exactly? Screening techniques are demanded in order to ensure the effectiveness of the earthquake-resistant evaluations when setting the priority of the counter measures. Because target section extension becomes enormous, we have to focus the target of the investigation. The river repair history of the natural serpentine river is important information.

5. CONCLUSION

In this paper, we observed the liquefaction damage in the Great East Japan Earthquake Disaster and pointed out some of problems to watch for in the present technique. It is thought that there are many points lacking any explanation because of page limitation. We would be very happy if this paper leads to the betterment of the liquefaction prediction measures technique of our country.

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