17th World Conference on Earthquake Engineering, 17WCEE

Sendai, Japan - September 13th to 18th 2020

LIQUEFACTION RESEARCH ON ANTI-CATASTROPHIC STRATEGIES FOR EXTERNAL FORCES EXCEEDING THE DESIGN SEISMIC LEVEL

M. Kazama(1), A. Kamura(2), J. Kim(3) and J. Han(4)

(1) *Professor, Tohoku University, motoki.kazama.b2@tohoku.ac.jp*
(2) *Assistant Professor, Tohoku University, akiyoshi.kamura.a4@tohoku.ac.jp*
(3) *Senior Researcher, Korea institute of civil engineering and building technology, jongkwan@kict.re.kr*
(4) *Research Fellow, Korea institute of civil engineering and building technology, jimmyhan@kict.re.kr*

Abstract

In this paper, liquefaction research strategies, from the standpoint of material testing and boundary value problems, aiming at the avoidance of catastrophic ground failure are discussed.

In Chapter 1, the transition of liquefaction research is overviewed from the 1964 Alaska and Niigata earthquakes to present. Liquefaction research is divided into three stages, from the 1964 to 1995 Kobe earthquake, from 1995 to the 2011 Great East Japan Earthquake, and from 2011 onwards. After the 1995 Kobe earthquake, the intensity of seismic ground motion that should be considered with regard to general structure design became critical, and the seismic design of structures shifted to a focus on performance-based design. Furthermore, the 2011 earthquake and subsequent major tsunami showcased the need to avoid catastrophic damage if the external force exceeds the design level.

In Chapter 2, the necessity to reconsider the definition of 'liquefaction', and the concept of liquefaction research under such circumstances, is explained. For example, excess pore water pressure is not a suitable indicator for the degree of liquefaction. This is because deformation characteristics of loose and dense sand are different, even if they both have an excess pore water pressure ratio near 100%. The problem is not the occurrence of liquefaction resulting in the loss of effective stress but its continuation and the speed of recovery of effective stress. Therefore, we must evaluate the effects on soil ductility nature against liquefaction.

In Chapter 3, it is stated that various soils, in addition to clean sand, should be tested for liquefaction. This is because earthquakes do not differentiate between different types of soils. By studying various soils, the resistance to soil liquefaction becomes clearer. We believe that the current soil testing method based on the stress-controlled test is inappropriate for evaluating the ductile property of soils, because residual deformation properties of soil subjected to cyclic shear cannot be evaluated. Residual strain is divided into residual shear and residual volume strain component, and the experimental results show that their generation is characterized by a trade-off relationship with each other. In addition, a new testing method to evaluate the residual deformation behavior of soil subjected to cyclic loading is introduced.

In Chapter 4, from the standpoint of boundary value problems in relation to liquefaction phenomena, it is stated that there are still several unresolved problems such as the progressive failure following liquefaction. This paper discusses the differences in liquefaction damage scenarios using Urayasu City, located in Tokyo Bay, during the 2011 Great East Japan Earthquake as an example, in the case where the liquefaction is considered a) an undrained phenomenon and b) a drained phenomenon.

In conclusion, liquefaction research still offers several new avenues of learning. Researchers and engineers need to recognize the current inadequate understanding of liquefaction phenomena and thereby present realistic solutions. Constant research and technological development are required to improve the relevant design methods.

Keywords: anti-catastrophe, external force exceeding design level, liquefaction research, residual deformation

1. Transition of Liquefaction Research

1.1 Overview

Liquefaction research passed through two different stages before 2011 [1]. The first stage began in 1964 following the Alaska and Niigata Earthquakes, and marked the beginning of scientific research on liquefaction. This stage ended around 1995 when the Kobe Earthquake exposed the limitations of the present approach while demanding a new approach. A second stage was needed to counter major seismic motion while also considering large external forces. At present, it is once again necessary to consider our approach to liquefaction following the 2011 Christchurch [2] and Tohoku Pacific Coast Earthquakes [3, 4] where liquefaction of extensive areas took place. The effects of a long duration of motion and force exceeding the design level are matters requiring attention as shown in Fig. 1. After 2011, liquefaction research entered the third stage.

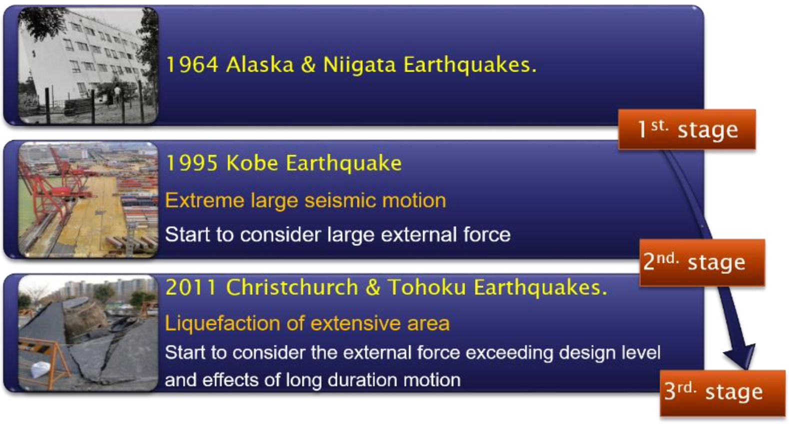

Fig. 1 Transition of liquefaction research from 1964 to present.

1.2 Liquefaction research targets

The distinctive targets of liquefaction research are shown in Fig. 2. For the first stage, an emphasis was made on the mechanism of liquefaction (excess pore water generation under the undrained condition), the prediction of occurrence, the countermeasures of liquefaction, and the constitutive model for simulation codes. The prediction of occurrence was estimated using Standard Penetration Test (SPT) N-value, Cone Penetrometer Test (CPT), grain size distribution curve, and response analysis. For the second stage, the target included the mechanism of post-liquefaction phenomena, evaluation of deformation after liquefaction, flow deformation, progressive failure, and a post-liquefaction analysis, which considered pore water migration. Before 1995, we were considering the process up to liquefaction but were not considering what happened after liquefaction.

Twenty-five years have passed since the Great Hanshin-Awaji Disaster of January 1995. How has the research on liquefaction developed since this earthquake? More importantly, even if we can determine that the research has developed, to what extent have the results helped to improve practical engineering? Unfortunately, practical liquefaction prediction and evaluation methods remain virtually unchanged since 1995.

The intensity of seismic ground motion that must be considered in regard to general structure design is critical, and the seismic design of structures shifted to a focus on performance-based design following the Kobe earthquake. The 2011 earthquake and subsequent major tsunami showcased the need to avoid catastrophic damage if the external force exceeds the design level. If a numerical analysis has already been established, estimating the residual displacement of the soil-structure system is possible. At the same time, the validity of numerical simulation results must be rigorously verified. Furthermore, the effectiveness of various ground improvements must be properly evaluated. We believe that the understanding of liquefaction phenomena so far has been insufficient, and the direction and purpose of the research have been wrong.

Liquefaction research should be aimed at avoiding catastrophic damage to external forces exceeding the design level as an anti-catastrophe strategy.

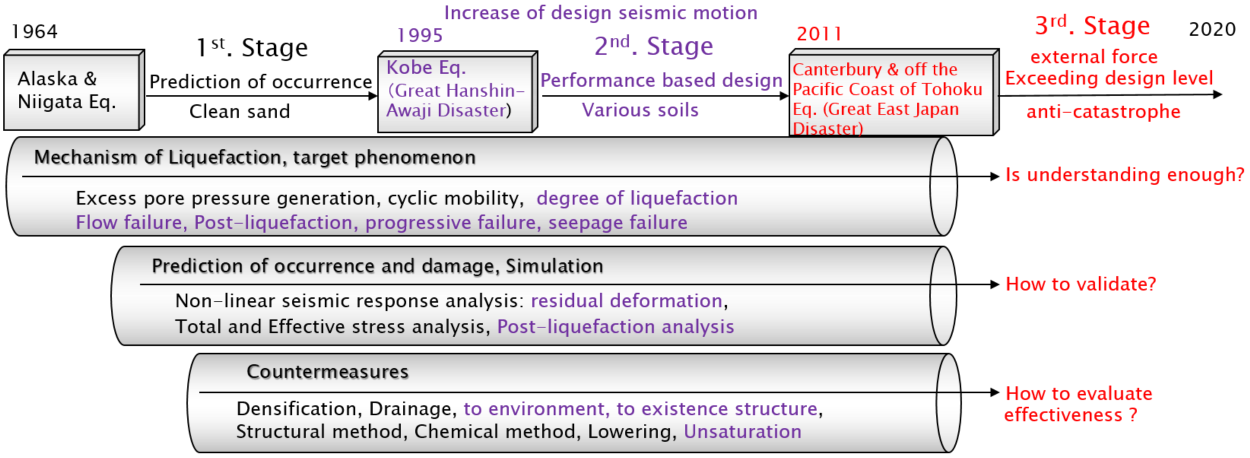

Fig. 2: Overview of liquefaction research targets.

2. Purpose of earthquake-proof design against liquefaction

2.1 Necessity to reconsider the definition of liquefaction

In general, "liquefaction" refers to a phenomenon whereby there is a loss of shearing resistance in saturated and loose sandy soil when subjected to cyclic shearing due to seismic ground motion. Excess pore water pressure rises gradually, consequently, the effective stress is lost. We fear that most geotechnical engineers believe the term “degree of liquefaction” simply indicates the level of the rise of excess pore water pressure. This is not true when considering the level of damage caused by direct liquefaction by gauging the amount of the residual deformation of the ground. The deformation characteristic of loose sand and dense sand is different, even if they both have an excess pore-water pressure ratio close to 100%. In the case of dense sand, the effective stress recovers after some deformation, and deformation does not proceed further. By contrast, the deformation of loose sand is considerable until the effective stress recovers or brittle failure occurs. This phenomenon is well known among researchers. To date, the terminology of liquefaction has been used as jargon by researchers when loose sandy soil causes brittle failure. It must be distinguished from cyclic mobility, which refers to the cyclic softening that occurs without runaway strains. Despite apparently limited strains, cyclic mobility is not benign to actual facilities.

When liquefaction is understood as brittle collapse type failures of the ground, from a practical perspective, the engineer tends to be concerned only about whether liquefaction occurs or not. Before the Kobe earthquake, engineers considered soil improvement necessary if the soil was judged to “liquefy” without the need for any further consideration, since such soil was likely to allow for catastrophic damage in the presence of liquefaction. As the intensity of seismic ground motion became the focus after the Kobe earthquake, engineers realized their way of thinking was irrational as they were judging even dense sandy and cohesive soils soil to be “liquefied.” The need to evaluate the ductility and the toughness of the soil to resist liquefaction became apparent for the first time [5], and a second stage of the research had been entered. This stage was defined by transformation of soil due to the presence or absence of liquefaction. It is necessary to reconsider the definition of soil liquefaction as a ground material in order to consider anti-catastrophe strategies.

2.2 Response to external forces exceeding the design level

Liquefaction is a common lecture topic in earthquake engineering. Researchers frequently believe the target of earthquake engineering should not be to make unbreakable structures and materials but to devise structures and materials that do not result in brittle failure. Over time, stronger seismic ground motion results in more data points that are useful for researchers. In the 2008 Iwate-Miyagi Inland earthquake [6], the PGA (Peak Ground Acceleration) was 3866 cm/s^2 for the up-down component of seismic motion, and over 4G when combined with the horizontal component. These were the highest numbers ever observed in Japan, according to PGA records observed at the ground surface [7]. Following the Kobe earthquake, the seismological network has been significantly improved in Japan. In our opinion, the earthquake-proof design code has not kept pace with the increase of the seismic design load in terms of resistance capabilities. In conditions where there is large uncertainty regarding seismic motion, it is critical that brittle failure does not occur. This is also crucial when seismic motion exceeds design expectations.

The status quo has not changed with regard to liquefaction avoidance design. For instance, soil is often prone to liquefaction even though the SPT-N value of the ground is over 30. Naturally, the strength of this soil is weaker than that of rock and concrete material. Nevertheless, such soil is naturally good material in terms of its ability to form structure and its ease of restoration. The primary consideration should always be to avoid fatal collapse. Engineers should primarily aim to alter soil characteristics that enable soil to be stabilized by liquefaction countermeasures in order to avoid catastrophic failure.

3. Anti-catastrophe strategy of liquefaction research in the material testing method

3.1 Liquefaction of various soils

One feature of liquefaction research following the Kobe earthquake that is that more extensive types of soils have been targeted for research. This can be attributed to the liquefaction of well-graded decomposed granite soil during the earthquake [8]. Thus far, research on liquefaction has mostly been conducted on clean sand, as clean sand is easy to handle. However, earthquakes do not naturally display a preference for soils. In practical engineering, liquefaction research for various kinds of soils is necessary as the actual ground is rarely composed of clean sand.

A relevant example can be seen with different soil types that give rise to different liquefaction resistant properties. Fig. 3a shows a comparison of normalized dissipation energy (NDR) for a strain double amplitude of 1.5% between Toyoura sand (clean sand) and weathered granite soil (reclamation material for the Kobe artificial island that was liquefied during the 1995 earthquake). Here, NDR represents the liquefaction resistance of soils and is also known as ductile index [5, 9, 10]. Fig. 3a shows that the liquefaction resistance of weathered granite soil does not increase with relative density, and weathered granite soil similarly absorbs energy to clean sand at a relative density of about 70%.

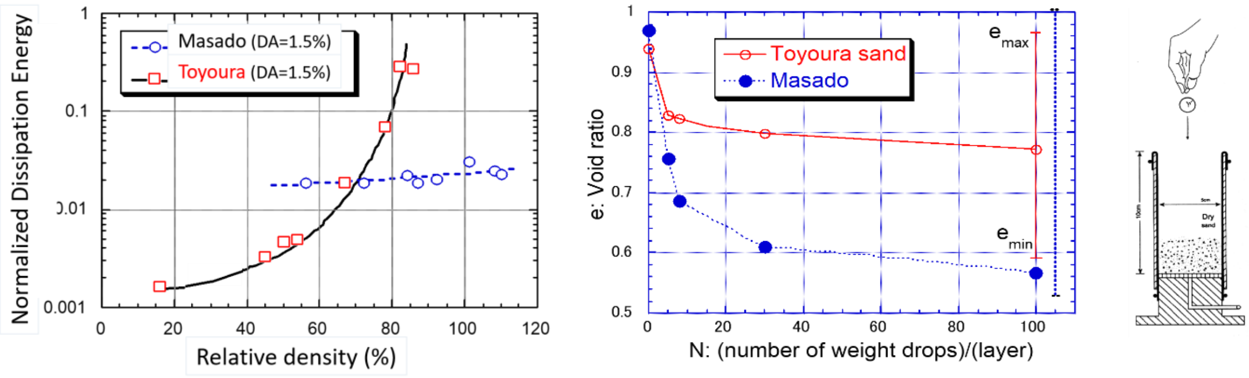

Fig. 3a Left: Comparison of liquefaction resistance between Masado (weathered granite soil) and Toyoura sand (fine clean sand). Fig. 3b Right: Compaction property difference between two kinds of soils.

Fig. 3b shows the compaction property of weathered granite soil and fine clean sand. It also shows the relation between the void ratio decrease and the number of weight drops with maximum and minimum void ratios. It was found that weathered granite soil has an easier compaction nature in comparison to fine clean sand. The difference between liquefaction resistance, as seen in Fig. 3a can be attributed to the compaction nature of soils, as seen in Fig. 3b. Soil that does not require energy for compaction quickly liquefies, even if the relative density is over 90%. From these results, it appears that relative density cannot be regarded as a universal indicator for representing the liquefaction resistance of soils.

There are many volcanic countries worldwide. Some of these include Japan, Indonesia, Italy, and New Zealand. Volcanic ash sandy soils are well known as easily liquefiable soils [13, 14]. In recent years, liquefaction of volcanic ash sandy ground has been observed in the 2016 Kumamoto earthquake [15] and the 2018 Hokkaido Eastern Iburi Earthquake. Fig. 4 shows an example of fill slope failure due to an earthquake involving volcanic sandy soils. This was observed during the 2003 Miyagiken-oki earthquake in Japan. As the right-hand side of Fig. 5 shows, volcanic sandy soils behave like a solid material before shaking. However, after shaking, the soils behave like fluid materials even if conditions are unsaturated [16, 17, 18].

Fig. 4: Example of fill slope failure due to an earthquake involving volcanic sandy soils.

3.2 Stress control test is not suitable for the soil with strain-softening behavior

The liquefaction test for soil includes a stress controlled undrained cyclic shear test. Results from this test are commonly reported in terms of a fatigue model. In this model, the number of cycles N required to reach a particular failure criterion is the basic result for the cyclic shear test. Commonly, data is presented in terms of the cyclic stress ratio versus the number of cycles of loading required to cause a 5% double amplitude strain. Sometimes, the condition where excess pore water pressure first reaches 95-100% initial effective stress is taken as an alternative failure criterion to the double amplitude of strain criterion.

As previously discussed, a new problem that arose from the Kobe earthquake concerned the mechanism by which soil reaches the failure state. The problem also involves subsequent soil behavior when it is subjected to large stress close to static shear strength with a low cycle. This soil naturally reaches the failure criterion state after two or three successions of cyclic loading if a constant stress amplitude near static strength is applied repeatedly. It is natural that soil will be liquefied according to this criterion if the external load level increases. However, it is questionable whether all types of soil become liquefied in seismic ground motion events as large as the Kobe earthquake. It is also questionable whether the extent of the damage is similar.

The first author of this paper showed that the constant stress amplitude controlled cyclic test was an unsuitable testing method for loads at large amplitudes with a low cycle and proposed a strain-controlled testing method [17]. This is necessary because it is impossible to repeatedly apply a large constant shear stress ratio to soils with strain-softening behavior. Despite this development, the framework of the liquefaction test has remained unchanged even in the wake of two great disasters. The conservatism of the cyclic testing code can be frustrating at times. Damage levels due to liquefaction are primarily related to the amount of the ground deformation. A significant problem can be seen when considering the damage that soil deformation will cause due to liquefaction as effective stress recovers following a rise in excess pore water pressure once effective stress is lost. This problem continues until recovery occurs, and the soil particle skeleton is capable of withstanding the original stress.

3.3 Experimental evaluation of residual shear and volumetric strain after liquefaction

The soil element breaks when a cyclic shear is applied under undrained conditions, and residual deformation remains. As an example of a simple condition, consider here the residual strain from an almost zero effective stress state following cyclic shear to the initial stress state after reconsolidation. First, the residual strain can be separated into residual shear strain and volumetric strain. A specific test can be carried out to evaluate how much shear and volumetric strain develops during and after the undrained cyclic shear. Fig. 5 shows the schematic diagrams of the testing procedure, which includes a monotonic shear loading following cyclic shear and re-consolidation afterward. The following three patterns regarding the development of shear and volumetric strains can be considered at the final stage after an earthquake.

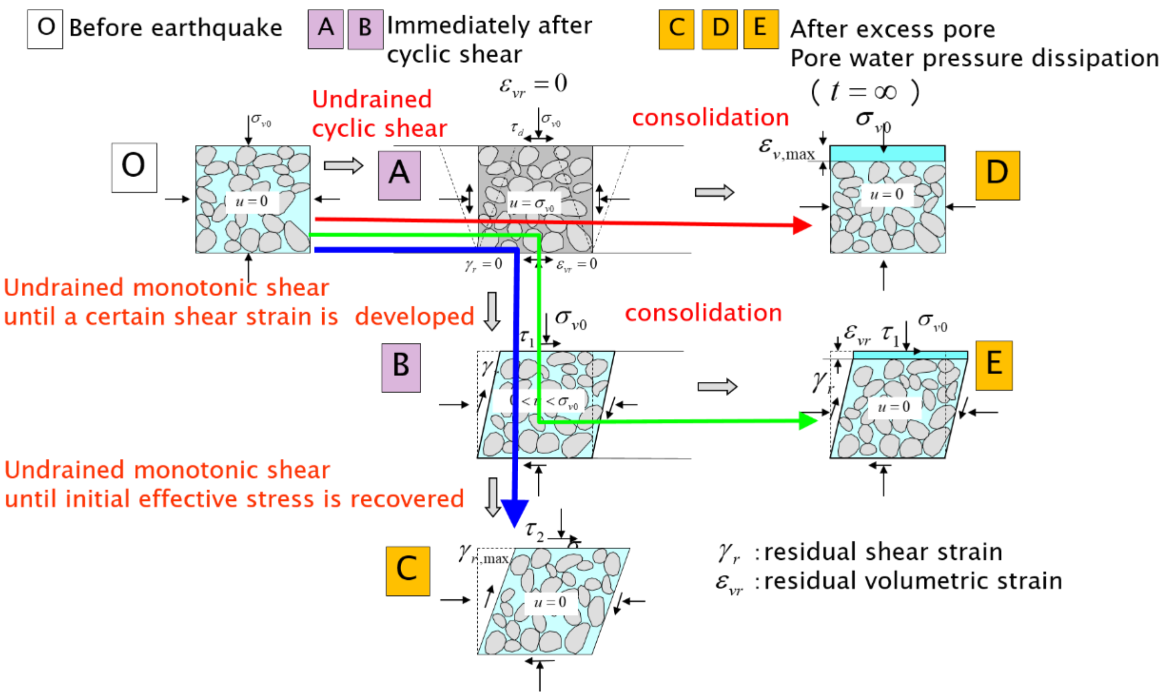

Fig. 5: Schematic diagram of the testing procedure for evaluating residual deformation properties during and after cyclic shear.

1) O→A→D: When the effective stress is not recovered at all, and everything stops at a neutral position by chance when the shaking stops. Only the residual volumetric strain due to drain occurs, without causing any residual shear strain.
2) O→A→B→C: When stopping with the effective stress recovered in the state of an initial vertical stress. Only the residual shear strain occurs because positive dilatancy has already occurred during undrained shear deformation. In this pattern, no excess pore water pressure remains, and no residual volumetric strain develops.
3) O→A→B→E: When the effective stress recovers to some degree at the same time as shear deformation develops to some degree and then stops. The residual volumetric strain also co-occurs with the residual shear strain.

Fig. 6 shows the typical results of the test described above. The graph shows the relation between residual shear strain and volumetric strain when the same shear strain history is applied to each soil specimen. The shear strain history applied consists of 100 waves of a constant single amplitude (0.5%). In the test, soil specimens were improved by the permeable grouting method [18, 19] with different concentrations of SiO_2, and non-improved (original) soil specimens with different relative densities were used. The soil was Silica sand called "Souma Keisa No.5" in Japanese, which is often used as a fine sand alternative to Toyoura sand.

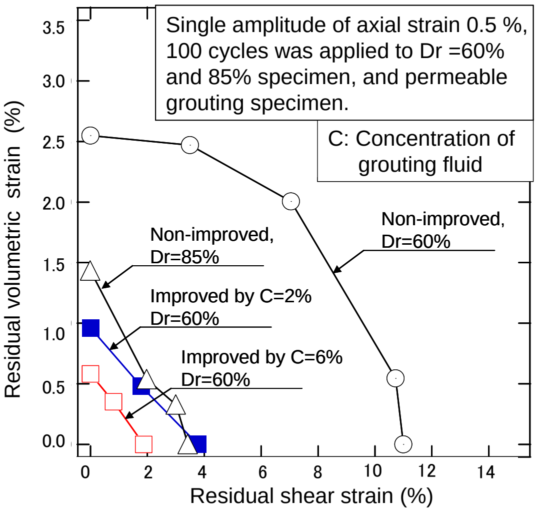

Fig. 6: Comparison of residual shear and volumetric strain after undrained cyclic shear, monotonic shear, reconsolidation [1, 20].

The plot on the vertical axis is obtained from the O→A→D pattern test, the plot on the lateral axis is obtained from the O→A→B→C pattern test, and the other plots are obtained from the O→A→B→E pattern test. The amount of residual deformation of the improved specimen is about 1/3 to 1/5 when compared to the deformation of the unimproved specimens. Effectiveness of an improvement in concentration of 2% is equal to one with a relative density of Dr=85%. Furthermore, the extent of the generation of residual volumetric strain and residual shear strain is related to the dilatancy characteristic of the sand. Notably, the relation between the shear and volumetric strains displays a trade-off relationship.

From the experimental results above, the residual shear deformation potential of the soil due to liquefaction where excess pore water pressure rises once can be quantified by subsequent drainage volume. This volume is the amount of the maximum volume shrinkage, and arises from a state of no residual shear strain. The amount depends on the shear strain history, in addition to material properties such as density of the soil and the concentration of the grouting fluid. It can be assumed that volumetric strain potential depends on the amount of the maximum shear strain obtained from the stress-controlled cyclic shear test shown previously [20, 21]. In our research [22-26], the amount of the accumulated shear strain is the best history index showing a positive relationship to the maximum volume shrinkage. This is because the maximum shear strain cannot explain the difference in volume change between one wave loading and several loadings of the same strain amplitude. Importantly, we did not make the testing conditions constant in the stress-controlled cyclic shear test that provided the load and examined the deformation. In addition, the volume change characteristics of soil subjected to repeated loading can only be compared if the strain history conditions are the same.

A determination of whether shear strain becomes a superior deformation pattern in the actual ground or becomes a deformation pattern mainly composed of the volumetric strain depends on the boundary condition and the loading condition. For instance, visual evidence of sand boiling is often observed in wide- open places like schoolyards and parks. In such cases, the volumetric strain is actualized. On the other hand, residual shear deformation happens easily because initial shear stress acts on ground that has even a slight incline. The lateral spreading observed at Niigata City, and Noshiro City in the 1964 Niigata and the 1983 Nihonkai Chubu earthquakes are examples of this [27, 28].

3.4 New soil testing procedure of the cyclic shear deformation property [29]

The first and third authors of this paper proposed a new laboratory testing procedure aiming to classify soils according to their likelihood to undergo liquefaction in the event of an earthquake. In this procedure, it is possible to classify soils as either ‘clearly safe’ or ‘likely to result in significant damage if liquefied’ by testing a small number of them. The specific procedure schematic is shown in Fig. 7. The procedure consists of the following steps (STEP 1 to 4): constant stress amplitude cyclic shear, constant strain amplitude cyclic shear, monotonic shear, and drainage. STEP 1 is similar to the conventional liquefaction strength test. In this step, it is determined whether liquefaction occurs against a certain stress ratio. Confirmation of whether a specimen is safe is also conducted. Additional excitation on a specimen is performed to evaluate the accumulation of damage in STEP 2. Next, STEP 3 provides a rough evaluation of the post-liquefaction damage level. In this step, a specimen with a high potential to induce considerable damage is classified. In STEP 4, shear stress is unloaded to zero, and volumetric strain is measured. Parameters for each step, such as stress ratio (STEP 1) and the number of cycles (STEP 1 and 2), can be established by the user in accordance with the intended purpose. The proposed testing procedure may be utilized to establish the parameters for numerical analysis, even though a constitutive model by which all procedures are reproduced is necessary.

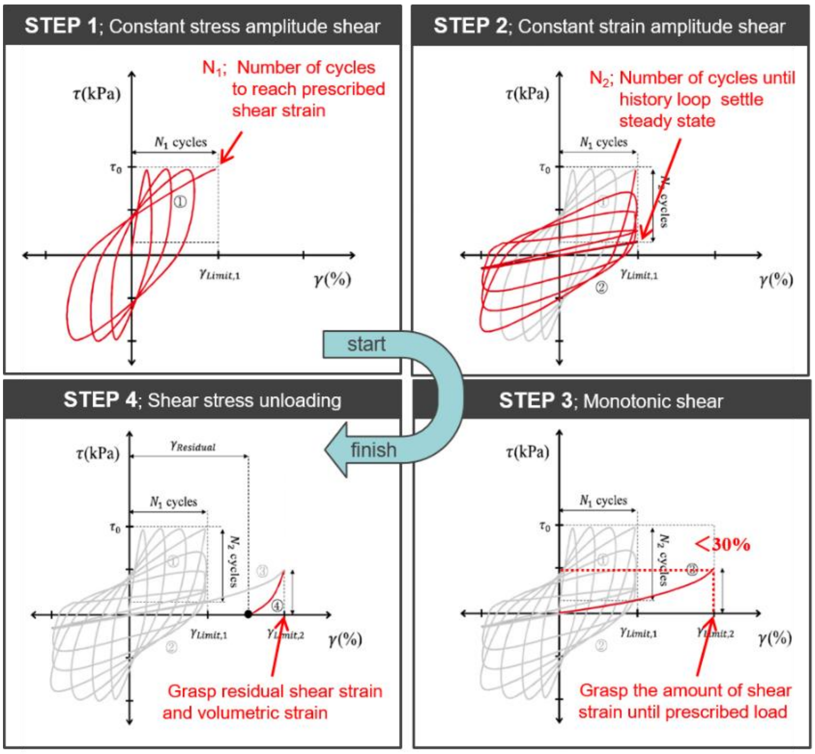

Fig. 7: Proposed new testing procedure for liquefaction resistance of soils [29].

A schematic of the liquefaction damage assessment based on the new test procedure is shown in Fig. 8. Liquefaction damage can be classified into three types: non-liquefaction (N.L.), limited deformation (L.D.), and catastrophic deformation (C.D.). The shear stress and strain components can be ascertained according to required performance specifications, their purpose, earthquake motion design, and the boundary condition. Detailed testing concepts are present in the reported literature [29]. At present, high seismic safety standards are required for nuclear power plant facilities in Japan. If this method is used, it is possible to appropriately evaluate the degradation of the stiffness of the soil composing the ground during an earthquake. This allows a deliberate avoidance of excessive measures for liquefaction analysis.

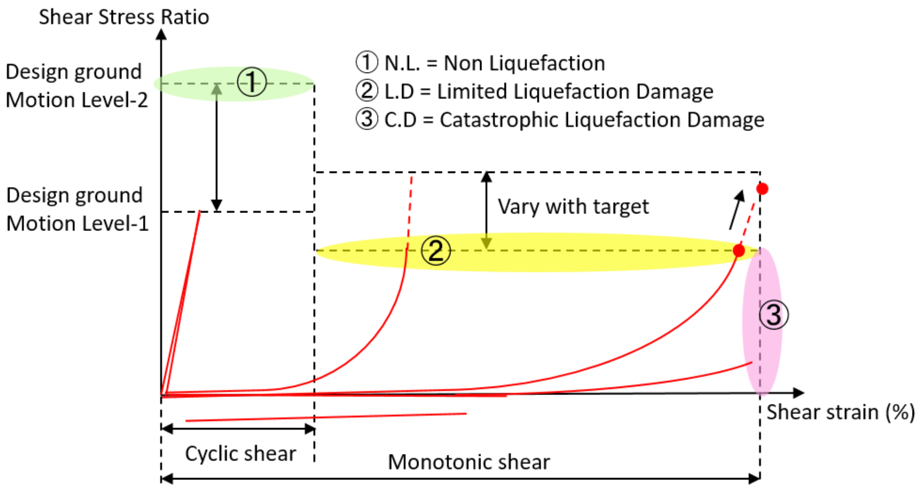

Fig. 8 Schematic diagram of liquefaction assessment [29].

4. Anti-catastrophe strategy for liquefaction research in boundary value problems

There are various unresolved boundary value problems related to liquefaction. Noda et.al [30] summarized issues related to the mechanism of liquefaction that emerged during the Great East Japan Earthquake of 2011.

4.1 Post liquefaction behavior of soil with regard to pore water seepage

The phenomenon following liquefaction due to seismic motion is known as post-liquefaction. Ground deformation and failure due to liquefaction occur both during vibration and following the cessation of earthquake motion. The destruction of the San Fernando dam in the San Fernando earthquake of 1971 is a famous example of progressive delayed fracture [31]. Another example of this can be seen in the collapse of the Showa Bridge in Niigata City in the 1964 Niigata earthquake. More recently, a slope collapsed following the main shock in the 2003 northern Miyagi earthquake [32].

The mechanism explaining the progressive delay fracture phenomenon following an earthquake is a seepage failure phenomenon caused by the migration of pore water due to liquefaction [33, 34]. Previously, it was shown that interstitial water was drained from the soil element during the effective stress recovery process. The extent of this process corresponds to an index indicating the degree of liquefaction. Drained pore water will naturally flow out beside the soil frame, but the soil element where the inflow of pore water occurs depends on the difference in permeability along with the hydraulic boundary condition. In general, excess pore water pressure in the ground causes water to rise to a higher level when liquefaction occurs at deeper levels. This only happens if the generated excess pore water pressure ratio remains the same. Pore water tends to move upward, which creates this pressure difference. The movement of underground water is an initial and boundary value problem relating to the permeability and compressibility of soil. It is also related to layer thickness and excessive initial pore water pressure distribution.

Residual lateral displacements of several meters resulting from liquefaction were identified in the 1964 Niigata earthquake [28]. During this earthquake, it was reported that the Showa Bridge collapsed a few minutes after the termination of the primary seismic motion [35]. The cause of the collapse was determined to be a deformation of bridge steel pipe pile piers by a lateral spreading caused by liquefaction. This type of damage is considered to be typical post-liquefaction progressive damage. The final structural damage of the pier pile has been studied using a seismic deformation method [e.g., 36], but there is still no explanation for the progressive damage. The deformation of a gently inclined slope during the dissipation process of excess pore water pressure generated by seismic shearing was demonstrated using the seepage failure mechanism described previously [34]. Additionally, an elastoplastic analysis of a pile subjected to external force from ground displacement can be performed using time histories of ground displacement and excess pore water pressure

[37]. Ground surveys called geo-slicers conducted 40 years after the Niigata Earthquake has revealed that traces of liquefaction and flow deformation are still present on riverbeds. [38]

4.2 Liquefaction damage scenario of Urayasu City caused by the 2011 East Great Japan Disaster

During the 2011 Tohoku earthquake off the Pacific coast, distinctive liquefaction occurred in Urayasu City, Japan. This liquefaction can be found in the metropolitan area of the Tokyo Bay area [3, 30]. The effects of long-duration earthquake motion have become a problem. Because liquefaction phenomena are assumed to exist under an undrained condition, there is a direct correlation between duration length and the number of repetitions. If liquefaction phenomena as a boundary value problem permitting pore water migration are examined, a different mechanism can be found. Fig. 9 shows other possible scenarios of Urayasu City damage caused by a long duration earthquake. Relatively loose homogeneous ground is assumed in the initial state, and the bottom layer is typically the most liable to liquefaction. If undrained, the upper layer is not liquefied due to a seismic isolation effect. If the lower layer is drained, however, then it is able to transmit shearing force, and pore water penetrates into the upper layer from the lower liquefied layer with drainage. As a result, the upper layer becomes liquefiable, and the liquefied layer becomes very thick. The amount of ejected subsidence and sand also increases.

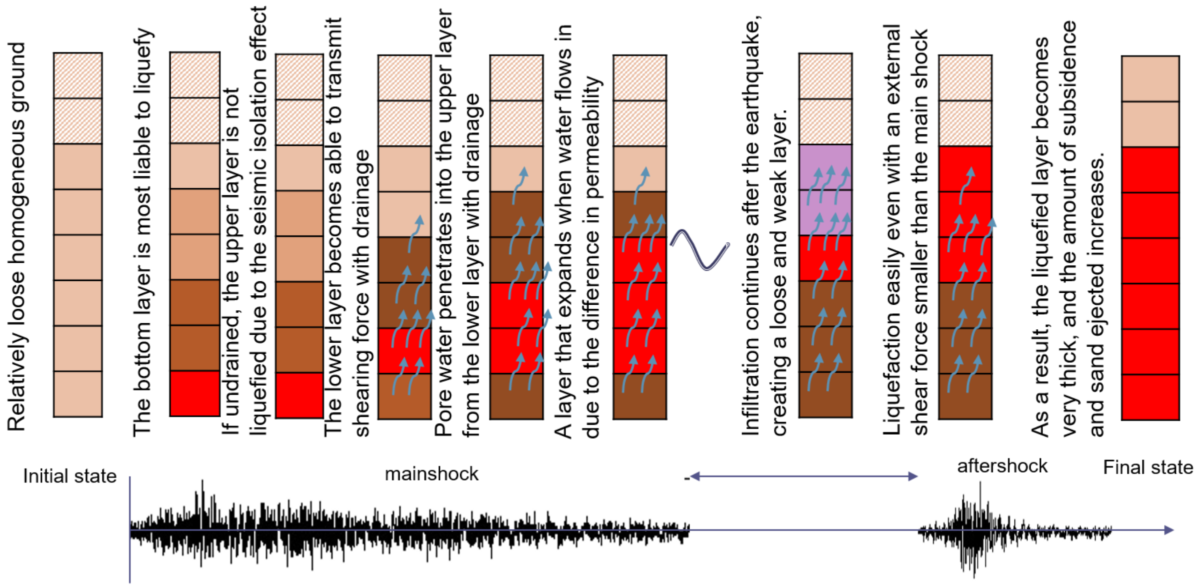

Fig. 9: Possible scenario of Urayasu City damage caused by the 2011 Great East Japan Disaster.

5. Conclusion

In this paper, we have discussed future liquefaction research for avoiding catastrophic ground failure from the standpoints of material testing and boundary value problems. There is still much to learn about liquefaction research. Researchers and engineers have a mission to present realistic solutions when considering that there is an inadequate understanding of liquefaction phenomena. Constant research and development of technology are required to improve design methods.

6. Acknowledgements

This work was supported by JSPS KAKENHI Grant Number JP15H02263 and JP16H02362. This work was conducted by cooperation between Tohoku University and Korean Institute of Civil Engineering and Building Technology. We would like to thank Editage (www.edutage.com) for English language editing.

7. References

[1] Kazama, M. (2011), Residual deformation property of liquefied soil and related phenomena, *The 14th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering*, Hong Kong, Keynote lecture, 85-97.

[2] Cubrinovski, M., Bradley, B., Wotherspoon, L., Green, R., Bray, J., Wood, C., Pender, M., Allen, J., Bradshaw, A., Rix, G., Taylor, M., Robinson, K., Henderson, D., Giorgini, S., Ma, K., Winkley, A., Zupan, J., O'Rourke, T., DePascale, G., & Wells, D. (2011): Geotechnical aspects of the 22 February 2011 Christchurch earthquake. Bulletin of the New Zealand Society for Earthquake Engineering, 44(4), 205-226.

[3] Yasuda, S., Harada, K., Ishihara, K., Kanemaru, K. (2012). Characteristics of liquefaction in Tokyo Bay area by the 2011 Great East Japan Earthquake, *Soils and Foundations*, **52**(5), 793-810.

[4] Yamaguchi A., Mori, T., Kazama, M. and Yoshida N. (2012): Liquefaction in Tohoku district during the 2011 off the Pacific Coast of Tohoku Earthquake, *Soils and Foundations*, **52**(5), 811-829.

[5] Kazama, M., Yamaguchi, A. and Yanagisawa E. (2000): Liquefaction resistance from a ductility viewpoint. *Soils and Foundations*, **40**(6), 47-60.

[6] Kazama, M., Kataoka, S. and Uzuoka, R. (2012): Volcanic mountain area disaster caused by the Iwate-Miyagi Nairiku Earthquake of 2008, Japan, *Soils and Foundations*, **52**(1), 203-219.

[7] Aoi, S., Kunugi, T. and Fujiwara, H. (2008): Trampoline effect in extreme ground motion. *Science*, 322, 727-730, doi:10.1126/science.1163113.

[8] Kazama, M., Kagatani, T. and Yanagisawa E. (2000): Peculiarities of liquefaction resistance of Masado (weathered granite soil). *JSCE, J. of Geotech. Eng.*, **645**(III-50), 153-166. (in Japanese).

[9] Law, K.T. and Cao, Y.L. (1990): An energy approach for assessing seismic liquefaction potential, *Canadian Geotechnical J.* **27**, 320-329.

[10] Okada, N. and Nemat-Nasser, S. (1994): Energy dissipation in inelastic flow of saturated cohesion-less granular media, *Geotechnique*, **44**(1), 1-19.

[11] Kazama, M., Unno, T., Sento, N. and Uzuoka, R. (2006): Liquefaction mechanism of unsaturated volcanic sandy soils. *JSCE, J. of Geotech. Eng.*, **62**(2), 546-561. (in Japanese).

[12] Orense, R. P. and Pender, M. J. (2016): From micro to macro: An investigation of the geo-mechanical behavior of pumice sand, *Volcanic Rocks and Soils, Rotonda et al. (eds)*, T& F, 45-62.

[13] Ogo, K., Hazarika, H., Kokusho, T., Matsumoto, D., Ishibashi, S. and Sumartini, W.O., (2018): Analysis of liquefaction of volcanic soil during the 2016 Kumamoto Earthquake based on boring data, *Lowland Technology International*, **19**(4), M 245-250.

[14] Kazama, M. and Unno, T., (2007): Earthquake-induced mudflow mechanism from a viewpoint of unsaturated soil dynamics. *Experimental Unsaturated Soil Mechanics*, *112 Springer proceedings in Physics*, T. Schanz (Ed.), 437-444.

[15] Unno, T., Kazama, M., Uzuoka, R. and Sento N. (2008): Liquefaction of Unsaturated Sand Considering the Pore Air Pressure and Volume Compressibility of Soil Particle Skeleton, *Soils and Foundations*, **48**(1), 87-99.

[16] Mori, T., Uzuoka, R., Chiba, T., Kamiya, K., and Kazama, M. (2011): Numerical Prediction of Seepage and Seismic Behavior of Unsaturated Fill Slope, *Soils and Foundations*, **51**(6), 1075-1090.

[17] Kazama, M., Yanagisawa, E. and Masuda, M. (1998): Evaluation of liquefaction resistance using strain controlled cyclic triaxial test. *JGS Getech. Eng, Magazine*, **46**(4), 21-24. (in Japanese).

[18] Kawamura, K, Kazama, M., Uzuoka, R., Sento, N., Suzuki, T. and Oshima, T. (2005): Liquefaction resistance of ground improved by permeable grouting method. *16th International Conf. on Soil Mechanics & Geotech. Eng., Geotech. Earthquake Eng. Satellite Conf.*, 130-137.

[19] Oshima, T., Kazama, M., Sento, N., Kawamura, K. and Hayashi K. (2008): Evaluation of liquefaction resistance and deformation characteristic after cyclic shear loading of improved sand by permeable grouting method, *JSCE, J. of Geotech. Eng.*, **64**(4), 732-745. (in Japanese).

[20] Ishihara, K. and Yoshimine, M. (1992): Evaluation of settlement in sand deposits following liquefaction during earthquakes. *Soils and Foundations*, **32**(1), 173-188.

[21] Shamoto, Y. Sato, M. and Zang, J-M. (1996). Simplified estimation of earthquake-induced settlements in saturated sand deposits. *Soils and Foundations*, 3**6**(1), 39-50.

[22] Sento, N., Kazama, M. and Uzuoka, R. (2004a): Experiment and idealization of the volumetric compression characteristics of clean sand after undrained cyclic shear., *JSCE, J. of Geotech. Eng.*, 764, 307-317. (in Japanese).

[23] Unno, T., Kazama, M., Uzuoka, R. and Sento, N., (2007): Volumetric compression behavior of sand under cyclic loading history with various stress paths. *Earthquake Geotech. Eng.* (CD-ROM). KYRIAZIS D. PITILAKIS Editor, Paper No.1278.

[24] Unno, T., Kazama, M., Uzuoka, R. and Sento, N. (2006): Relation of volumetric compression of sand between drained cyclic shear and reconsolidation after undrained cyclic shear. *JSCE, J. of Geotech. Eng.*, **62**(4), 757-756. (in Japanese).

[25] Uzuoka, R., Shimizu, Y., Kamura, A., Sento, N. and Kazama, M., (2007): A unified prediction for liquefaction and settlement of saturated sandy ground. *Earthquake Geotechnical Engineering* (CD-ROM), KYRIAZIS D. PITILAKIS Editor, Paper No.1382.

[26] Kim, J., Kawai, T. and Kazama, M. (2019): Minimum void ratio characteristic of soils containing non-plastic fines., *Soils and Foundations*, **59**(5), DOI: 10.1016/j.sandf.2019.08.001/.

[27] Hamada, M. (1992): Large ground deformations and their effects on lifelines: 1964 Niigata earthquake, *Case studies of liquefaction and lifeline performance during past earthquakes, 1, Japanese case studies*, Technical report NCEER-92-0001.

[28] Hamada, M., Yasuda, S., Isoyama, R. and Emoto, K. (1986): Observation of permanent ground displacements induced by soil liquefaction, *JSCE, J. of Geotech. Eng.*, **376**(III-6), 211-220. (in Japanese).

[29] Kim, J., Kawai, T. and Kazama, M. (2017): Laboratory testing procedure to assess post-liquefaction deformation potential, *Soils and Foundations*, **57**(6), 905-919.

[30] Noda, T., Kazama, M. and Asaoka, A. (2017): New developments related to clarification of the mechanism of ground deformation caused by earthquakes – in the wake of the Great East Japan Earthquake-, Journal of JSCE, Special topic -Restoration and Recovery from the 2011 Great East Japan Earthquake (invited paper), (5),133-144.

[31] Seed, H.B., Lee, K.L., Idriss, I.M. and Makadisi, F.I. (1975): The slide in the San-Fernando Dams during the Earthquake of February 9, 1971, *ASCE, J. of the Geotech. Eng. Div.*, GT7, 651-688.

[32] Uzuoka, R., Sento, N., Kazama, M. and Unno, T., (2005): Landslide during the earthquakes on May 26 and July 26, 2003 in Miyagi, Japan, *Soils and Foundations*, **45**(4), 149-163.

[33] Boulanger, R.W. and Truman, S.P. (1996): Void redistribution in sand under post-earthquake loading, *Canadian Geotech. J.*, **33**, 829-834.

[34] Sento, N., Kazama, M., Uzuoka, R., Ohmura, H. and Ishimaru, M. (2004b): Possibility of Post-liquefaction Flow Failure due to Seepage. *ASCE, J. of Geotech. and Geoenvironmental Eng.*, **130**(7), 707-716.

[35] Japanese Society of Civil Engineers, (1966): The report of damage investigation in the 1964 Niigata Earthquake, (in Japanese).

[36] Mori, S., Shimizu, K., Suzuki, N., Takagi, M. and Nakamura, S. (1989): Measurement and discussion of permanent ground displacement due to liquefaction, *20th Japan earthquake engineering symposium*, 25-28 (in Japanese).

[37] Kazama, M., Sento, N. Uzuoka, R., and Ishimaru, M. (2008): Progressive damage simulation of the Showa Bridge caused by lateral spreading during the 1964 Niigata Earthquake. *Geotechnical Engineering for Disaster Mitigation, Springer, Proc. of the 2nd International; Conference GEDMAR08*, 171-176.

[38] Kazama, M., Shimizu, Y., Mori, T., Sento, N. and Uzuoka, R., (2007): Trace of Liquefaction under the Shinano river bank during the 1964 Niigata Earthquake. *Earthquake Geotech. Eng.*, (CD-ROM), KYRIAZIS D. PITILAKIS Editor, Paper No.1209.