13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 492

LABORATORY TEST FOR DEFORMATION CHARACTERISTICS OF SAND UNDER LEVEL 2 GROUND MOTION

Nozomu YOSHIDA[1], Susumu YASUDA[2] Hiroyoshi KIKU[3], and Sumio SAWADA[4]

SUMMARY

Laboratory test methods are investigated and discussed in order to grasp stress-strain behavior of sand to be used in dynamic response analyses of ground under level-2 ground motion (very severe ground shaking) by which accurate prediction of ground displacement is required. Existing laboratory test methods such as dynamic deformation characteristics test and liquefaction strength test are shown to be insufficient to grasp them from small strains up to large strains where liquefaction occurs or even more, which indicates that a new test method must be developed. Considering the number of specimen in the engineering practice, it is necessary to grasp the range of stress-strain behavior during actual earthquakes and a new test is conducted focusing on that range. A new method to trace the stress-strain behavior is proposed, by which stress, strain and total amount of past loading history can be expressed in one plane. Through the case study, it is found that behaviors at larger stresses than that in the conventional test is necessary, but number of loading cycle may be smaller than that in the conventional test.

INTRODUCTION

After the 1995 Hyogoken-nambu earthquake (Kobe earthquake), performance-based designs are becoming to be employed in various design specifications under the level-2 ground motion or very severe ground shaking. In this design, a structure is allowed to deform or to enter nonlinear range and is also allowed to collapse partially if required performance is maintained. Under this situation, deformation of ground during and after earthquakes has become one of the important issues. For example, settlement is one of the main interests in the design of river dikes. Piles have been damaged at the intersection between soft and stiff layers; therefore, horizontal displacement is necessary to design. In order to predict displacement during and after the earthquake, it is necessary to know stress-strain relations from small strains to large strains at which the ground fails by, for example, soil liquefaction.

Both the dynamic deformation characteristics test and the liquefaction strength test are usually carried out in the engineering practice in order to grasp or obtain stress-strain behaviors of sand during earthquakes.

[1] Managing Chief Engineer, Oyo Corporation, Saitama city, Japan, yoshida-nozomu@oyonet.oyo.co.jp
[2] Professor, Tokyo Denki University, Saitama, Japan, yasuda@g.dendai.ac.jp
[3] Professor, Kanto Gakuin University, Yokohama, kiku@kanto-gakuin.ac.jp
[4] Professor, Kyoto University, Kyoto, sawada@catfish.dpri.kyoto-u.ac.jp

Secant shear modulus and damping ratio are computed as a function with respect to shear strain amplitude in the dynamic deformation characteristics test. This test is applicable at strains up to a little larger than 0.1%. On the other had, behaviors around several percent strain are obtained by the liquefaction strength test. Since only liquefaction strengths are of main interest in this test, stress-strain relations are out of conventional output. Therefore, it seems impossible or very difficult to grasp the stress-strain behavior from small strains up to large strains where performance-based design requires, especially under level-2 ground motion where the ground may fail.

One of methods to grasp the behavior at large strains is to make additional series of test. In the engineering practice, however, number of test specimens is limited to be around 5 considering the sampling of test specimen from a borehole. Since dynamic deformation characteristics uses one specimen and the liquefaction strength test uses 4 specimens, there may be no test specimen for additional tests. It clearly indicates that a new test procedure must be developed. To do so, it is necessary to grasp the stress-strain range during earthquakes, because earthquake response analysis and post-liquefaction residual displacement analyses will use only this range.

In this paper, we intend to show how laboratory test should be improved in order to grasp the stress-strain behaviour at large strains, or an index to evaluate necessary stress-strain behavior.

EXISTING PROBLEM OF CONVENTIONAL TEST AND ITS IMPROVEMENT

According to Japanese Geotechnical Society [1], the dynamic deformation characteristics test is conducted by, so-called, stage test method. Cyclic shear stress with constant amplitude is applied 11 cycles, and hysteresis loop at 10 cycle of loading are used to evaluate nonlinear properties. Shear modulus *G* is computed from the slope of the line connecting two strain reversal points. Damping ratio is computed from the area of the same hysteresis loop by dividing the strain energy. The test starts under small shear stress amplitude. Then, the shear stress amplitude increased a little. The same procedures are repeated until hysteresis loop becomes unstable. If excess porewater generates during one stage of loading, it is dissipated before moving the next stage. The hysteresis loop becomes stable under 10 cycles of loading when stress amplitude is small. However, it becomes unstable when stress amplitude becomes large, which occurs at strains a little larger than 0.1 %. An example is shown in Figure 1 in which shear strain increase as cycles although shear strain amplitude is less than 1 %.

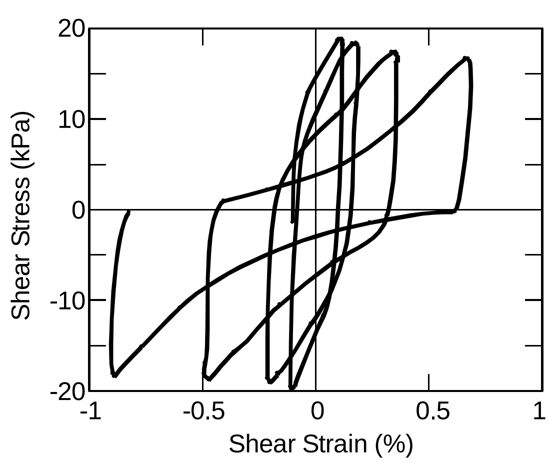


Figure 1 Examples of unstable hysteresis loop in the dynamic deformation characteristics test.

Several shortages come up from this procedure. The one is that hysteresis loop at the 10th cycles of loading may not be relevant to be used in the dynamic response analysis. Hysteresis loop at the first cycle and the 10th cycle are different from each other and the first cycle behavior is more important because maximum strain occurs at the first cycle. We will not discuss it partly because it is not the main topic of this paper and partly because it was already discussed by one of the authors [2] with some topics related to the dynamic deformation characteristics.

Another shortage comes from the drainage of excess porewater pressure between each stage. This procedure directly indicates that excess pore generates during loading, which yields instability of hysteresis loop at large strains. It can be recognized in Figure 1 where hysteresis loop is not an ordinary spindle shape, but an inverse S shape. This shape is similar to the one during liquefaction strength test. Therefore, it is obvious that these behaviors come from dilatancy characteristics of sand, and also indicate that excess porewater pressure plays an important role in the stress-strain behavior. Drainage of the excess porewater pressure results in densification of soil samples. Therefore, it will not show the behavior of original specimen.

A typical example is shown in Figure 2, in which result of the dynamic deformation characteristics test and the liquefaction strength test for Toyoura sand with 50 % relative density are compared. Here σ_d indicates axial stress amplitude, and σ'_{m0} is initial effective confining stress. Horizontal axis is named "Number of cycles causing liquefaction" following the conventional data compile method in the liquefaction strength test, but actual meaning is "number of cycles causing specified shear strain single amplitude in this situation because strains much smaller than liquefaction is also drawn in the figure.

Solid lines come from the liquefaction strength test. Numerals from 0.3 to 9 are single strain amplitude in percent. Liquefaction strength ratio, stress ratio at which liquefaction occurs under just 20 cycles of loading, is about 0.2.

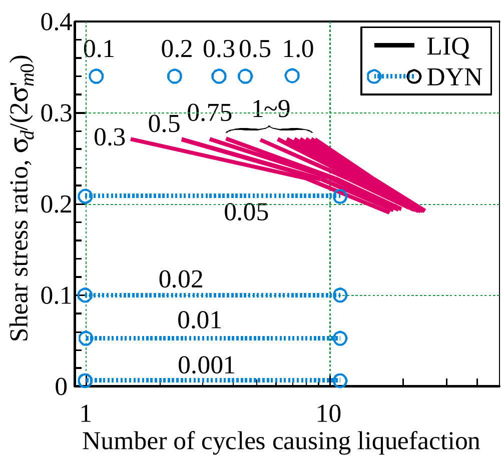


Figure 2 Comparison of dynamic deformation characteristics test and liquefaction strength test.

On the other hand, hollow circles and dashed line come from the dynamic deformation characteristics test. Numerals again indicate shear strain single amplitude in percentage. Hysteresis loops in 11 cycles of loading at each stage are nearly identical (hysteresis loops stabilize) up to shear strain amplitude 0.05. Therefore, results in these stages are shown as horizontal lines with hollow circle at both ends that corresponds to the first and the 11th cycles. However, shear strain begins to increase in each cycle when shear stress amplitude is 0.35. Shear strain amplitude is 0.1 % at the first cycle and it increases gradually to 1 % under the subsequent loading until test terminates, which are shown in hollow circles.

Let's focus on the behavior at shear strain amplitude of 0.5 %, for example. The dynamic deformation characteristics test indicates that this strain occurs after 5 cycles of loading under shear strain ratio amplitude of 0.35. Oh the other hand, result of the liquefaction strength test indicates that this strain occurs after 5 cycles of loading under strain ratio amplitude of 0.24. The dynamic deformation characteristics test gives higher strength than the liquefaction strength test, and it is obvious that this overestimation comes from the drainage of excess porewater pressure at each stage.

As shown above, stress-strain behavior obtained in the conventional test is not satisfactory from the point of view to evaluate deformation during and after the earthquake. Stress-strain behavior at large strains is not obtained. In addition, the dynamic deformation characteristics test and the liquefaction strength test do not give continuous feature of stress-strain behavior around a few percent strain range, too.

A simple idea to improve the test methods is to conduct fresh test same with the liquefaction strength test in the dynamic deformation characteristics test. However, it is not a realistic solution. It requires huge number of test specimen. On the other hand, it is impossible to take such number of test specimen from in-situ; number of samples should be 5 or 6 at maximum. This number is same with the number of test specimen that conventional tests use. Therefore, it is impossible to increase number of test specimens.

Another idea is to conduct test under the loading condition that will happen in the actual earthquake. In order to do so, we need to know shear stress amplitude and number of cycles under actual earthquakes. Unfortunately, there is no such report. In this paper, we are going to give an idea to measure it.

STRESS-STRAIN PATH MODEL

As discussed in the preceding section, it is necessary to grasp the relationship between the number of cycles and stress-strain behavior. We propose a method to draw stress-strain paths in the liquefaction strength expression.

Expression of state point: contour of strain in liquefaction strength curve plane.
Number of cycles or past loading history is important in the liquefaction behavior. Therefore, we need to

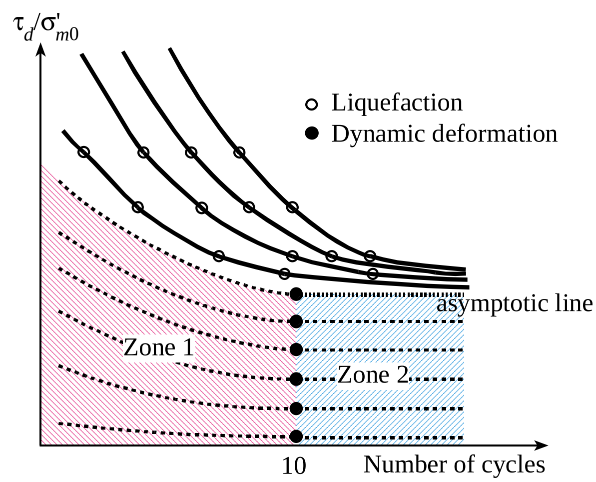


Figure 3 Contour line of single strain amplitude in stress ratio-number of cycles plane

grasp stresses, strains and number of loading cycles at the same time. We prefer the expression used in the liquefaction strength curve to express three indices at the same time.

Figure 3 shows shear stress ratio versus number of cycles relationships schematically. The figure is similar to the figure obtained by the conventional liquefaction strength test, but a little different. Shear strain contour lines are drawn based on double amplitude in the conventional test, whereas single amplitude is used in this figure. Moreover, not only result of the liquefaction strength test but also result of the dynamic deformation characteristics test is overdrawn. Zones 1 and 2 are regions where data are obtained from the dynamic deformation characteristics test and the rest from the liquefaction strength test.

Conventional output of the dynamic deformation characteristics test and the liquefaction strength test are not sufficient to fill this plane. For example, stress-strain relation is output only at the 10th cycle in the conventional test. This result is expressed by just a point shown in solid circle in Figure 3. Therefore, we need to evaluate the rest, which does not seem difficult. If hysteresis loop is stable under 10 cycles of loading as conventional dynamic deformation characteristics test assumes, contour line is horizontal in the region larger than 10 cycles (Zone 2). Counter line in zone 1, a zone below the lowest liquefaction strength curve and number of cycles smaller than 10, can be drawn when all the hysteresis loops during the dynamic deformation characteristics test are given. Even if they are not given, we can interpolate it easily by, for example, interpolating from the lowest liquefaction strength curve. An asymptotic line may exist between zone 2 and region of the liquefaction strength test toward which liquefaction strength curve asymptotic.

Hysteresis rule

Next, let's assume that constant strain contour lines are fixed in this plane, or are uniquely defined as material characteristics. Then, one can follow the movement of state point by the figure shown in Figure 4(a) and Figure 5(a), in which vertical axis (shear stress ratio axis) has both positive and negative value and contours of strain are drawn in both side.

Let's follow the move of state point under simple loading conditions. Behavior under constant shear stress amplitude test is shown in Figure 4. Here, horizontal axis is modified to number of half cycles to make expression simple. Loading starts from point A, i.e., zero shear stress and 1 half-cycle loading. State point moves vertically as shear strain increases. Direction of loading is reversed at point B where shear stress reaches shear stress amplitude τ_a. Contour line passing the point B is drawn in solid line; the same contour line is drawn in the negative side, too.

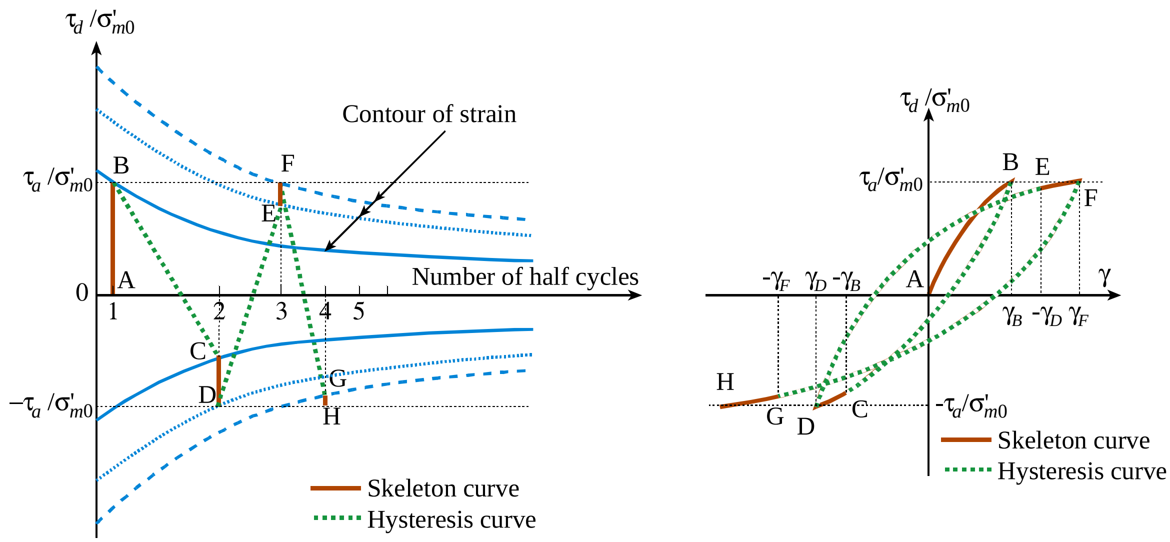
After unloading, state point moves toward point C which is located on the contour line with shear strain at unloading point, and number of half cycles is 2. In the sense of conventional stress-strain model, state from A to B corresponds to skeleton curve as schematically shown in Figure 4 (b). On the other hand, state from B to C corresponds unloading curve or hysteresis curve. This situation is again easily understood by Figure 4 (b). The difference between skeleton and hysteresis curve is that stress and stress can be expressed in the contour lines in Figure 4 when the state point lies on the skeleton curve, whereas it need not in the hysteresis curve. In other words, stress-strain relationships on the skeleton curve can be expressed in Figure 4 (a), but cannot be for the hysteresis curve. So, linear line is used for the path from B to C. After reaching the contour line whose shear stress is same with previous unload from the skeleton curve, the state point moves toward D along the skeleton curve.

In the same manner, move of subsequent state point can be expressed by the combination of the skeleton and hysteresis curves as shown in lines E→F→G→H, during which shear strain increases gradually.

Another example is a loading under constant shear strain amplitude shown in Figure 5. Path from A to B is same with previous example; state point moves vertically at number of cycles 1. Path from B to C is also same with previous example. However, magnitude of shear strain at C is same with that at point B; therefore, unload occurs. In other words, state point just enter the skeleton curve and then apart from it. This situation keeps for subsequent loading from C to E. Then, shear stress decreases gradually as loading continues.

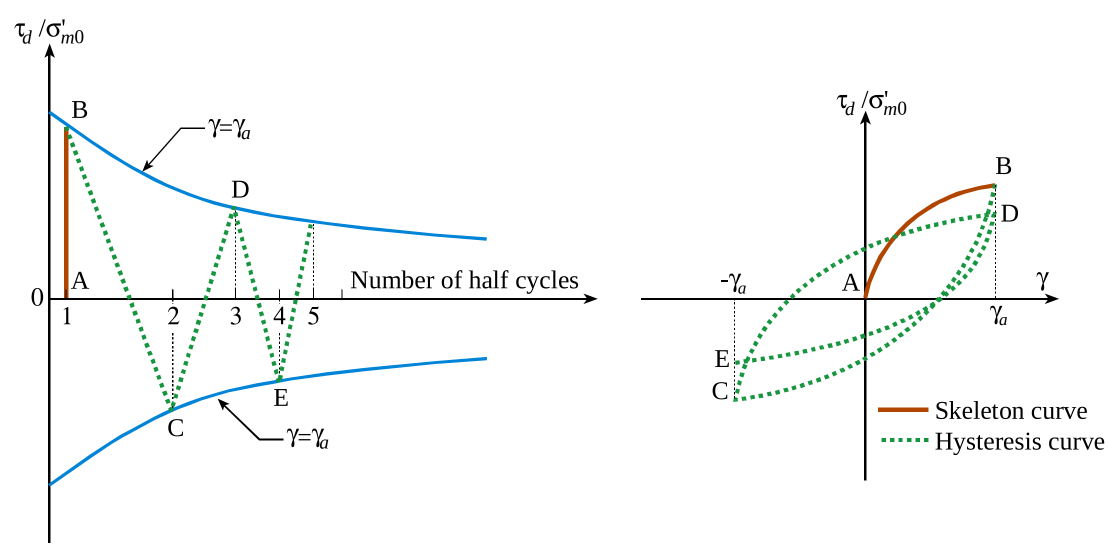
Rule for unloading and reloading

In the preceding section, behavior is defined when unloading occurs from the contour line or skeleton line. However, it is not sufficient to describe the whole behavior because unload or reload may occur from the



(a) Path of state point (b) Stress-strain curve.

Figure 4 Paths during constant stress amplitude test.



(a) Path of state point (b) Stress-strain curve.

Figure 5 Paths during constant strain amplitude test.

hysteresis curve. Figure 6 shows movement of the state point in such situation schematically. Here, state point moved from Q towards B that is located on the contour line and unloaded at point A and reloaded at point P. Points B, C, D and E lies on the contour line and are target points for each path as easily seen in Figure 6. Number of half cycles at point E is larger than that at point C by 1, which can be recognized from the constant shear strain amplitude case in Figure 5.

The target point D when unload occurs at point P lies on the contour line as described in the preceding. Therefore, location of point D can be determined if the number of half cycles is specified. Linear interpolation is employed in evaluating it, which yields

$$\frac{N_P - N_A}{N_C - H_A} = \frac{N_D - N_B}{N_E - H_B}$$

where N is number of half cycles and subscript indicates point in Figure 6. Since all the numbers of cycles except N_D are known quantities when unload from point P occurs, location of the target point D can be computed.

Change of state point can be completely described by applying the rules in the preceding and this sections.

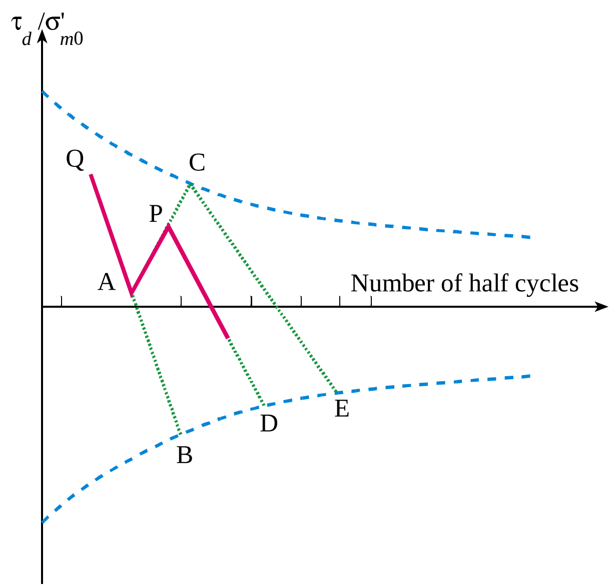


Figure 6 Schematic figure showing the unloading and reloading rule.

NUMERICAL EXAMPLE

The Port Island vertical array site is examined. Figure 7 shows location of the investigated site. The site is a man made island located at Kobe city, Japan. Significant soil liquefaction was observed during the 1995 Hyogoken-nambu earthquake (See reference [3] in detail of damage, for example). Soil profile is shown in Figure 8. Fill material, Masado or decomposed granite, was liquefied; thickness of Masado is about 18 m.

Four sets of seismographs were installed as shown in Figure 8 and vertical array records were obtained during the 1995 Hyogoken-nambu earthquake.

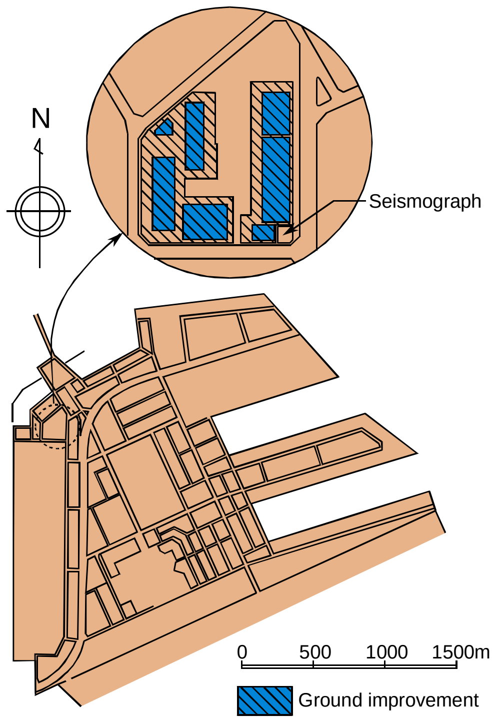


Figure 7 Investigated site

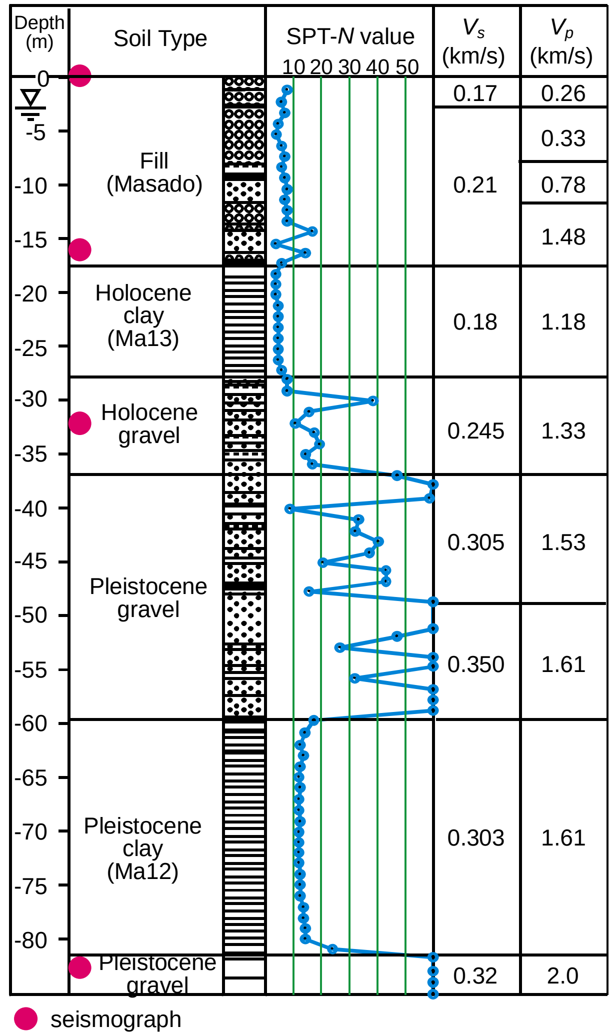
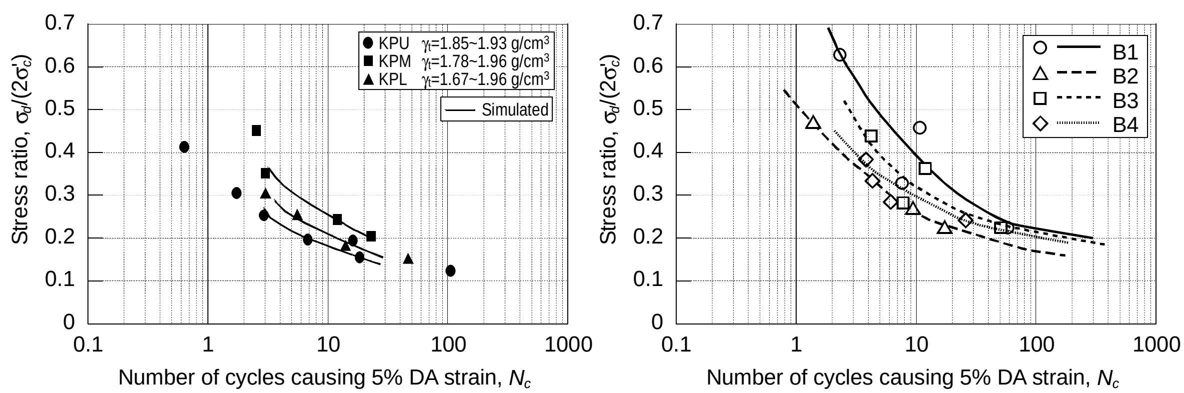


Figure 8 Soil profiles at vertical array site



(a) Unimproved ground

(b) Improved ground

Figure 9 Liquefaction strengths

There were site improved by rod compaction method just neighboring to the site as shown in Figure 7, and suffered much smaller damage compared with surrounding unimproved ground. Then a detailed investigation at this site was made after the earthquake [4][5]. On the other hand, detailed soil investigation was made in the unimproved ground at about 70 m south to the site by Hatanaka et al. [6] after the earthquake, too. Liquefaction strengths obtained by these studies are shown in Figure 9.

Effective stress analyses were made at these two sites [7][8] by YUSAYUSA-2 [9]. Responses at the layer where shear strains were largest are used in the analysis. Figure 10 is an example of the stress-strain curve and stress path. It is noted that not only this analysis but also other analyses (e.g. [4][8]) showed that liquefaction occurred in the improved ground at deep depth although it did not occur near the ground surface. These results seem to be consistent with observation because ground surrounding the pile-supported building was found to settle although ground deformation was not observed during the earthquake.

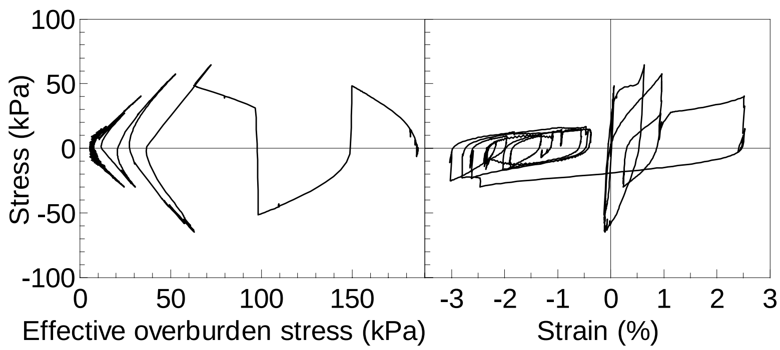
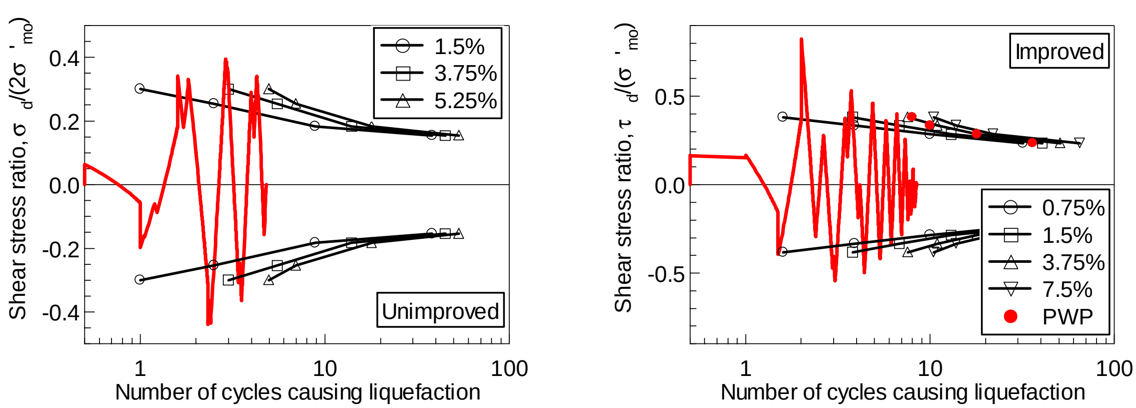


Figure 10 Example of stress-strain curve and stress path



(a) Unimproved ground

(b) Improved ground

Figure 11 State path trajectory

State path trajectories are shown Figure 11. It is noted that contour of shear strain (or liquefaction strength) is drawn based on single amplitude shear strain as explained in the preceding. Maximum shear stress ratio is larger than 0.4 in the unimproved ground. However, the liquefaction strength test was carried up to shear stress ratio amplitude of about 0.3. Same situations, i.e., state point lie outside the liquefaction strength curves, are observed at several points. This indicates that shear stress amplitude must be larger than actual test in order to describe the whole behavior during earthquake. It is also noted that maximum number of cycles is about 5, which is smaller number compared with the cycles that conventional liquefaction strength test is conducted.

The same situations occur in the case of improved ground. The liquefaction strength test was carried out in larger shear stress ratio than previous, but it is not sufficient to grasp whole behavior, because maximum shear stress is larger than maximum shear stress amplitude. It is also noted that maximum number of cycles in the improved ground is larger than that of unimproved ground, but it is still less than 10. Again number of cycles is smaller than that by conventional liquefaction strength test.

CONCLUDING REMARKS

A method to trace the move of the state point is proposed by which effect of past loading can be taken into account in the stress-strain behavior.

Case studies are made at both improved and unimproved ground at Port Island, Japan, where vertical array earthquake records were obtained during the 1995 Hyogoken-nambu earthquake, and detailed soil tests were made.

Result of the analysis indicates that shear stress amplitude must be larger than conventionally conducted. On the other hand, number of cycles can be smaller than that by conventional test.

ACKNOWLEDGMENTS

This study was performed through the Special Coordination Funds, titled ‘Study of the master model for strong motion prediction toward earthquake disaster mitigation’, of the Ministry of Education, Culture, Sports, Science and Technology of the Japanese Government.

REFERENCES

1. Japanese Geotechnical Society "Methods and commentary of soil test, 1st revise." 2000 (in Japanese with English summary)
2. Yoshida, N. "Processing of strain dependent characteristics of soil for nonlinear analysis." Proceedings of the First International Conference on Earthquake Geotechnical Engineering, Tokyo: 473-478, 1995
3 Soils and Foundations "Special Issue on Geotechnical Aspects of the January 1995 Hyogoken-Nambu Earthquake.": Japanese Geotechnical Society: No. 2, 1998
4 Geotechnical Research Collaboration Committee on the Hanshin-Awaji Earthquake Disaster "Report of Activity", 347p., 1998 (in Japanese)
5 Ito, K., Goto, Y., Ishihara, K., Yasuda, S. and Yoshida, N. "Detailed in-situ and laboratory tests on the improved ground in Port Island." Proceedings of the 2nd International Conference on Earthquake Geotechnical Engineering, Lisboa, Portugal: 47-52, 1999
6 Hatanaka, M., Uchida, A. and Ohara, J. "Liquefaction characteristics of a gravelly fill liquefied during the 1995 Hyogo-ken nanbu earthquake." Soils and Foundations; 37 (3), 107-115, 1997
7 Yoshida, N. "Earthquake response analysis at Port Island during the 1995 Hyogoken-nanbu earthquake." Tsuchi-to-Kiso; 43 (10), 49-54, 1995 (in Japanese)
8 Yoshida, N. and Ito, K. "Liquefaction of improved ground at Port Island, Japan, during the 1995 Hyogoken-nanbu earthquake." Proceedings of the 2nd International Conference on Earthquake Geotechnical Engineering, Lisboa, Portugal: 1999, 531-536
9 Yoshida, N. and Towhata, I. "YUSAUISA-2 and SIMMDL-2 theory and usage", 1991 revised in 2003