EFFECT OF THE GRID-SHAPED STABILIZED GROUND IMPROVEMENT TO LIQUEFIABLE GROUND

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SUMMARY

Effective countermeasures for liquefaction of sandy ground under or adjacent to existing structures are very limited. The authors have developed the grid-shaped stabilized ground improvement method using such procedures as Deep Cement Mixing and the jet grouting method as countermeasures for prevention of ground settlement caused by liquefaction during a strong earthquake. It was shown that these countermeasures are effective for reduction of excess pore water pressure, however, the effectiveness for prevention of permanent deformation or settlement was not well understood. The authors have attempted to elucidate the mechanism of the improvement method, and develop the design procedures for this kind of improvement.

In the first stage of the investigation, we conducted a shaking table test associated with the basic concept of the grid-shaped improvement method. The numerical analysis method using the 3 dimensional dynamic effective stress FEM procedure (code name, "EFECT") was verified by the results of the shaking table test. The second stage of the investigation was the application of this improvement procedure to the ground prototype. In this stage, the variable of the investigation was the distance between the walls.

The test results obviously indicated the effectiveness of the improvement method. It was shown that the liquefaction resistance of the saturated sandy ground increases in proportion with the distance between the walls.

The numerical simulation analysis coincided with the test results concerning not only the maximum pore water pressure built in the improved ground, but also the settlement of the ground surface. The numerical analysis results associated with the ground prototype indicated that the maximum settlement of the ground decreased by about 30% of the liquefied free field ground.

INTRODUCTION

The purpose of the grid-shaped ground improvement (GGI), as a countermeasure for liquefaction, is to reduce shear deformation due to the enclosure by the grid walls installed using procedures such as the deep cement mixing method in liquefiable layers. Design criteria for GGI have included values of excess pore water pressure ratio and settlement of the ground surface. It is thought that these design criteria affect the strength and thickness of the walls, or spacing between the underground walls.

In the present investigation, a large scale shaking table test was performed to confirm the effectiveness of GGI for the first stage. Numerical analysis, such as dynamic effective stress analysis, is very useful for directly determining the strength of the walls and the arrangement of the grid.

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The effectiveness of the Dynamic FEM analysis code (EFECT) developed by the authors was verified using the shaking table test data in the second stage. The applicability of the numerical analysis method to a prototype model of saturated sandy ground and the optimum design method for GGI is presented as the last stage of the investigation.

A SHAKING TABLE TEST OF THE GRID-SHAPED IMPROVEMENT METHOD

A large scale shaking table test was conducted to confirm the basic effect of the grid-shaped ground improvement method for the prevention of liquefaction [1].

Fig.1 shows the profile and plane of the test case. The geometrical scale factor of the test was set up to 1/10. For the grid-shaped improvement, underground walls using lightweight reinforced concrete of 1.0m depth were installed in the ground model at a depth of 1.3m. The ground model was contained in a rigid steel box of 8.0m length by 1.0m width.

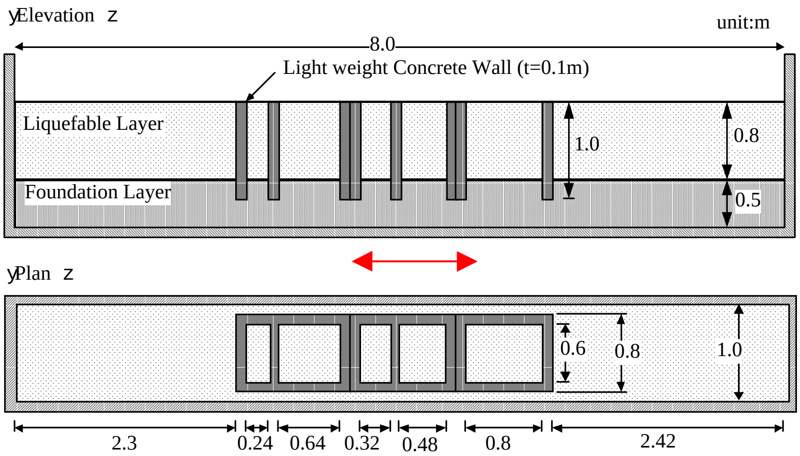

Fig.1 The profile and planes for large scale shaking table test

Table 1 Material properties for TOYOURA-sand

SPECIFIC GRAVITY OF SOIL PARTICLE	2.653
Maximum dry density (t/m^3)	1.663
Minimum dry density (t/m^3)	1.357

Table 2 Mechanical Properties For Each Layer

	Liquefiable Layer	Foundation Layer
Wet density	1.95	2.2
Velocity of shear wave (m/s)	80-85	180

The ground model consisted of impermeable stiff layers as a base foundation (lower layer of 0.5m thickness) and a soft liquefiable layer (upper layer of 0.8m thickness). The foundation layer, consisting of crushed stone mixed with TOYOURA-sand, was scattered and compacted well after installation of the GGI model.

The liquefiable sandy layer was made from saturated TOYOURA-sand. Table1 tabulates material properties for TOYOURA-sand, and Table 2 tabulates mechanical properties for each layer. The wet density was measured from dry samples using air scatter, under the assumption that the ground model was saturated. The velocity of the shear wave was measured using the plank hammering method.

The grid-shaped walls, made of lightweight reinforced concrete, was 0.1m in thickness, 1.92 g/cm^3 in density, and 0.2m in penetration depth to the foundation layer. Intervals between the walls in terms of "L" were applied from 0.24m to 0.80m. The ratio of L to a depth of the liquefiable layer in terms of "L/H" was set up from 0.3 (=0.24m/0.8m) to 1.0 (=0.8m/0.8m). Sinusoidal waves of 5Hz were applied to the model for 4 seconds. The amplitudes of input motion were set up from 48~~-~~to 721cm/sec^2. The shaking direction is shown in Fig.1.
The results of the shaking table test were compiled in the form of the following remarks:

1) The outer ground of enclosure (thought of as a free field) liquefied due to input motion over 166cm/sec^2.
2) The inner ground of the enclosure did not liquefy under the input motion of 200 cm/sec^2 ,despite the intervals between the walls.
3) For the cases in which the amplitude for input motion was over 238cm/sec^2, the excess pore water pressure ratio at the inner ground of the 0.24 and 0.32m grid cases built up insignificantly. On the contrary, that of the grid intervals over 0.48m built up due to the same input motion.

DYNAMIC EFFECTIVE STRESS ANALYSIS OF THE SHAKEING TABLE TEST

The dynamic effective stress FEM analysis is an appropriate method to verify the effectiveness in prevention of liquefaction in the ground prototype. However, it is unknown whether the analysis model~~s~~ can simulate the effect of restraint caused by shear deformation by the grid walls in estimating the precise excess pore water pressure in the ground. Therefore, we performed a simulation analysis for the shaking table test to verify the analytical modeling method of GGI. The FEM code employed to simulate the test result was "EFECT" [2)]. It is based on the constitutive model originally proposed by Matsuoka, incorporated with the Bio's formulation. EFECT is useful in predicting permanent deformations, such as ground settlement due to liquefaction, as well as accumulation and dissipation of excess pore water pressure.

The FEM model represented a unit enclosure by grid walls. Two-dimensional plane-strain models were adopted for ground and walls elements as shown Fig.2 (presenting the case of the 0.32m grid as an example). Although a unit grid has 3-dimenssional effect as shown in Fig.3, the present 2-dimensional FEM model has to represent the confining effect of the walls using each element of the ground and the walls. The double elements consist of an A-A' line and a B-B' line, as shown in Fig.3. The cross section of A-A' includes a liquefiable layer, a foundation layer and walls perpendicular to the shaking direction, while the cross section of B-B' line includes a foundation layer and walls parallel to the direction of shaking. The nodal points on the side boundaries and base boundary of two separate planes were jointed together.

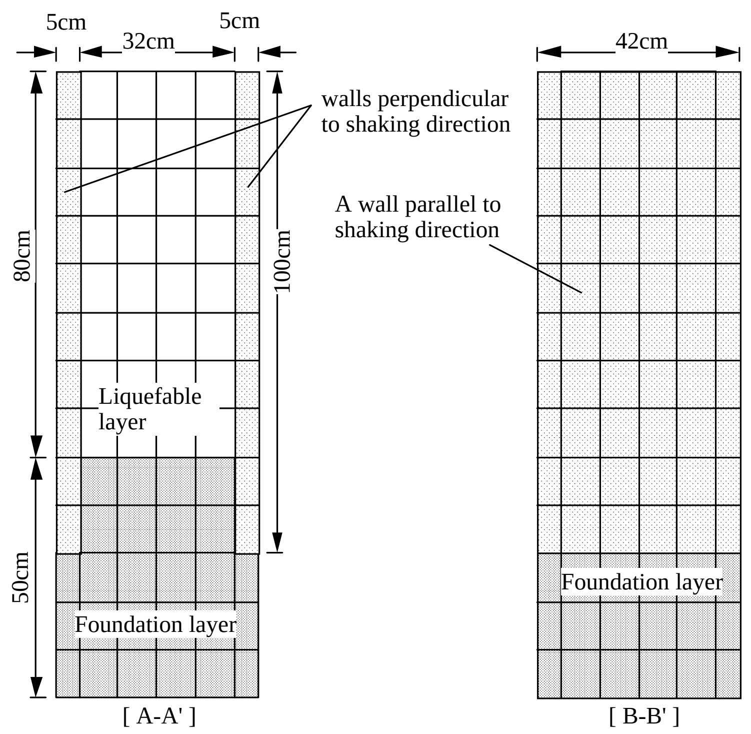

Fig.2 FEM model

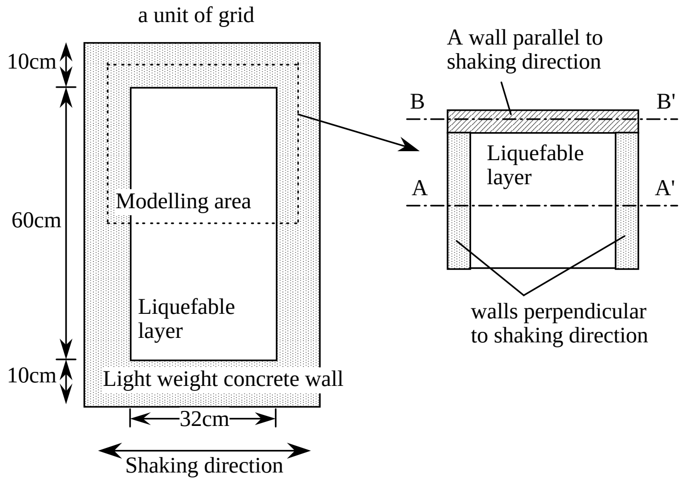

Fig.3 A unit grid of GGI

In addition to the walls perpendicular to the shaking direction, the wall parallel to the shaking direction influences the shear deformation of the liquefiable layer in this model. For this reason, the model can reasonably simulate the effect of shear deformation restraint by the grid walls. However, it is necessary to show how the direction of the wall affects the changes. The elemental thickness of the walls perpendicular to the shaking direction was defined as 1.0. In this case, the relationship between the elemental thickness of the wall and the distance between square in the grid is defined as (W-2s)/W; where W is the distance, and s is the thickness.
Both sides of the model were assumed to have a continuum boundary, and the base of model was assumed to be a viscous boundary. Though these boundary conditions are strictly different from the conditions of the shaking table test, it is a reasonable and simple method for the modeling of grid-shaped improvement.

Table 3 Cases of analysis (Shaking table test models)

No.	Case	The grid size (grid size/depth of liquefiable layer)
(1)	Free field model	
(2)	Soil Improvement model	32cm (0.4)
(3)	Soil Improvement model	64cm (0.8)
(4)	Soil Improvement model	80cm (1.0)

Table 4 Material properties for the analysis

	Liquefiable layer	Foundation layer	Concrete wall
Unit weight (kN/m^3)	19.12	21.56	18.82
Young's modulus of grains E (kN/m^2)	33214	212414	21600000
Poisson ratio of grains ν	0.33	0.33	0.20
Angle of Internal friction ϕ (DEG)	35	40	-----
Initial Void ratio e0	0.739	0.510	-----
Rayleigh damping of skelton αs	0.0	0.0	0.0
Rayleigh damping of skelton βs	0.00153	0.00153	0.00153
Dilatancy parameter λ	1.2	1.2	-----
Dilatancy parameter μ	0.213	0.280	-----
Coefficient of permeability k (m/s)	0.0006	-----	-----
Hardening parameter ks	0.002	0.000001	-----

The liquefiable layer was assumed to be nonlinear and consist of two phase (soil particle and water) materials.

The foundation layer was assumed to be nonlinear and consist of only one phase (soil particle). The grid walls were assumed to be linear and consist of one phase. Then, the drained boundary was assumed to consist only of the surface of the liquefiable layer.
Prior to the experiment, ground settlement was estimated with the dissipation of excess pore water pressure. However, the joint elements were not installed between the liquefiable layer and the grid walls parallel to the shaking direction. The reason for this being that if the liquefiable layer and the walls behaved individually, there was a possibility that the amount of ground settlement was overestimated [3)]. There was no friction between the liquefiable layer and the wall perpendicular to the shaking direction.
Table 3 shows the analysis data for a free field model and three improvement models. Material properties for the analysis are tabulated in Table 4. These parameters were defined by the element test as well as by observation. The input motion consisted of 5Hz sinusoidal waves with amplitude of 316cm/sec^2.
Fig.4 and Fig.5 show the results of this analysis. Fig.4 shows the time dependency of the excess pore water pressure ratio in the liquefiable layer at the elemental depth of 0.4m, and in the center of two walls. The free field model liquefied because the excess pore water pressure ratio reached 1.0. The improvement models indicated the effectiveness of liquefaction prevention, because the excess pore water pressure ratio in the improvement models was restricted to below 0.8.
The restraining effect in the excess pore water pressure ratio depends on the distance between the walls; the capacity increasing the smaller this distance becomes. Therefore, it is thought that the restraining effect in the excess pore water pressure ratio was influenced by the shear deformation of the liquefiable layer. It is thought that the walls increased the ground rigidity, since the oscillating component, in response to the excess pore water ratio, coincides with input motion.
Fig.5 shows the time dependency of the ground settlement, and compares the results of the free field model with those of the improvement models. The results of the improvement models indicate the average at every nodal point on the ground surface. The restraining effect in the residual settlement depends on the distance between the walls, the capacity increasing as the distance narrows. This tool can be used to make a comparison between the case of grid size 0.32m and the cases of grid sizes 0.64 and 0.80m.
These characteristics found in the results of the analysis were also indicated in the results of the shaking table tests. Therefore, it is thought that the analysis models are suitable to verify the effectiveness of soil improvement methods like GGI. However, the absolute values of the excess pore water pressure and ground settlement differ slightly between the results of the analysis and the results of the shaking table tests as shown in Fig.6. It is thought that the determined method of the parameter and the assumption of the boundary conditions are the reasons for this.

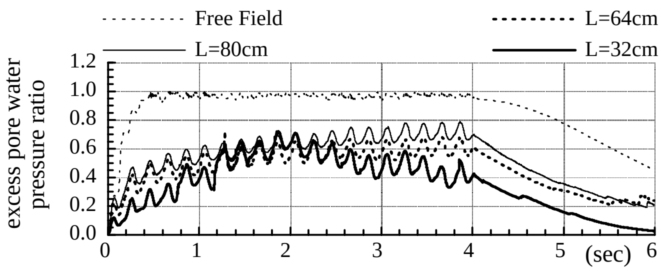

Fig.4 Results of analysis for shaking table test (excess pore water pressure ratio)

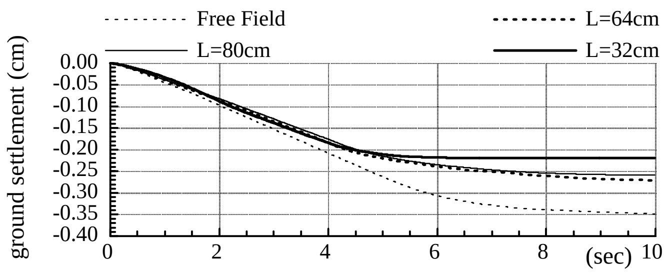

Fig.5 Results of analysis for shaking table test (ground settlement)

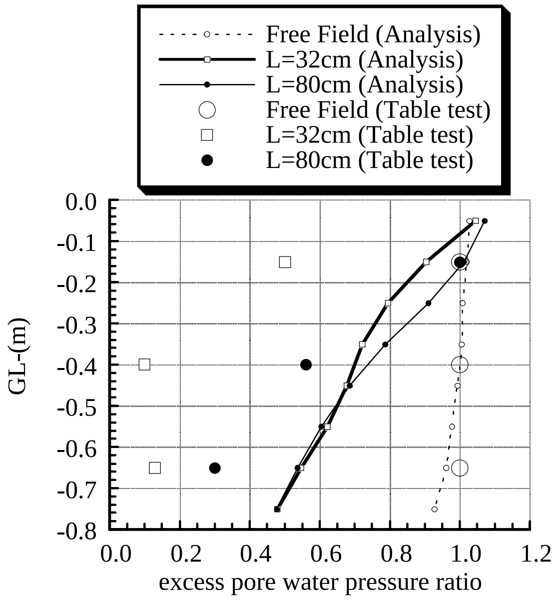

Fig.6 Comparisons of maximum excess pore water pressure ratio between Analysis and Table tests

DYNAMIC EFFECTIVE STRESS ANALYSIS OF THE PROTOTYPE GROUND

The authors verified the analysis models through the simulation of the shaking table tests. In the next stage, the design procedures for the grid size were examined by the analysis of the prototype ground models.

The layer constituents and the grid shapes of the prototype models were the same as that of the shaking table test ground model. However, the variables such as the layer depth, wall depth, grid thickness, distance between the walls, and other variables were 10 times that of the shaking table test ground model. The depth of the liquefiable layer was 8.0m, and the cases of analysis presented distances of 3.2m, 6.4m, and 8.0m between the walls. The soil parameters, determined by confining pressures such as the initial Young's modulus value, were set to suitable values for the prototype ground model.

The base motion at GL-83m of Port Island during 1995 the Hyogo-ken Nanbu earthquake, shown in Fig.7, was used for the experiment's input motion. The maximum amplitude was 610.7cm/sec^2. Table 5 shows the analysis of the cases.

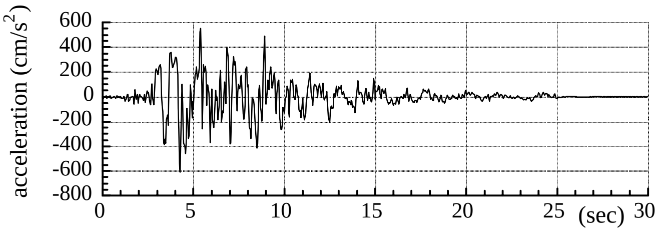

Fig.7 Input motion for prototype models

Table 5 Cases Of Analysis (Prototype Models)

No.	Case	The grid size (grid size/depth of liquefiable layer)
(1)	Free field model	
(2)	Soil Improvement model	3.2m (0.4)
(3)	Soil Improvement model	6.4m (0.8)
(4)	Soil Improvement model	8.0m (1.0)

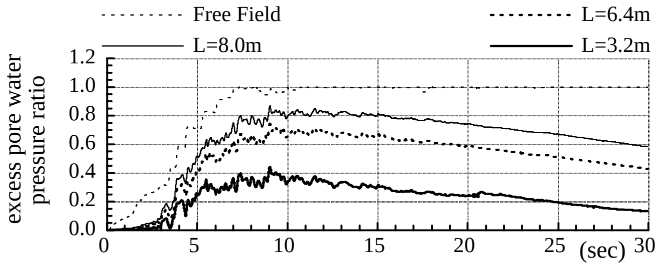

Fig.8 Results of analysis for prototype models (excess pore water pressure ratio)

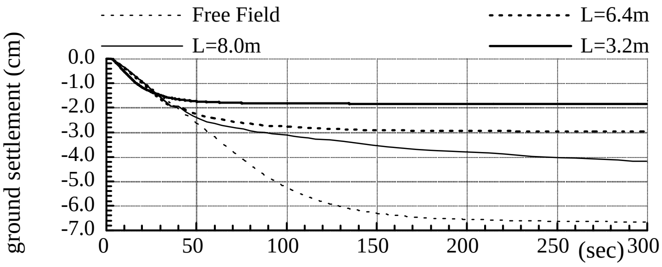

Fig.9 Results of analysis for prototype models (ground settlement)

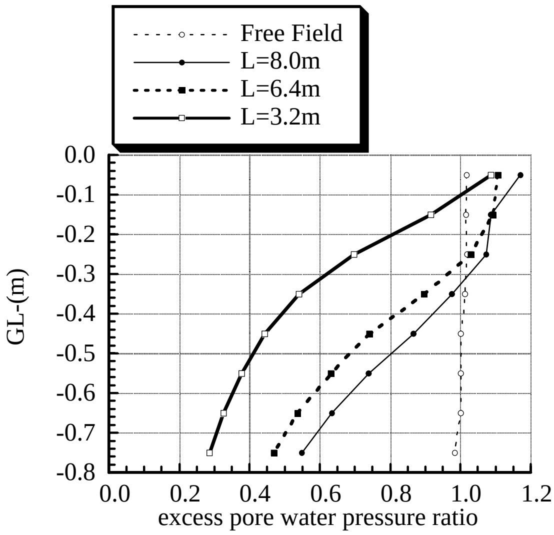

Fig.10 Results Of Maximum Excess Pore Water Pressure Ratio

Fig.8 and Fig.9 show the analysis results for the ground prototype. Fig.8 shows the time dependency of the excess pore water pressure ratio in the liquefiable layer at the elemental depth of 4.0m, and at the center of two walls. Fig.9 shows the time dependency of the ground settlement. In the cases of the improvement models, the walls prevented the increase of excess pore water pressure and ground settlement. The restraining effectiveness of the walls was dependent on the distance between walls, the capacity increasing as this distance narrows. The difference in the grid-size effectiveness was more apparent in this case than in the analysis results of the shaking table test. Fig.10 shows the distributions of the maximum excess pore water pressure ratio, and compares this with the free field model and the improvement models. The difference in the grid-size effectiveness was also indicated.

Fig.11 and Fig.12 show the relationship between the maximum response level and the grid size. The maximum values of the excess pore water pressure ratio and ground settlement is plotted on the vertical axis, and “L/H” is plotted on the horizontal axis. “L” is the distance between the walls, and “H” is the depth of the liquefiable layer. The vertical axis, in Fig.11, shows the mean values of the maximum excess pore water pressure ratio of the liquefiable layer. Fig.11 and Fig.12 show that the excess pore water pressure and the ground settlement increase linearly, as “L/H” increases. These results indicate the following design procedures:

(1) Set the limit value to restrain the excess pore water pressure ratio (or ground settlement).
(2) Analyze the free field model and the two improvement models using the FEM code.
(3) Plot the relationship between the excess pore water pressure ratio (and ground settlement) and “L/H”.
(4) Determine the most suitable value of “L/H” within the limits of the excess pore water pressure ratio (or ground settlement), using a linear relationship between the maximum response and the grid size.

Although effective stress analysis was used in this investigation, using total stress analysis, a suitable grid size can be assumed from the relationship between the maximum shear stress and “L/H” [4].

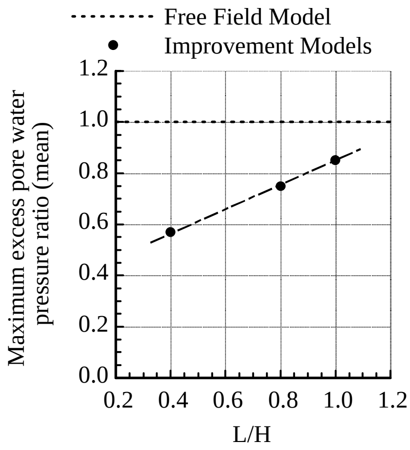

Fig.11 Relationship between the grid-size and excess pore water pressure ratio

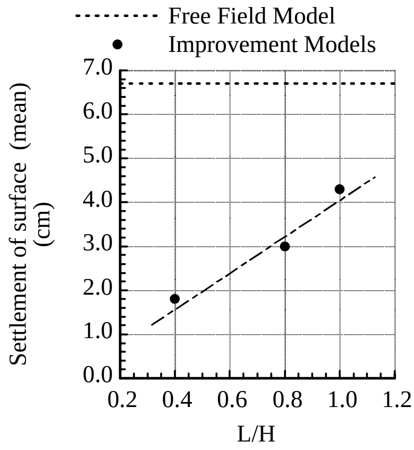

Fig.12 Relationship between the grid-size and ground settlement

CONCLUSIONS AND REMARKS

The design variables of the GGI system are the strength of the walls and the distance between the walls. Thus, it is necessary to estimate the effects of design prevention. However, since the quality of the ground inside of a grid does not improve, it is difficult to estimate the effectiveness of liquefaction prevention using in-situ or elemental test results. Therefore, a reasonable design requires numerical analysis to accurately estimate the effects.
The results of this investigation indicated that the relationship between the excess pore water pressure ratio (and ground settlement) and “L/H” is linear when the range of “L/H” is between 0.4 and 1.0. The authors intend to improve the analysis models and increase the number of simulation samples in order to store useful data for the GGI design.

EFERENCES

1. Public Works Research Institute and 5 firms (1991) ; Report of the Joint Research on Aseismic Soil Improvement Method, No.5, January 1991, Joint Research Report, No.52, Public Works Research Institute (in Japanese)
2. Ito,K. (1995) ; EFECT: The Code of Effective Stress Analysis (Part1) –Basic Theory and Constitutive Model of Soils-, Report of Obayashi Corporation Technical Research Institute, No.51,pp7-14. (in Japanese)
3. Sato,K. , Matsuda,T. (1998) ; Study on a relation between an effect of grid type soil improvement against liquefaction and the grid size, Proceedings of the 53th Annual Conference of the Japan Society of Civil Engineers, 3-(A), pp304-305. (in Japanese)
4. Furuya,H. , Sato,K. and Matsuda,T. (1999) ; Analytic investigations of the grid-shaped stabilized ground improvement as a countermeasure for prevention of liquefaction, Proceedings of the 25th JSCE Earthquake Engineering Symposium,Volume1, pp397-400. (in Japanese)