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INVESTIGATION OF THE COEFFICIENT OF EARTH PRESSURE FOR IMPROVED GROUND BY COMPACTION GROUTING IN THE FULL-SCALE FIELD LIQUEFACTION EXPERIMENT

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ABSTRACT

This paper describes the effect of countermeasures for liquefaction by compaction grouting, which was investigated by the experiment of full-scale field liquefaction by controlled blast technique. The experiment was conducted to assess the performance of airport facilities subjected to liquefaction, to investigate damage mechanism, and to estimate the effect of countermeasures for liquefaction by compaction grouting applied to liquefiable sand layer under runway pavement. In this study, before and after grouting and after artificial liquefaction caused by in-situ blasting, self boring pressure-meter tests at the center and the edge of a grouted area were carried out to investigate the coefficient of earth pressure, *K*, for evaluation of the improved ground because it is generally known that compaction grouting makes *K*-value increase in and around the grouted area. Additionally, to estimate the continuation of improving effect after liquefaction, K-values after blast were also investigated at same points. As the results of investigation, it was found that post-liquefaction *K*-value was higher than that of untreated ground before improvement and compaction grouting with cost-reduction design examined in this study, that is, the cost-reduction design is effective.

INTRODUCTION

The role of an airport during and after a great earthquake is important because of its helpful function for emergency medical services and special operations of transporting relief supplies to the earthquake disaster area. In order to investigate the influence of a great earthquake on the airport's function, a full-scale liquefaction experiment employed by controlled blast technique at the Ishikari Bay New Port in Hokkaido Island, Japan, was conducted. For this experiment, full-scale airport facilities including runway pavement were constructed in the experiment site. Its primary subjects were to assess the performance of airport facilities subjected to liquefaction, to investigate damage mechanism, and to estimate the effect of countermeasures for liquefaction. Especially two types of countermeasure for liquefaction, which were compaction grouting and permeable grouting, with a new design concept subjected to liquefaction due to large earthquake motions were tried to be used it for the experiment site.

This paper describes the effect of compaction grouting with cost reduction design to evaluate its aptitude for runway pavement by lower improved ratio and reduction of improved area compared with a usual design through investigation of the coefficient of earth pressure by the results of self boring pressure-meter tests and residual deformation of the runway pavement after liquefaction.

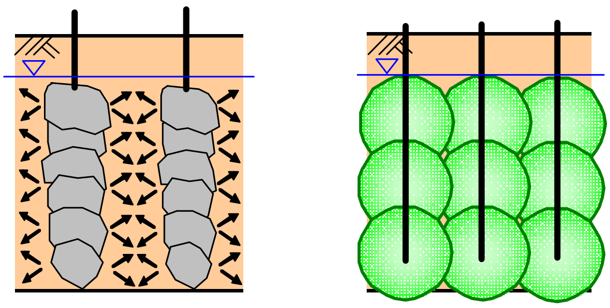
CONSEPT OF COUNTERMEASURE FOR LIQUEFACTION TO AIRPORT FACILITIES

Recently, the concept of performance-based design considering level1 and level2 ground motions has been treated in various kinds of structures from the lessons of past earthquake damages in Japan. Therefore, the concept of countermeasure for liquefaction has been also different from usual earthquake resistant designs. In case of an airport, its function requests smoothness and baring capacity of runway pavement to restart its operation after a destructive near-field earthquake, as presented in Table 1 (Service Center of Port Engineering, 2008). The new design concept includes two key points: a) necessity to pay attention to not only the stability of structures but also the function of facilities and b) necessity to develop earthquake resistance while keeping present serviceability of existing facilities.

In terms of the concept of a), the limit value of runway pavement against level2 ground motion is defined as presented in Table 1. On the other hand, regarding the concept of b), a countermeasure for liquefaction to runway pavement is required that its construction in an airport site must be carried out in several hours of midnight to avoid interrupting the time of airport operation. Under this limited condition, compaction grouting and permeable grouting are generally employed in runway pavement and apron in Japan. Compaction grouting is the method that increases density of liquefiable ground, while permeable grouting is the method that changes characteristics of liquefiable soil structure by replacing groundwater with colloidal silica, as shown in Fig.1a and b respectively (Coastal Development Institute of Technology, 2007; Yamazaki et al., 2002).

Table 1 Limit Values for Present Serviceability of Runway Pavement against Level2 Groung Motion

Facility	Deformation of ground by liquefaction	Bearing capacity of ground
Center part in Runway Pavement （Range of 2/3 of width of R/W）	The maximum inclination of R/W shows 1.5% in the direction of crossing and 1.0% in the direction of running though.	Required bearing capacity for present serviceability of R/W is decided by time history of dissipation of excess pore water pressure.
Edge part of Runway Pavement (Range of 1/3 of width of R/W）	The maximum inclination of R/W shows the value within 1/2 in the crossing direction and 1.5% in the running direction. (Partial inclination of 50% in edge part of R/W is permissible.)	



(a) Compaction grouting *(b) Permeable grouting*
Fig.1 Examples of Improving in Airport Site

OUTLINE OF THE EXPERIMENT

A full-scale liquefaction experiment was carried out on October 27, 2007, at the Ishikari Bay New Port (Ishikariwan-shinko) in Hokkaido Island, Japan. Figure 2 shows the location of the Ishikari Bay New Port in Otaru city. The Port is very close to Sapporo city within 15km or 40 minutes by car from the city central.

The primary objective of the experiment carried at the Port was to assess the performance of full-scale airport facilities during liquefaction. This experiment focused on the behavior of full-scale airport facilities during liquefaction state that was induced by controlled blast technique. In this experiment, various kinds of observation and measurement before and after liquefaction were executed.

Fig.2 Location of Experimental Site

Photo 1 Aerial Photograph of Experimental Site

Experimental Site Conditions

Photo 1 shows the location of the site in the coastal line of Ishikari Bay. The site consists of gentle slope of sea bed surface reclaimed with dredged sands and sand dunes. The typical soil profile is shown in Fig.3 and its characteristics are the following:
1) Soil strata: The top layer called Fs consisted of dredged sand 5-6m thick from Ishikari Bay and very loose with 8 or smaller of *N*-value obtained by standard penetration tests (SPT). Sandy deposit bellow Fs was the layer As1 1-2.5m thick, which was equivalent to the coastal line before reclamation. The layers As1 and As2 bellow As1 showed the ranges of 3 to 12 and 8 to 20 of *N*-values respectively. The groundwater level was about GL.-2.5m located in the Fs layer.
2) Physical properties: The physical properties are shown in

Table 2. The ranges of fine content, Fc, of each layer were 7 to 38% for Fs, 5 to 22% for As1 and 8 to 32% for As2 respectively. The grain size distributions of the soil are shown in Fig.4. Based on these distributions, most layers were poorly distributed, that is, they were within the “possibility of liquefaction zone” defined by the technical standards for port and harbour facilities (Japan Port and Harbour Association, 2007).

3) Liquefaction strength: Liquefaction strength curve obtained from undrained cyclic tri-axial shear tests of undisturbed samples is shown in Fig.5. Liquefaction strength, R_L, of Fs and As1, which is defined as $\sigma_d/2\sigma_c$’ at 5% of double amplitude, DA, and 20 cycles of cycle number, N_c, ranges between 0.189 and 0.244.

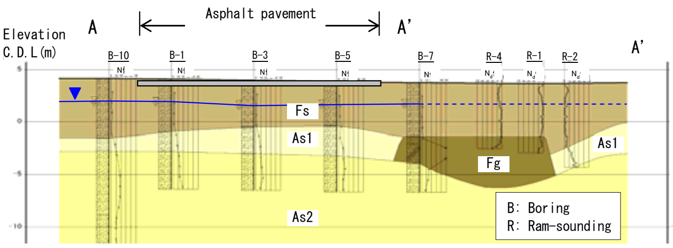


Fig.3 Soil Profile of the Runway Area

Table 2 Characteristics of soil layers

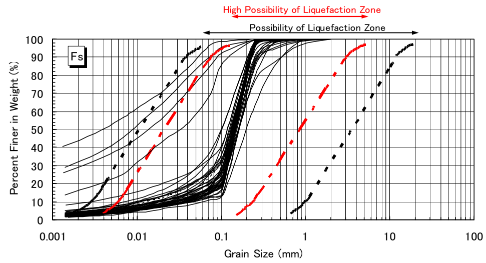
Soil Layer	N-Value (Average)	Liquefaction Strength, R_L	Fine Content Fc(%)	Wet Density ρ_t(g/cm^3)	Plastisity Index, Ip
Fs	1～8（2.6）	0.189～0.244	7～38	1.827～1.867	*NP*
As1	3～12（7.9）	-	5～22	-	*NP*
As2	8～20（14.1）	0.204～0.222	8～32	1.796～1.819	*NP*

Layout of the Experimental Site

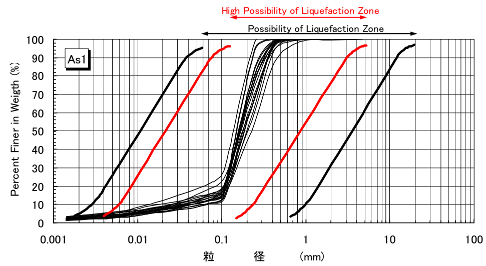
As shown in Fig.6, lots of airport facilities were configured in the experimental site. The main airport facility was runway pavement 50m long and 60m wide. The runway pavement area were divided into four parts such as 1) the left top area improved by a compaction grouting method, 2) the left bottom area improved by a multi-permeation grouting method, 3) the middle area improved by a permeable grouting method, and 4) the right area that was unimproved. All improvement methods were installed with cost reduction design specification in contrast with usual design procedures.

Controlled Blast Sequence

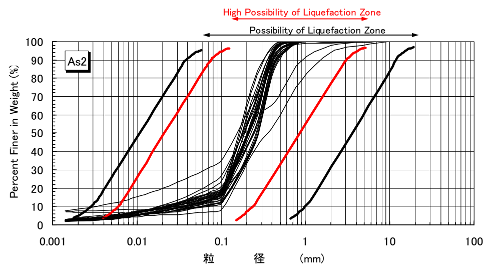
Figure 7 and Photo 2 shows the vertical boreholes charged with 4kg explosive at GL-9m and 2kg explosive at GL-4.5m. The under-path boreholes of pavement made by a horizontal directional drilling machine were also charged with 4kg explosive at GL-9m and 2kg explosive at GL-4.5m. Each charged explosive was ignited domino-toppling manner with the interval time of 200ms. It took about 139 seconds to complete the blasting of 538 explosives.



(a) Fs Layer



(b) As1 Layer



(c) As2 Layer

Fig. 4 Grain Size Distributions of Soils Tested before Blasting

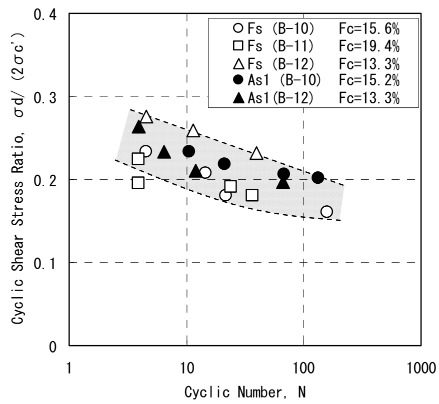


Fig. 5 Liquefaction Characteristics before Blasting

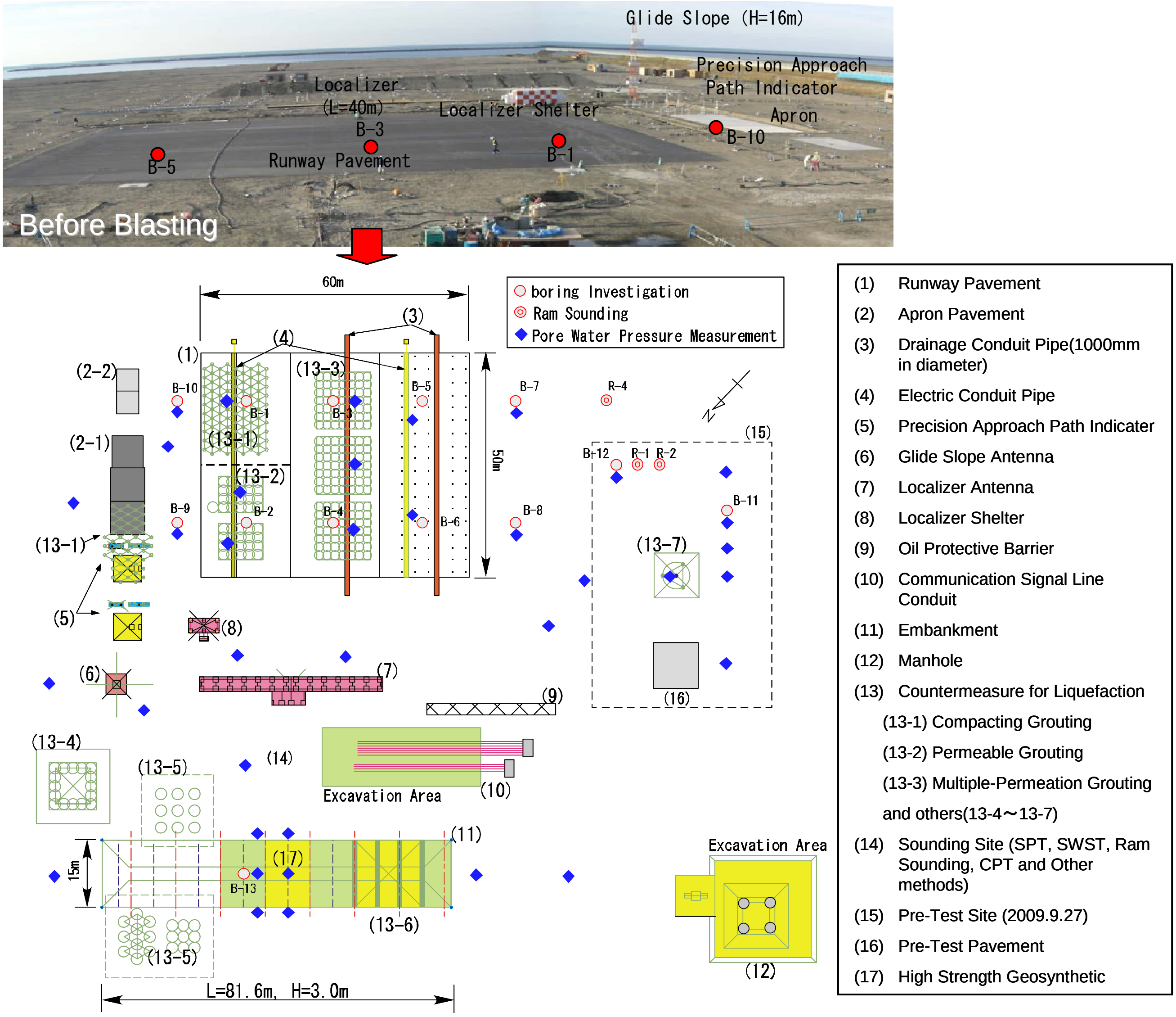
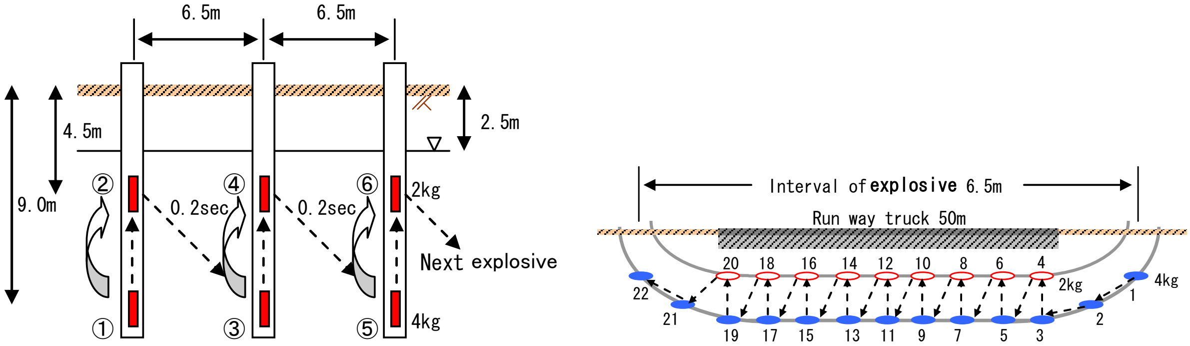


Fig.6 Plan View of Experimental Site



(a) Vertical boreholes *(b) Horizontal directional drilling boreholes*

Fig.7 Outline of Controlled Blast Sequence

Photo 2 Charge Process of Explosive

The controlled blasting is able to reproduce excess pore water pressure build-up to a level of effective overburden pressure, σ_v', which is a similar phenomenon of an earthquake inducing liquefaction. However, it is hard to reproduce acceleration and cyclic loading conditions.

SPECIFICATION OF COMPACTION GROUTING

Feature of Compaction Grouting

Compaction grouting, called as CPG in Japan, is a method of pumping grout with very low mobility of less than 5cm slump into the ground under pressure to form grouts without vibration and shocks. As the grouts increase their volume, the surrounding ground is compacted to increase its density.

As shown in Fig.8, the compaction grouting method, free from the problems of use of large construction machines, vibration and noise, and adverse effect on the neighboring areas, is exceptionally effective for protecting existing structures and underground installations.

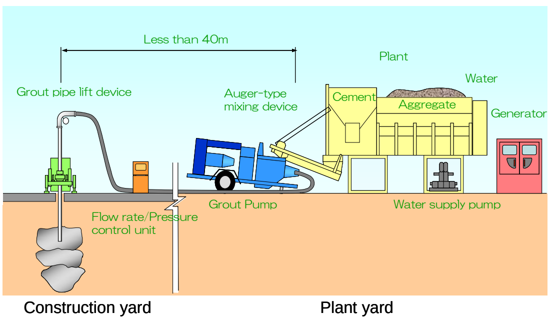


Fig.8 General Use of Compaction Grouting

According to the recent result of the shaking table test on CPG using 1/10 model ground, the coefficient of earth pressure, *K*, defined as σ_h'/σ_v' increased after grouting and liquefaction was effectively suppressed, compared with the untreated soils of the theoretically similar relative density (Harada et al., 2007). Actually, it was reported that the effectiveness of CPG was estimated by *K*-value obtained from self boring pressure-meter test (SBP), which is a kind of Lateral loading test, at Tokyo International Airport site (Zen et al., 2002). It is confirmed that the *K*-value improved by CPG is retained until occurrence of liquefaction.

New Design Concept Tried in This Experiment

The current design practice assumes that improved ground becomes dense due to the void change that volume equals to the volume of grouting material injected to the ground. General design of CPG is explained as following:

1) The range of improvement rate, αs, is 8 to 15%. The grouting points form regular triangle layout and its intervals of grouting point are usually 1.2 to 1.7m.

2) The extra improved area to avoid propagating excess pore water pressure build-up around an untreated liquefied layer into improved area is required. The extra improved area is the range of 30 degrees gradient from the bottom of soil improvement from the edge of improved area.

In this experiment, a new design concept in contrast to a usual design condition was determined as following:

1) αs and the intervals of grouting point were 5% and 2.0m, respectively.

2) CPG was constructed only under runway pavement and the extra improved area was omitted, as shown in Fig.9.

These employed design conditions aimed at the cost reduction and shortening of the term of improvement works to satisfy the limited condition in airport construction.

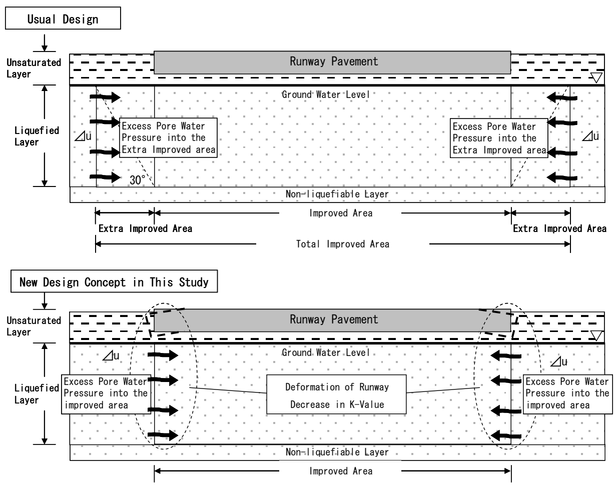


Fig. 9 Comparison of Design Concepts

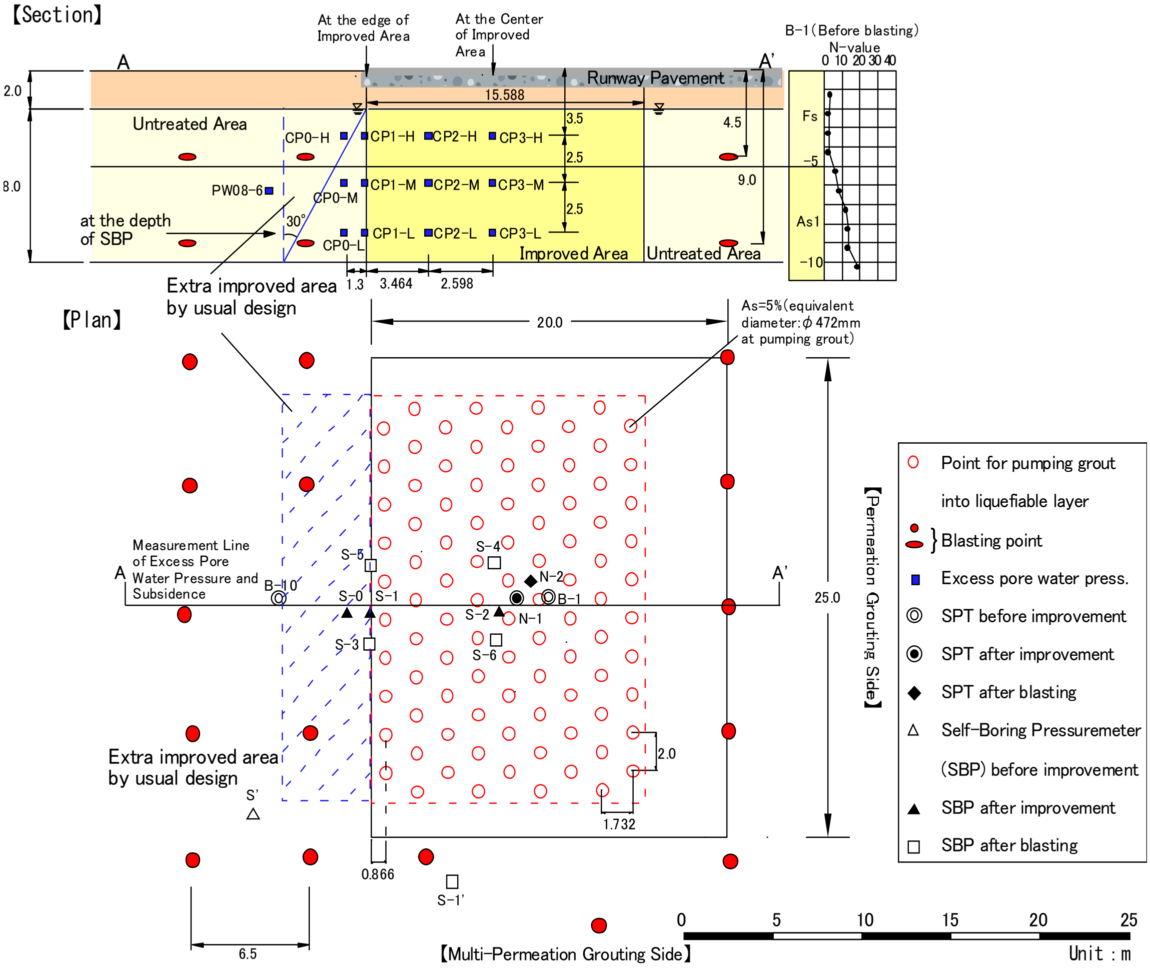


Fig.10 Experimental Layout of CPG Area

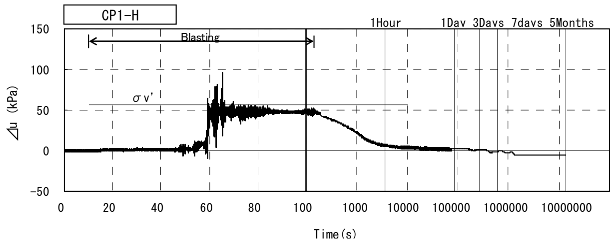
Figure 10 shows the experiment layout of CPG area. The measurements of differential leveling on the surface of asphalt, and pore water pressure in CPG area and its surroundings were planned to be carried out on A-A' line. Also *K*-values at the depth of GL-7.0m before and after improvement, and after blasting were investigated by SBP at the center and the edge of improved area.

EXPERIMENTAL RESULT

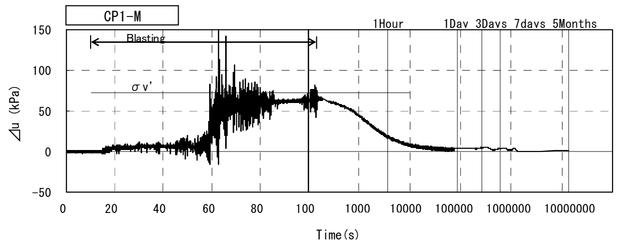
Photo 3 shows the bird's-eye view of the experimental site after blasting. The sand boil and water due to liquefaction induced by the blast are observed. Most extreme sand boils were observed around untreated area.

Excess pore water pressure induced by blasting was measured in untreated and improved area. Figure 11 shows the time histories of excess pore water pressure, Δu, obtained at the points CP3-H, L and M at the center of improved area, and the points CP1-H, L and M at the edge of improved area during blasting and dissipation processes of Δu. The arrangement of the graphs of Fig.11 is the same as the arrangement of the measurement points of Δu shown in Fig.10. To compare Δu measured in improved area, the time history of Δu at PW08-6 in untreated area is shown in Fig.12.

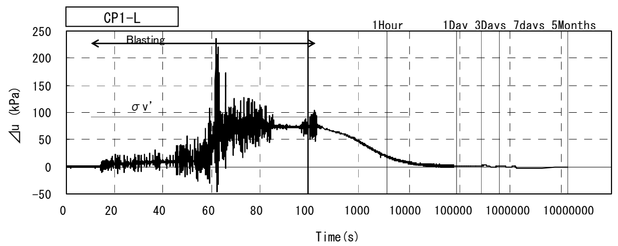
Photo.3 Bird's-Eye View of Experimental Site



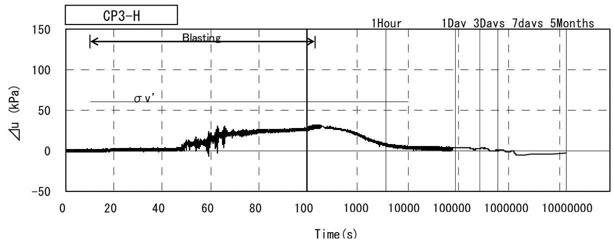
(a) Δu atGL-3.5m of the edge of improved area



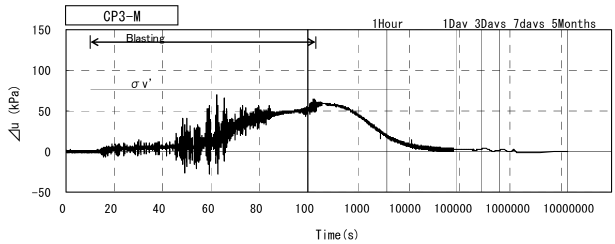
(b) Δu atGL-6.0m of the edge of improved area



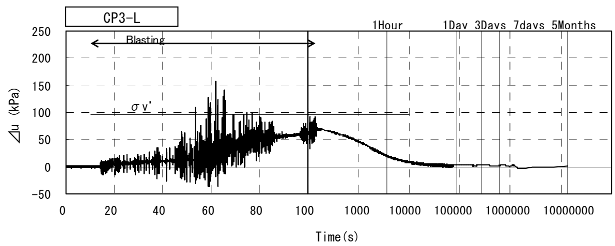
(c) Δu atGL-8.5m of the edge of improved area



(d) Δu atGL-3.5m of the Center of improved area



(e) Δu atGL-3.5m of the Center of improved area



(f) Δu atGL-3.5m of the Center of improved area

Fig. 11 Time Histories of Measured Excess Pore Water Pressure

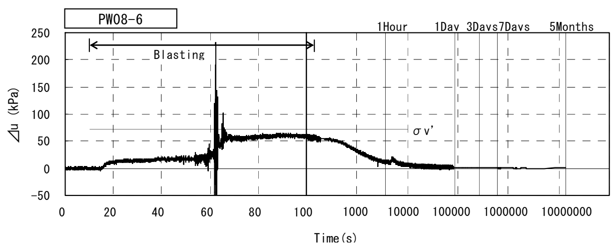


Fig.12 Δu at GL.-6.0m in Untreated Area (PW08-6)

At all measurement points, *Δu* built up by blasting near improved area and completely dissipated in a day after blasting. However, tendency between build-up processes of *Δu* at the center of improved area and in the untreated area seems different. *Δu* in untreated area built up and reached to σ_v' rapidly during blasting, while *Δu* at the center of improved area built up gradually and reached to liquefaction perfectly.

Figure13 shows the subsidence contour chart based on the result of differential leveling after one hour from the end of blasting. The subsidence in untreated area, which is on the right side of runway pavement, is remarkable compared with that of each improved area.

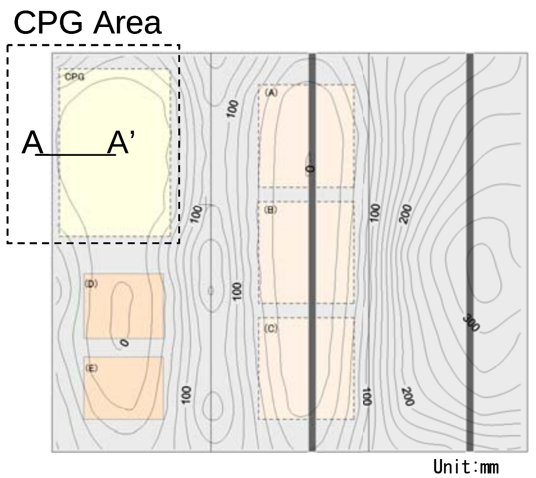


Fig.13 Subsidence contour chart at one hour after blasting

Distribution of subsidence after 1 hour to 7 days from the end of blasting on A-A' line is shown in Fig.14. It can be seen that most of subsidence was caused within one hour after blasting. In untreated area, the behavior of subsidence was similar to the behavior of Δu, which value showed more than 0.5 within 1hour after blasting. Though the subsidence of about 40mm at the edge of runway pavement was observed, it is thought that the design without extra improved area in this study did not influence on the function of runway pavement because its inclination in the distance of 5m from the edge shows 0.8% and satisfied the requirement shown in Table 1.

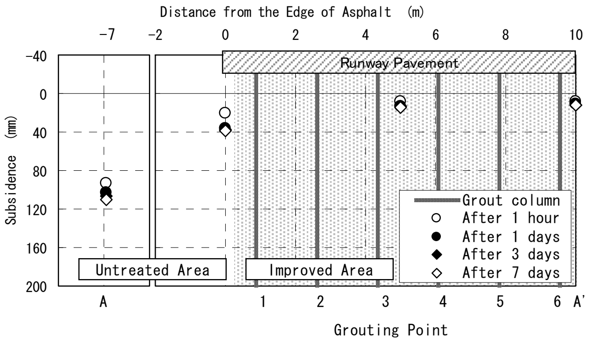


Fig.14 Distribution of Subsidence on A-A' Line

ESTIMATION OF IMPROVING EFFECT

Figure15 and 16 show the distribution of *N*-values and Increments of *N*-values after grouting and blasting at the center of improved area at B-1, N-1 and N-2 points shown in Fig.10. The investigation at N-2 point was carried out after the complete dissipation of Δu. It is understood that an increase in *N*-value after grouting is remarkable in the deep range equivalent to As1 layer and As2 layer though *N*-value at GL.-4.7m increased slightly. The post liquefaction *N*-value could be kept higher than *N*-value before grouting though that decreases in As1 layer and As2 layer as compared with *N*-value after grouting.

To estimate the continuation of effectiveness of CPG at the center and edge of improved area after grouting and blasting, SBP was carried out at GL-8.5m to investigate *K*-values as well as *N*-values. The investigation points in improved area and untreated area are previously shown in Fig.12. The list of measured *K*-value data is shown in Table 4 and Fig.17 shows *K*-values after grouting and blasting as compared with *K*-values obtained from untreated area. *K*-values increased from 0.36 to 0.92 at the center of improved area and 1.00 at the edge of improved area by grouting. On the other hand, post-liquefaction *K*-value at the center of improved area showed 0.82 and approximately kept the same level as the *K*-value after grouting, while the edge one showed 0.63 and decrease of about 40% with liquefaction around the improved area. However, this fact suggests that post-liquefaction *K*-values at the edge of improved area could be kept effective enough to maintain the function of runway pavement as compared with *K*-value before grouting.

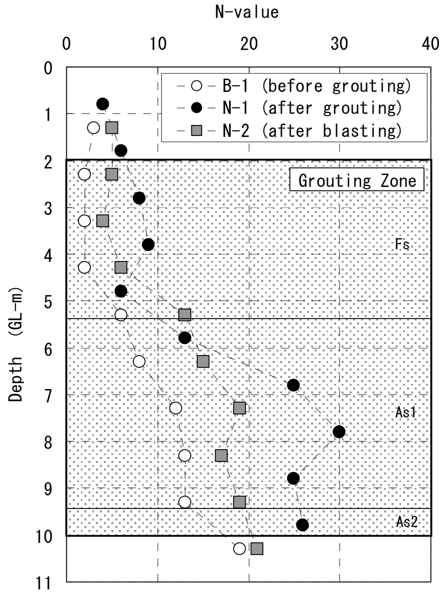


Fig.15 Distribution of N-values after grouting and blasting at the center of improved area

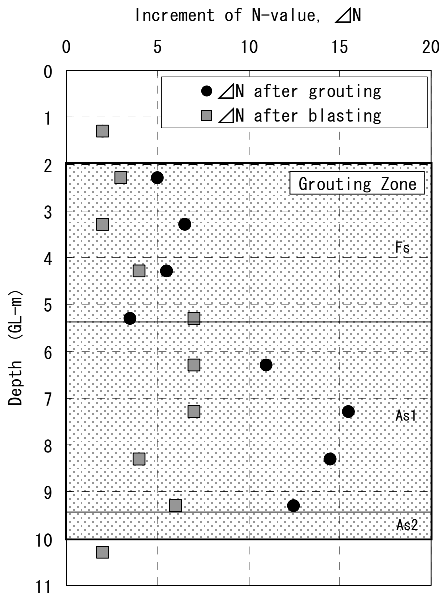


Fig.16 Increment of N-values after grouting and blasting at the center of improved area

Figure18 shows the relationship between αs and *K*-value based on the results investigated in Tokyo International

Airport site (Zen et al., 2002). As shown in this figure, *K*-values show the tendency to increase with αs and the results in this study, including *K*-value after grouting and blasting, are corresponding to the range of those obtained from the past investigation. Therefore, it is thought that a decrease in *K*-value in improved area after liquefaction is not so significant in the judgment of the effectiveness of improvements.

Table 4 Investigation Results of *K*-values

No.	Condition of Investigation		Depth GL.-(m)	Soil Type	Coeficient of Earthpressure K_0
S'	Before Blasting	Untreated Area	8.5	Fine Sand	0.36
S-0		Outside of Improved Area	8.5	Fine Sand	0.96
S-1		Edge of Improved Area	8.5	Fine Sand	1.00
S-2		Center of Improved Area	8.5	Fine Sand	0.92
S'-1	After Blasting	Untreated Area	8.5	Fine Sand	0.39
S-3		Edge of Improved Area	8.7	Fine Sand	0.63
S-4		Center of Improved Area	8.7	Fine Sand	0.82

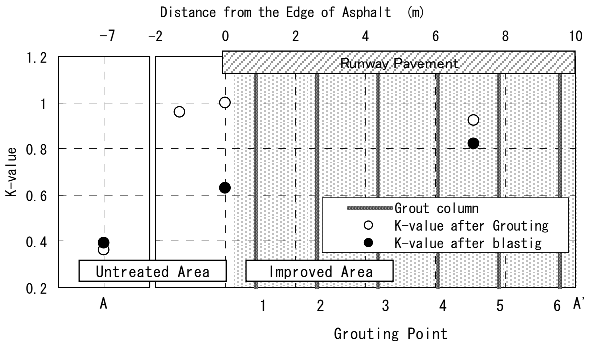


Fig.17 Comparison between K-values after grouting and Post-liquefaction K-values on A-A' Line

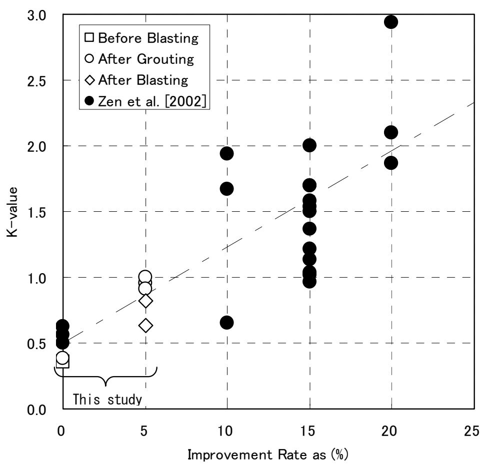


Fig. 18 Relationship between Improvement Rate and K-values

CONCLUSIONS

The coefficient of earth pressure in ground improved by compaction grouting was investigated in a full-scale field liquefaction experiment. This study concludes that compaction grouting with cost-reduction design keeps its effectiveness after liquefaction. In addition, although post-liquefaction *K*-value decreased, it can be estimated that runway pavement will work after liquefaction because post-liquefaction *K*-values at the center and the edge of improved area were higher than those before grouting.

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