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FULL-SCALE FIELD TEST ON LIQUEFACTION-INDUCED DAMAGE OF RUNWAY PAVEMENT BY CONTROLLED BLAST TECHNIQUE

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ABSTRACT

The sustainable function of an airport during and after a great earthquake is very important from the viewpoints of emergency medical services and special operations for transporting relief supplies to disaster areas. However, it is difficult to estimate the damage to airport facilities by a great earthquake. In order to assess the performance of airport facilities that were constructed in waterfront liquefiable soft ground, a full-scale field experiment was carried out. In the experiment site, liquefaction was artificially caused by controlled blast technique. This paper describes the observations of the experiment concerning the liquefaction behavior of runway pavement and the results of damages of the pavement on improved and unimproved grounds, and discusses the effectiveness of countermeasures for runway pavement against liquefaction.

Introduction

The role of airports during and after a great earthquake is functionally important in terms of 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 functions of airports, a full-scale liquefaction experiment was conducted using controlled blast technique at the Ishikari Bay New Port in Hokkaido Island, Japan in October 2007. The objectives of the experiment are to assess the performance of airport facilities subjected to liquefaction and to investigate their damage mechanism. In this experimental site, various full-scale structures such as a runway, an apron, an air traffic control glide slope antenna and countermeasures against liquefaction were constructed, and about 400 measurement devices were installed. In addition, several types of new non-destructive health monitoring devices, including a laser profiler, a ground radar and a falling-weight deflect meter were also adopted. The experiment also focused on evaluation of the effect of countermeasures against liquefaction. In the site, two types of countermeasures, the Compaction Grouting Method (CPG) and the Chemical Grouting Method (CGM), were introduced, based on a new design concept to counter liquefaction due to a large earthquake.

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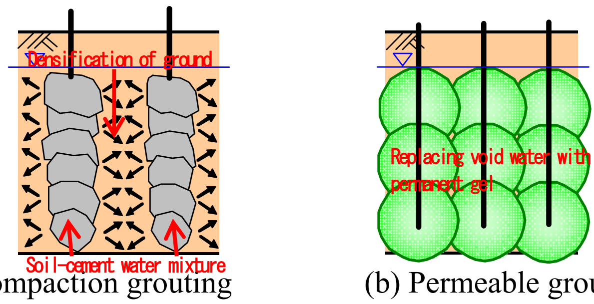
This paper introduces the results of the experiment including the liquefaction behavior of runway pavement, and the results of damage analysis applying to the pavement on improved and unimproved grounds. The effectiveness of countermeasures for runway pavement against liquefaction is also discussed.

New Design Concept of Countermeasures for Runway Ground against Liquefaction

Recently, the concept of performance-based design regarding strong motions induced by a great earthquake has been applied to airport facilities from the lessons of past earthquake damages in Japan. Therefore, the concept of countermeasure against liquefaction is also different from that of current earthquake-resistant designs based on liquefaction strength. In case of airport runways, smoothness and baring capacity of runway pavement must be in acceptable range 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) functions of airport runways need to be estimated as well as their stability, and (b) existing runways must be reinforced in order to develop seismic performance required in the new design concept. In terms of the concept of (a), the function of the runways can be assessed referring the limit values given in Table 1. Regarding the concept of (b), a construction of liquefaction countermeasures for existing runway pavements is usually performed in several hours of midnight to avoid interrupting the time of airport operation, and then, compact construction methods which do not require removing large area of runway pavement are appropriate. Figure 1 shows two of available methods generally employed in Japan for sites with such a restricted construction condition; i.e. Compaction Grouting Method (CPG) that increases density of liquefiable ground and Chemical Grouting Method (CGM) that changes characteristics of liquefiable-soil structure. More details of these methods are described later.

Table 1. Required performance of runway pavement (Service Center of Port Engineering 2008)

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

Figure 1. Examples of Improving in Airport Site

Outline of the Experiment

This experiment mainly 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 investigations, observations and measurements before and after liquefaction were conducted.

Experimental Layout and Site Conditions

Figure 2 and Photo 1 show the location of the site in the coastal line of Ishikari Bay and the experimental site before blasting respectively. The site was located in the coastal area of Ishikari Bay, consisting of a gentle slope of sea bed surface reclaimed by dredged sands and sand dunes.

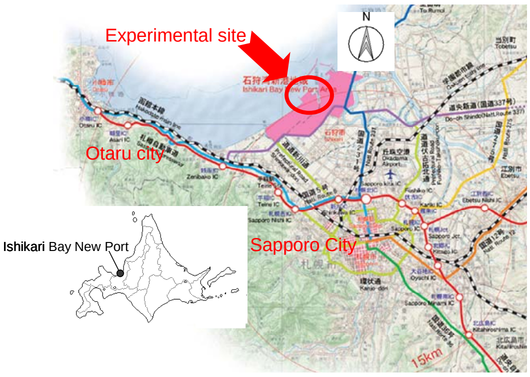


Figure 2. Location of experimental site

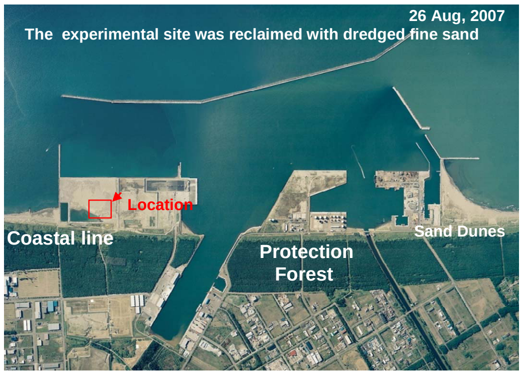


Photo 1. Aerial Photograph of experimental site

As shown in Fig. 3,the runway pavement, defined as R/W below, which was the main airport facility, was 50m long and 60m wide. The R/W area was divided into three parts including (a) the left top area improved by the CPG, (b) the left bottom and the middle area improved by the CGM and (c) the right area that was unimproved. All improvement methods except the Case_D of the CGM in Fig. 3 were installed with cost reduction design specification by reducing improved area or low improvement rate in contrast with current design procedures.

The typical soil profile is also 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 and very loose with 8 or smaller of *N*-value obtained by standard penetration tests, SPT. The layer As1 underlying Fs was 1-2.5m thick, which was equivalent to the coastal line before reclamation. The layers As1 and As2 showed the ranges of 3-12 and 8-20 of *N*-values respectively. The groundwater level was about GL-2.5m located in the Fs layer.
2) Physical properties: The grain size distributions of the soil are shown in Fig.4. The ranges of fine content, *F*c, of each layer were 7-38% for the layer Fs, 5-22% for the layer As1 and 8-32% for the layer As2 respectively. Based on these distributions, most layers ware poorly graded and they were within the “possibility of liquefaction zone” defined by the technical standards for port and harbour facilities (Japan Port and Harbour Association 2007). Incidentally, liquefaction strength, R_L, defined by $\sigma_d/2\sigma_c$’ at 5% of double amplitude and 20 cycles, were ranged between 0.189 and 0.244 in the layers Fs and As1. According to these site conditions, it was judged that the soil layers would be liquefied by controlled blast technique.

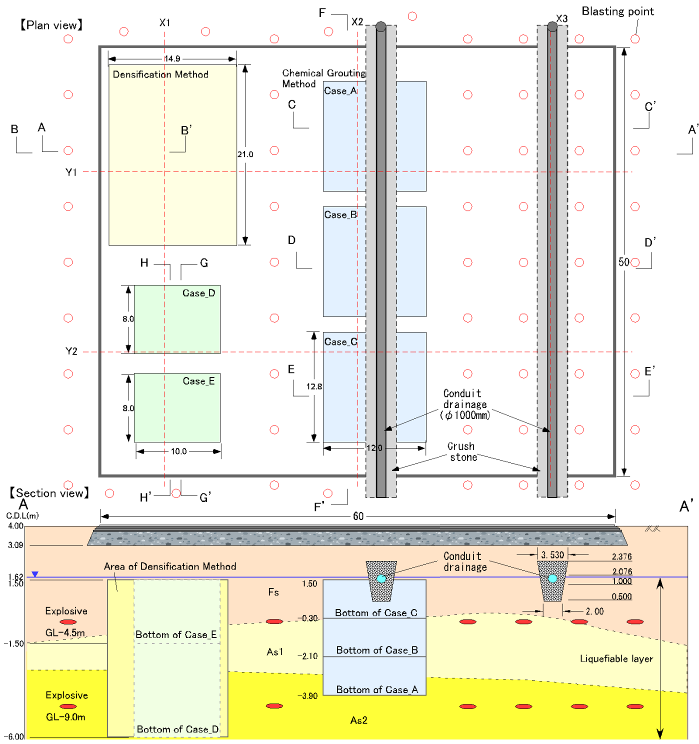
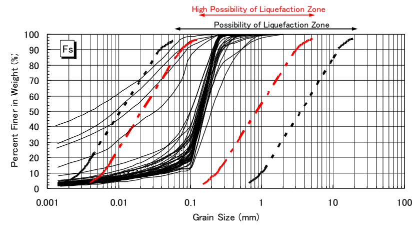
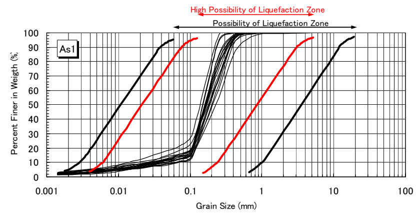


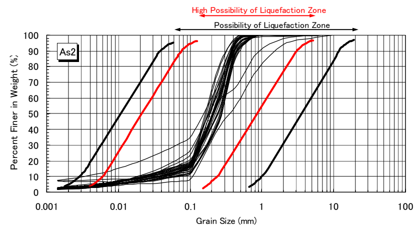
Figure 3. Plan and section view of runway pavement area



(a) Fs layer



(b) As1 layer



(c) As2 layer

Figure 4. Grain size distribution

Countermeasures against liquefaction

In this study, two types of countermeasures against liquefaction were examined. Details of their features are described below.

Compaction Grouting Method (CPG)

Compaction Grouting Method, called CPG in Japan, is one of ground improvement methods against liquefaction. Grouting soil-cement-water mixture, densification of ground can be achieved as shown in Fig. 1a. The specification of CPG is currently designed assuming that improved ground becomes denser due to the void change, and that the amount of the volume change is equal to the volume of grouting material injected to the ground (Coastal Development Institute of Technology 2007).

The plan and section view of CPG-improved area in this experiment are shown in Fig. 5. In this experiment, in contrast to a current design condition in Table 2, a new design concept was determined in order to reduce the construction cost and shorten the term of improvement works for satisfying the limited condition of airport construction. Figure 6 shows the most important viewpoint that the extra improved areas in order to avoid propagating excess pore water pressure build-up in a surrounding unimproved liquefied area into improved area. The extra improved area is usually the range of 30 degrees gradient from the bottom of soil improvement from the edge of improved area in the current design, but it was omitted in the cost reduction design and in this experiment.

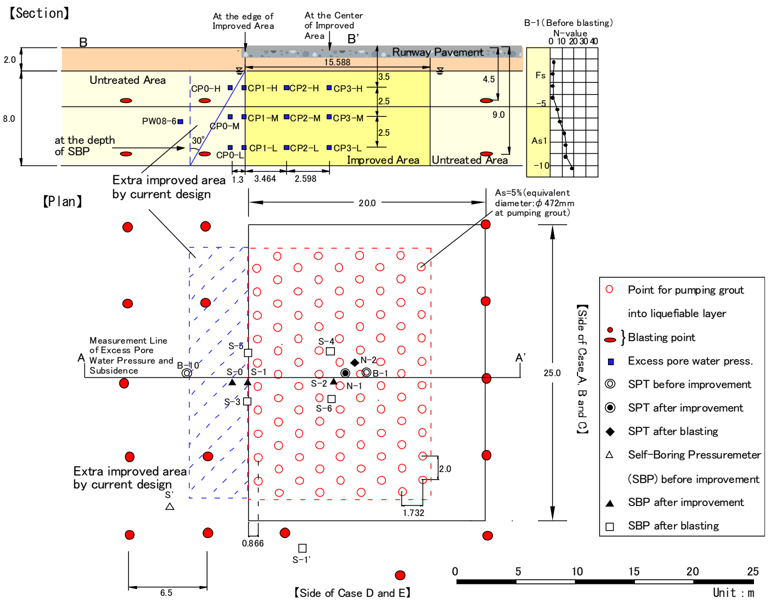


Figure 5. Plan and section view of compaction grouting area

Table 2 Comparison between current design and condition of this experiment

Condition	Current design	New design concept (Cost reduction design)
Arrangement of grouting points	triangle or square	same as current design
Improvement rate (%)	8～15	5
Intervals of grouting points (m)	1.2～1.7	2
Extra-improved area	required	possible to reduce

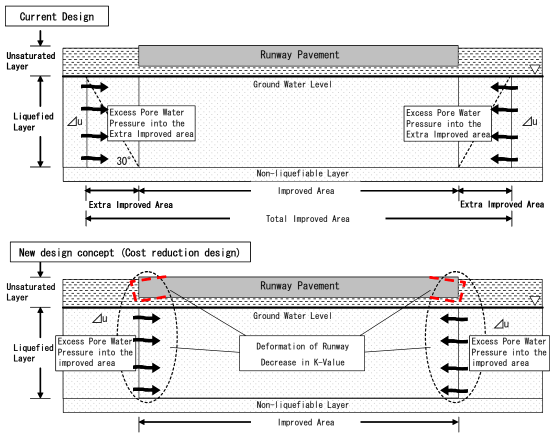


Figure 6. Comparison between current and new design concept

Chemical Grouting Method (CGM)

Chemical Grouting Method, called CGM herein, is a ground improvement method which can increase liquefaction strength of soil replacing void water with permanent gel to avoid excess pore water pressure build-up to effective overburden pressure. CGM has the following advantages: (1) CGM are appropriate for construction in narrow site because their device is very compact, (2) controlling carefully, CGM can be performed near existing facilities and structures without damages. Though there are several types of CGMs, Permeable Grouting Method shown in Fig.1b (Yamazaki et al. 2002) was used in the experimental site. The plan view of CGM-improved area in this experiment is shown in Fig. 7. Figure 8 is a conceptual drawing how partial improvement prevents differential settlement. A list of the improvement cases in this experiment is shown in Table 3. In this paper, the ratio of improved thickness to whole liquefiable layer thickness is defined as partial improvement ratio, and the effect of partial improvement was estimated based on the results of the settlement induced by liquefaction and excess pore water pressures in the unimproved layer beneath the improvement.

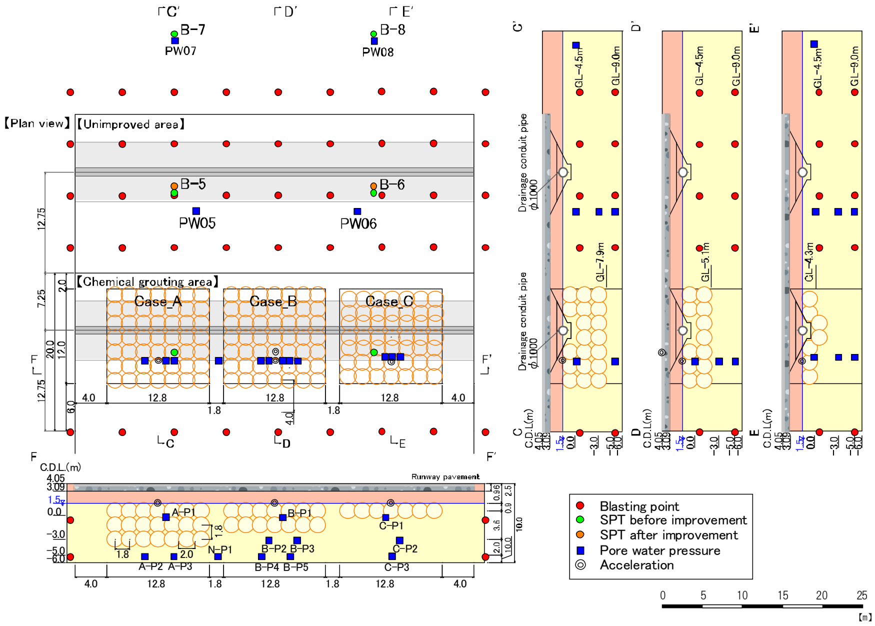
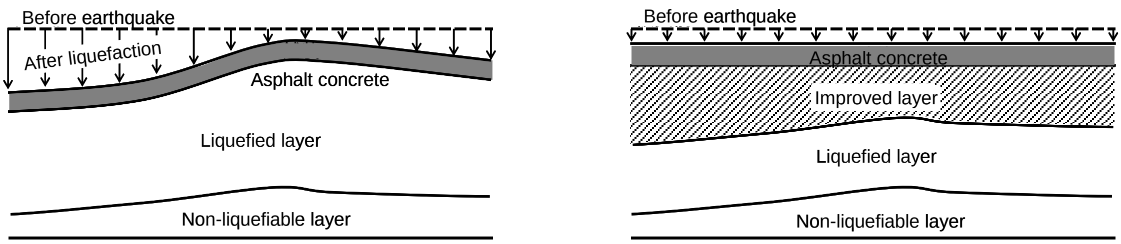


Figure 7. Plan and section view of Chemical grouting area (Case_A, B and C)



(a) Residual deformation occurred by liquefaction (b) Effect of partial improvement

Figure 8. Concept of partial improvement by chemical grouting

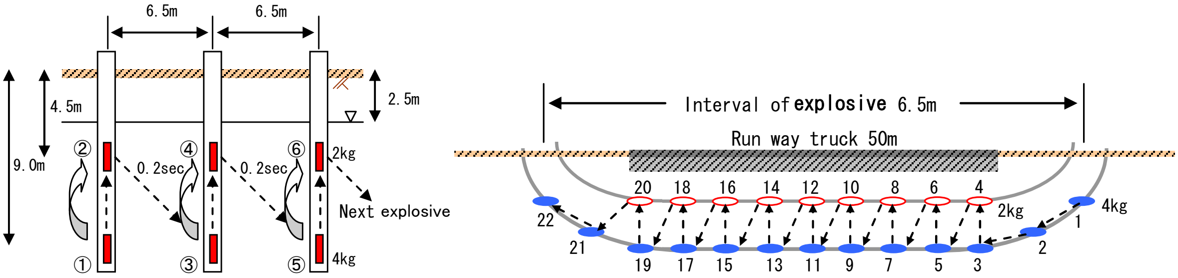
Table 3. Specifications of partial improvements by chemical grouting

Case	Improvement rate (%)	Depth at the bottom of improvement GL-(m)	Partial improvement rate (%)
A	70	2.5～7.9	72
B	70	2.5～6.1	48
C	70	2.5～4.3	24
D	100	2.5～10.0	100
E	100	2.5～5.5	40

Controlled Blast Sequence

Figure 9a 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

(Fig. 9b). Each charged explosive was ignited in the domino-toppling manner at intervals of 200ms. It took about 139 seconds to complete the blasting of 538 explosives all over the blasting area. Arrangement of explosives around R/W is also shown in Fig. 3.



(a) Vertical borehole type (b) under-path borehole type

Figure 9. Outline of controlled blast sequence

The typical mechanism of liquefaction during an earthquake is as follows:

a) Shear waves propagate and shear deformation occurs in the sandy soil layer.
b) Negative dilation causes.
c) Excess pore water pressure builds up.

In contrast, the mechanism of controlled-blast induced liquefaction is as follows:

d) Blast causes destruction of the sand particle structure.
e) Re-location of sand particles or densification takes place between explosives.
f) Excess pore water pressure builds up.

The controlled blast technique can reproduce excess pore water pressure build-up to the level of effective overburden pressure, σ_v', which is a similar phenomenon of liquefaction. However, it is difficult to simulate seismic acceleration and cyclic loading conditions.

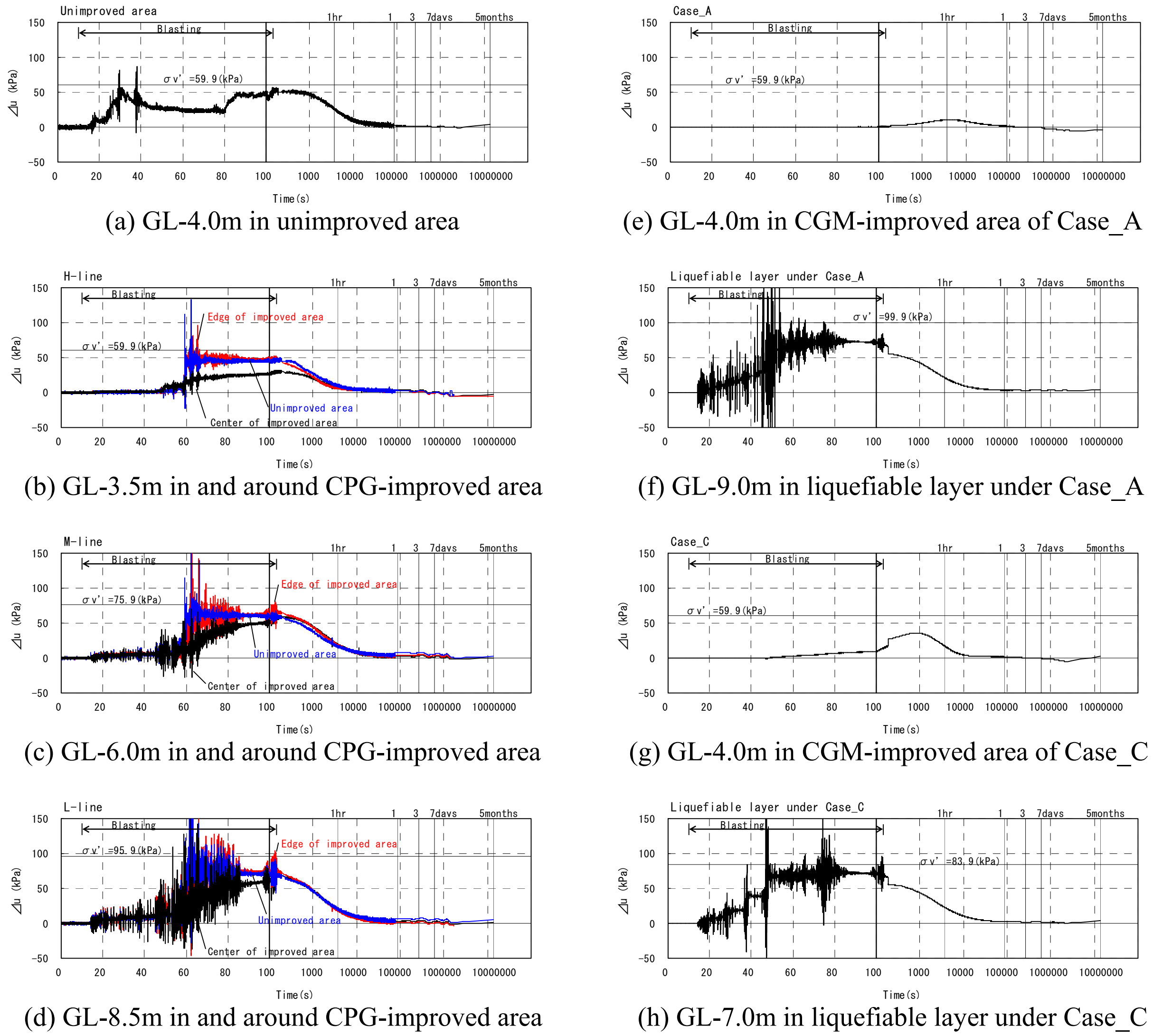
Experimental Result

Photo 2 shows the bird's-eye view of the experimental site after blasting. Liquefaction-induced sand boil and water boiling out through borehole for explosives were observed. The most extreme sand boil was found around the unimproved area.

Photo 2. Bird's eye view of the experimental site after blasting

Liquefaction Behavior in the Experimental Site

Figure 10 shows time histories of excess pore water pressure, Δu, in the unimproved, the CPG- and the CGM-improved areas. As shown in Fig. 10a, Δu in the unimproved area reached overburden pressure during blasting. Δu decreased to about 50% of σ_v' at one hour after complement of blasting, and was dissipated perfectly within a day.



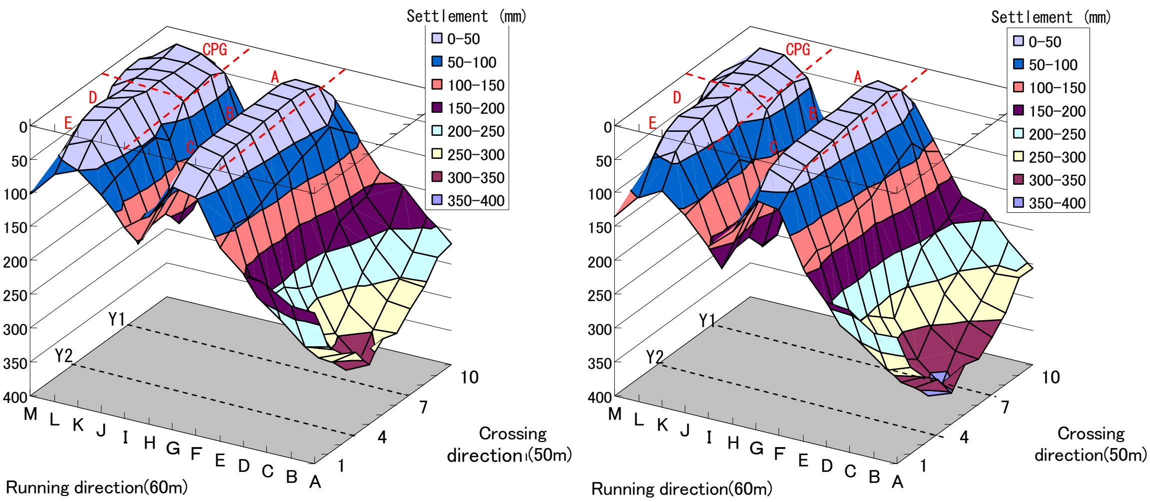
(a) GL-4.0m in unimproved area
(e) GL-4.0m in CGM-improved area of Case_A
(b) GL-3.5m in and around CPG-improved area
(f) GL-9.0m in liquefiable layer under Case_A
(c) GL-6.0m in and around CPG-improved area
(g) GL-4.0m in CGM-improved area of Case_C
(d) GL-8.5m in and around CPG-improved area
(h) GL-7.0m in liquefiable layer under Case_C

Figure 10. Time histories of measured excess pore water pressure

Figures 10b through 10d show time histories of Δu measured in the CPG-improved area. Δu at the depth of 3.5m, 6.0m and 8.5m in the CPG area, shown in Fig. 5, built up to effective overburden stress by blasting, and was completely dissipated in a day after blasting as well as that in the unimproved area. However, the tendency of build-up processes of Δu at the center of the CPG-improved area was different from one in the unimproved area. Δu near the unimproved area built up and reached to σ_v' rapidly during blasting, while Δu at the center of the improved area built up gradually and reached to 50-70% of σ_v'.

Time histories of Δu measured in the CGM-improved area and liquefiable layer under the partial improvement are shown in Figs. 10e to 10h, respectively. According to these graphs, Δu measured in a lump of the CGM-improved soil in Figs.10e and 10g shows the same tendency that Δu indicated the range of about 0.2-0.6 of σ_v' after blasting in spite of different partial improvement rate between 70% and 25%. On the other hand, Δu measured in the liquefiable layer under partial improvement in Figs. 10f and 10h shows about 80% of σ_v', and reached to the state like liquefaction.

Figure 11 shows the subsidence contour charts based on the results of differential leveling after one hour and 7 days from the end of blasting. The subsidence in the unimproved area, which is on the right side in Fig. 11, was more than 35cm, and significantly different from that in both the improved area by CPG and CGM. This fact indicates that the cost reduction design, including CPG-improvement without an extra improved area, and partial improvement by CGM, was effective to prevent significant settlement of airport runways.



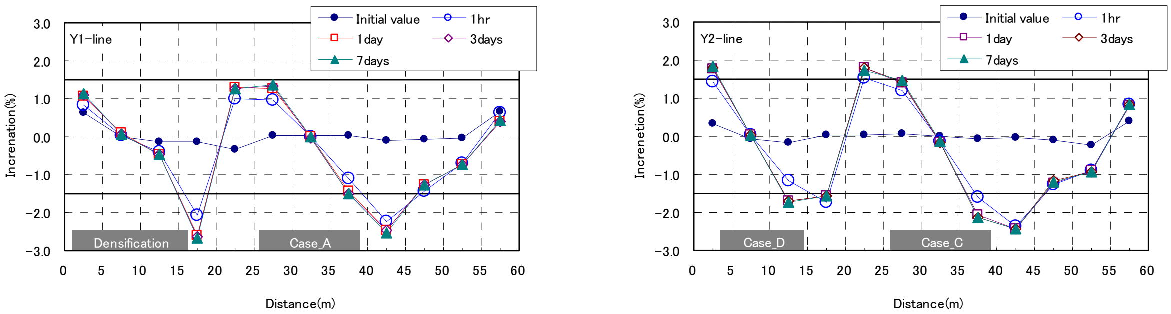
(a) 1 hour after blasting (b) 7 days after blasting

Figure 11. Distribution of subsidence in runway pavement

Estimation of Residual Deformation on Runway Pavement

Present serviceability of R/W is usually estimated by maximum inclination in the direction of crossing and the direction of running through as shown in Table 1. It is certain that the most serious problem of R/W is differential settlement as well as local deformation and cracks with large deformation of ground occurred by liquefaction. According to distribution of subsidence shown in Fig. 11, it can be seen that most of subsidence was caused within one hour after blasting because a remarkable difference can not be confirmed by comparing these two graphs.

In order to estimate the R/W function in this experiment, distribution of R/W inclination along Y1 and Y2 line in the distance of 5m from the edge was shown in Fig. 12 based on measured subsidence. Though the inclination nearby the edge of R/W and the unimproved area between the improvements shows more than 1.5% remarkably, it does not significantly influence to the R/W function on the improved ground because its inclination shows within limit value of present serviceability, and almost satisfies the requirement shown in Table 1.



(a) Y1-line (b) Y2-line

Figure 12. Inclinations of runway pavement compared with limit value for present serviceability

Conclusions

This study was performed in order to estimate residual deformation of runway pavements. Also, two types of countermeasures against liquefaction, Compaction Grouting Method (CPG) and Chemical Grouting Method (CGM) with cost reduction designs were assessed. A lot of valuable knowledge was obtained through this full-scale experiment.

As a result, noticeable residual deformations on the improved areas did not occur after liquefaction. However, the settlement of 35cm and the inclination over 1.5% of the runway pavement were observed on the unimproved area, meaning the loss of airport function when encountering a large earthquake that tends to cause liquefaction. On the other side, it can be confirmed that tried countermeasures against liquefaction with cost reduction deigns were effective. However, we have to find the relationship among aircraft loads, runway support systems during liquefaction and excess pore water pressure dissipation states after a large earthquake and more research efforts are needed.

Acknowledgments

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