Shaking model tests on mitigation of liquefaction-induced ground flow by new configuration of embedded columns

Essais sur table vibrante pour une attenuation de l'écoulement des sols du a la liquefaction par une nouvelle configuration de colonnes enterrees

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ABSTRACT: Traditionally, countermeasures against seismic liquefaction aimed to prevent significant development of excess pore water pressure. Although this aim was achieved by a variety of measures, the limitation has been understood as well in the recent times. The limitation is typically found in lifelines and transportation lines (road embankment) together with river levees for which financial requirement is more strict and also residual deformation is allowed to occur to a certain extent. In this regard, the present study addresses installation of stable cement-mixed soil columns in liquefaction-prone subsoil so that ground deformation may be constrained during earthquakes and lateral flow of liquefied sand may be reduced. For its validation, two types of shaking-table model tests have been conducted in 1-G environments.

RÉSUMÉ : Traditionnellement, les mesures de lutte contre la liquéfaction sismique visaient à empêcher un développement significatif de la pression interstitielle de l'eau en excès. Bien que cet objectif ait été atteint par une variété de mesures, leurs limites ont également été perçues récemment. Les limitations concernent typiquement les infrastructures critiques et les lignes de transport (remblai routier) ainsi que les digues fluviales pour lesquelles les besoins financiers sont plus stricts et les déformations résiduelles sont tolérées dans une certaine mesure. À cet égard, la présente étude porte sur l'installation de colonnes de mélanges sol-ciment stables dans des sous-sols susceptibles de liquéfaction afin que les déformations du sol puissent être limitées durant les séismes et que l'écoulement latéral du sable liquéfié puisse être réduit. Pour la validation de l'étude, deux types d'essais sur table vibrante ont été effectués dans des environnements 1-G.

KEYWORDS: liquefaction, deformation, mitigation, model test.

1 INTRODUCTION

Recent developments of deep soil mixing for ground improvement have produced different variations in geometry of solidified soil. For example, uniform mixing of the entire soil with grouting agent is the original idea and, because sand grains are thereby bonded with each other, it is very reliable. However, the construction cost for this is the highest among all other options. Accordingly, improvements of grid type, wall type, and columnar type have been attempted.

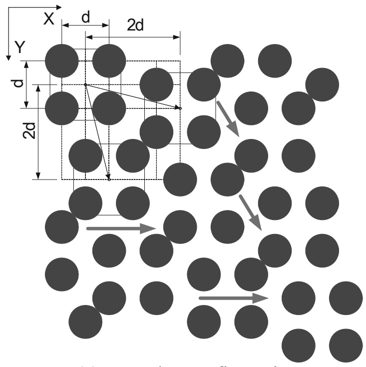
For the aim of liquefaction mitigation, either grid or wall types have been used in practice. It is expected therein that the rigidity of the improved grid or wall constrains the shear deformation of sand inside the grid (or between parallel walls), and hence pore pressure development during strong shaking is significantly reduced (Suzuki et al., 1989). The effect of the grid-type mitigation was validated during the 1995 Kobe earthquake in which subsoil liquefaction was prevented and an overlying water-front building was completely protected from seismic damage (Suzuki et al., 1995).

In contrast to grid- or wall-type grouting, the columnar type was considered less effective than others in liquefaction mitigation in spite of its lower installation cost (Koga et al. 1986). However, more recent shaking model tests by Yasuda et al. (2003) and Tanaka et al. (2003) indicate that columns are able to constrain shear deformation of soil and hence mitigate the onset of liquefaction. Moreover, Yamamoto et al. (2006) carried out numerical analyses to indicate that columnar improvement develops similar prevention of pore pressure development as grid type does if the improvement ratio exceeds 35%, although the grid type is better in performance if the improvement ratio is less. In this regard, the present study conducted 1-G shaking model tests on liquefaction mitigation achieved by assembling four columns into one (CDM-Land4 method; CDM Association, 2002) so that the rigidity is significantly increased. Furthermore, attention was paid to the geometry of columns, supposing that an irregular installation of columns may be able to achieve better mitigation than conventional regular (square or triangular) configuration of columns. (Towhata et al. (2010) and Takahashi et al. (2010))

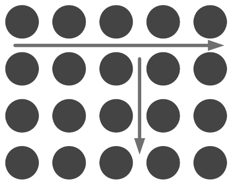
The following text addresses first more details of irregular installation of columns, followed by shaking model tests to examine the effects of irregular configuration of columns in 1-G environment. The first series of shaking model tests were conducted in a rigid soil box in which a sloping and liquefiable sandy layer was placed at the top. The second series of tests were performed on sheet-pile quay wall models with liquefiable backfill sand.

2 IRREGULAR CONFIGURATION OF UNDERGROUND COLUMNS

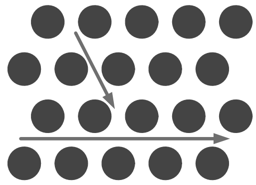
Figure 1 illustrates three kinds of column configurations that are addressed in this paper. The irregular configuration in Fig. 1(a) shows that a 2*2 square grid with a spacing of "d" is shifted either by 2d or d/2 distance in X and Y directions, respectively. This configuration does not allow free passing of liquefied subsoil through column spaces in contrast with the square and triangular configurations (Figs.1(b) and (c)) where a straight flow passage is available; see red arrows in these figures. This lack of free and open space in the irregular configuration is further expected to restrain cyclic shear straining during shaking and reduce the probability of liquefaction.



(a) Irregular configuration



(b) Square configuration



(c) Triangular configuration

Figure 1. Geometry of column installation

3 METHOD OF SHAKING TABLE TESTS

The shaking model tests were conducted at the University of Tokyo by using a soil container that measured 2,650 mm in length, 390 mm in width, and 600 mm in depth. The scale of modeling was supposed to be 1/20.

3.1 Tests in a rigid box on sloping ground model

Figure 2 shows the schematic view of a sloping ground model. The model ground consisted of a base unliquefiable layer of 150 mm in thickness, and a sloping and liquefiable sandy layer at the top. The surface gradient was set equal to 10%. The entire ground was made of Toyoura sand (G_s=2.684, e_{min}=0.605, e_{max}=0.974, D_{50}=0.21mm) and was submerged in water.

The base layer was prepared by air pluviation of dry Toyoura sand, followed by compaction to the relative density of 75 %. This dry layer was then saturated by one-hour slow percolation of water. The upper liquefiable layer was prepared by water pluviation to attain 40% relative density. This low relative density was employed to cancel the effects of low effective stress level on dilatancy and liquefaction resistance of sand (Towhata, 2008). The height of fall was maintained constant, irrespective of the ongoing height of sand surface, while the water depth was also controlled to be 20 cm that was expected to remove pore air from the falling sand and help achieve high degree of saturation. During shaking, the water surface was set at the same elevation as the top of the slope (Fig. 2).

The embedded columns were modeled by acryl pipes that measured 26 mm in the outer diameter and 20 mm in the inner diameter, respectively, implying the equivalent diameter of 520 mm in the prototype. The bottoms of the pipes were fixed, as stated above, by screwing into a PVC (polyvinyl chloride) plate of 20 mm thickness. On the other hand, the top of the pipes were connected with another PVC plate of 5 mm thickness by two O-rings. Thus, rotation was possible to occur at the top.

During shaking, acceleration, excess pore water pressure, and lateral displacement of liquefied soil were recorded (Fig. 2). The lateral deformation was recorded by photographs and motion pictures of colored sand in the cross section (Fig. 3) and on the surface. As Fig. 2 illustrates, the central part of the slope model had vertical columns and the time history of lateral soil displacement was recorded at both upstream and downstream sides of the columns by using embedded inclinometers. Moreover, some of the columns (acryl pipes) were equipped

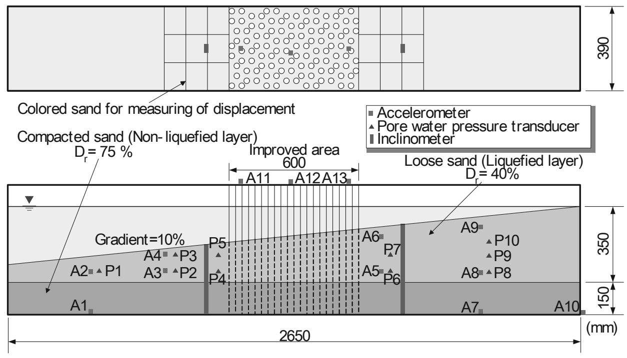


Figure 2. Schematic view of sloping ground model

Figure 3. Schematic view of sloping ground model

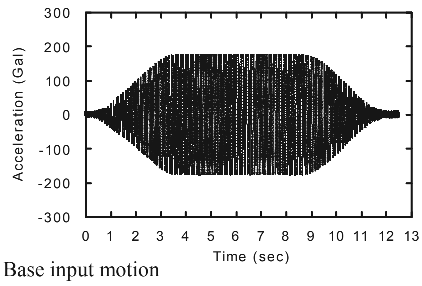


Figure 4. Base input motion

Table 1. Details of sloping ground model

Test case	CASE1	CASE2	CASE3	CASE4	CASE5
Configuration of columns	-	Irregular		Regularly triangular	
Improvement ratio (%)	0	25	35	25	35
Maximum acceleration (Gal)			200		

with strain gages to record bending strain therein.

Horizontal shaking took place in the longitudinal direction of the slope with 10Hz and 200 Gal at the maximum, while the duration time was 6 seconds (Fig.4). More details of tests are summarized in Table 1 where configuration of columns and improvement ratio are varied from tests to tests.

Table 2 summarizes the law of similitude concerning the present study.

3.2 Tests in a rigid box on quay wall model

The second series of shaking tests were performed on sheet-pile quay wall models with liquefiable backfill sand. The columnar soil improvement was intended to reduce the distortion of the quay wall and the backfill sand.

As illustrated in Fig. 5, a limited part of the backfill was improved by columns. The sheet-pile quay wall was supported by an anchorage plate (Fig.6) that was further supported by the improved part of backfill sand.

The model ground consists of 100 mm compacted sand at the bottom (relative density = 85%) and the upper 400 mm of liquefiable sand (relative density = 40%).

The columns were modeled by PVC pipes whose outer diameter was 26 mm, thickness 3 mm, and length 500 mm. Pipes were filled with sand for equilibrium of weight and buoyancy. Both top and bottom of the pipes were fixed to avoid lateral deformation by connecting them to a 5 mm-thick plastic boards. By assuming the scale ratio of 1/20, the prototype diameter of the columns is 520 mm. The range of soil improvement (shown by orange color in Fig. 5) is 600 mm in length and 390 mm in width with the improvement ratio of 25%.

Table 2. Similitude law for 1-G model tests

Items	Model/Prototype	Prototype	Model
Scale(N=20)	1/N	1	0.05
Pile diameter	1/N	520mm	26mm
Frequency	$N^{-0.75}$	1.06Hz	10Hz
Relative density		60%	40%
Acceleration	1	200Gal	200Gal

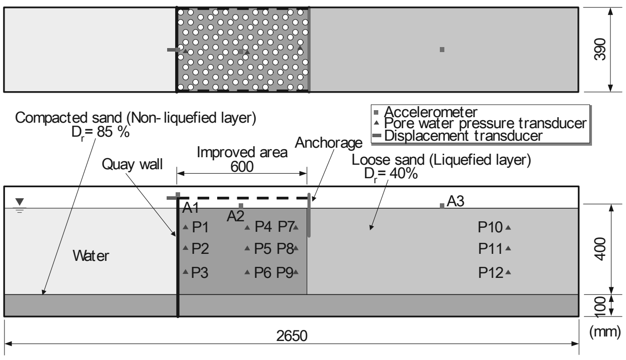


Figure 5. Schematic view of quay wall model

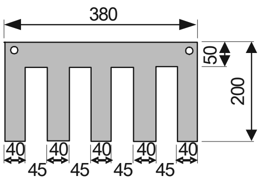


Figure 6. Vertical shape of anchorage plate (connected with quay wall at top left and right by rods; see white circles there)

Table 3. Details of quay wall model

Test case	CASE6	CASE7	CASE8
Configuration of columns	-	Irregular	Regularly triangular
Improvement ratio (%)	0	25	25
Maximum acceleration (Gal)		200, 500	

The sheet-pile quay wall was modeled by an aluminum plate having 3 mm thickness and 510 mm height with its width equal to that of the soil box. The bottom of the wall was placed in a socket at the bottom and had no mechanical fixing. This wall was supported by an anchorage.

Accelerometers and pore pressure transducers were embedded in the model ground, while acceleration and displacement at the top of the sheet pile wall were recorded as well (Fig. 5). Further, motion picture was taken of the lateral cross section of the ground in which lines of colored sand was installed for easy interpretation.

The base shaking is identical with the one in Fig. 4 with the maximum acceleration of either 200 or 500 Gal. For details of 3 tests run, see Table 3. Both regular (triangular) and irregular configurations of columns were tested with the improvement ratio of 25 %.

4 RESULTS OF SLOPING GROUND MODEL

Deformations of testes models are illustrated in Figs. 7-9. While deformation of vertical columns of colored sand in contact with the side window is shown by black lines, the inclinometer data in the central part of the mode is indicated by black dots. First, the deformation of ground without columnar improvement

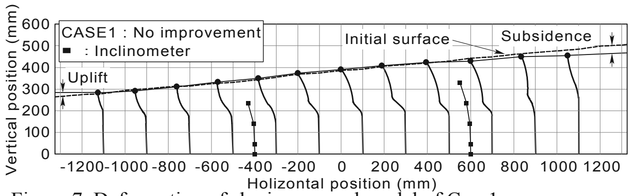


Figure 7. Deformation of sloping ground model of Case1

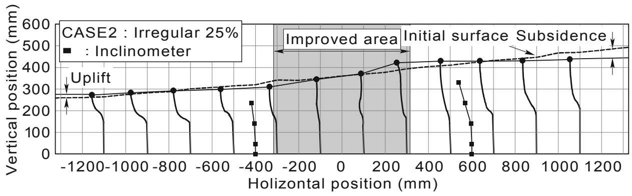


Figure 8. Deformation of sloping ground model of Case2

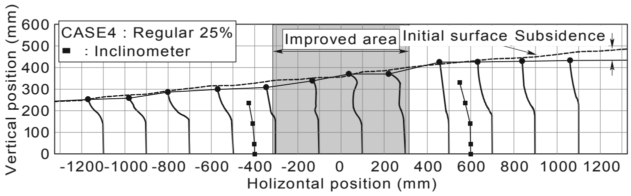


Figure 9. Deformation of sloping ground model of Case4

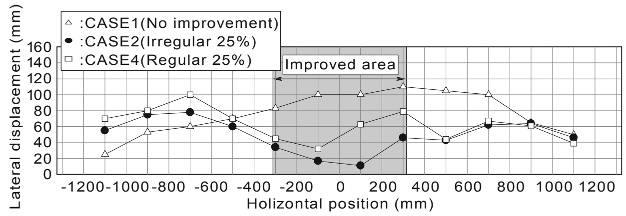


Figure 10. Lateral displacement at the surface at the end of shaking (improvement ratio 25%)

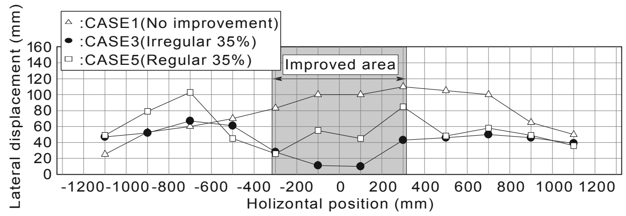


Figure 11. Lateral displacement at the surface at the end of shaking (improvement ratio 35%)

(Case 1) shows that the most part of deformation occurred in the upper liquefied layer, while the deformation in the unliquefiable base layer is insignificant. The higher upstream side subsided and the lower downstream side uplifted, consequently reducing the slope gradient. Second, the Case 2 test with the irregular column configuration developed uplift in the upstream proximity of the improved area (+200 to +500 mm) probably because the columns reduced and dammed the lateral flow of liquefied sand. This finding is a good contrast with the deformation of Case 4 (triangular configuration) where the lateral flow of liquefied sand was easier. Similar difference was observed in the cases of 35% improvement ratio as well.

Figures 10 and 11 compare the lateral displacement at the top (at the surface) of colored sand as indicated by black dots (inclinometers) in Figs. 7 to 9. It is first found that cases with any kind of column configuration (Case 2 to 5) was of less extent of displacement than in Case 1 without columns. Thus, columns mitigate the lateral displacement of liquefied subsoil. Further, the cases with the irregular configuration (Cases 2 and 3) showed the displacement even smaller than in the cases of triangular configuration (Cases 4 and 5). Thus the mitigative effect of the irregular configuration of columns is superior to that of the regular triangular configuration.

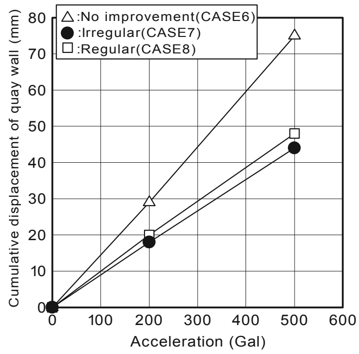


Figure 12. Accumulated residual displacement at the top of the quay wall with or without columnar soil improvement

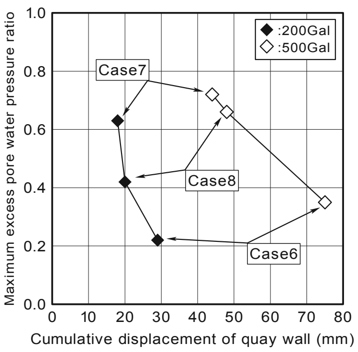


Figure 13. Relationship between maximum excess pore water pressure ratio and cumulative displacement of quay wall

5 RESULTS OF QUAY WALL MODEL

The interest in this series of tests is focused on the residual deformation of a model quay wall with or without columnar soil improvement in the backfill. This idea is certainly in line with the recent movement towards performance-based seismic design principle.

Figure 12 illustrates the lateral displacement that was accumulated at the top of the quay wall with repeated shaking. Obviously the regular configuration of columns reduced the displacement and the irregular configuration achieved even better mitigation. Note that this mitigation is partially because the anchorage was stabilized by the columnar soil improvement. Another reason is the stabilization of the backfill that exerts directly earth pressure on the quay wall.

Figure 13 shows the relationships between the ratio of developed excess pore water pressure (% of the initial effective vertical stress) and cumulative displacement of quay wall. The excess pore water pressure ratio is measured (p6 in Fig.2) at 350mm depth in the region at the central position of the improved area.

It is important that lower pore water pressure is caused by such two mechanisms as 1) greater resistance of sand against liquefaction, and 2) positive dilatancy after large shear deformation of sand. In the present case, Case 6 without improvement shows the lowest pore pressure ratio because of the large shear deformation and positive dilatancy of sand behind the wall. Conversely, the higher pore pressure in Cases 7 and 8 is the consequence of reduced soil deformation. It is noteworthy, therefore, that mitigative effects should not simply be evaluated by the magnitude of excess pore water pressure. A similar finding was reported in a former study of the authors' group (Mizutani et al., 1998).

6 CONCLUSIONS

The mitigation of lateral displacement of liquefied ground was studied by running 1-G model tests with a special interest in the irregular configuration of vertical embedded columns. The following conclusions were drawn from the present study.

(1) The sloping ground model tests showed heaving on the upstream side of the improved area. This deformation was induced by the damming effect of the irregular configuration of columns.
(2) The triangular configuration did not exhibit such a damming-up effect because its mitigative effect is less significant and passing-through of liquefied sand was easier.
(3) Consequently, the lateral displacement was reduced more efficiently by the irregular configuration than by the regular triangular configuration of columns.
(4) The irregular installation of columns can better reduce the residual deformation of a quay wall than a conventional regular one because the irregular pattern does not allow free motion of sand so much as the regular pattern.
(5) A special care should be taken of pore pressure development when the mitigative effect is evaluated. This is because pore water pressure is possibly made lower by both large deformation and positive dilatancy of sand.

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