1170/5/A

LIQUEFACTION OF EMBANKMENTS ON SANDY SOILS AND THE OPTIMUM COUNTERMEASURE AGAINST THE LIQUEFACTION

Young-Ho PARK[1]

Sung-Ryul KIM[2]

Sung-Hwan KIM[3]

Myoung-Mo KIM[4]

SUMMARY

Shaking table tests for 12 different cases are performed to investigate the behavior of embankments resting on liquefiable sand grounds in a model box, and to compare the effectiveness of several methods of countermeasures in mitigating earthquake induced liquefaction phenomena. The countermeasures employed here are installation of sheet-pile walls, gravel drains, sand compaction piles, and pipe piles. The dimensions of the transparent model box are 194^{cm} long, 44^{cm} wide, and 60^{cm} deep. Saturated sand grounds are prepared for the tests to have a relative density of about 30% by water sedimentation method, and 15cm high embankments are constructed on the model grounds. 12 pore water pressure transducers, 4 accelerometers, and 2 LVDTs are installed throughout the model set-ups to monitor the response of the ground as well as the embankments.

The shaking table, whose size is 2m x 2m, is excited from 0.1 g up to 0.4 g. Sand compaction piles are found to be the most effective in mitigating the liquefaction damages among the methods applied in the model tests, even if all the methods are proven to be effective tool in preventing the occurrence of liquefaction at normal range of excitations. Further, the optimum location of each countermeasure is also determined in this study.

[1] *Hyundai Institute of Construction Technology, 102-4 Mabuk-li, Kuseong-myun, Yongin-si, Kyounggi-do, Korea E-mail:yhpark@hyundai.hdec.co.kr*

[2] *Department of Civil Engineering, Seoul National University, San 56-1, Shinlim-dong, Kwanak-gu, Seoul, Korea E-mail:yawnful@chollian.net*

[3] *Korea Highway Cooperation, 293-1 Kumto-dong, Sujeong-gu, Sungnam-si, Kyounggi-do, Korea E-mail:shkim@freeway.co.kr*

[4] *Department of Civil Engineering, Seoul National University, San 56-1, Shinlim-dong, Kwanak-gu, Seoul, Korea E-mail:geotech@gong.snu.ac.kr*

1. INTRODUCTION

Embankment on saturated loose sandy ground is easily liquefied and damaged during earthquake shaking. There are many case histories to illustrate the damaging effects of liquefaction on river dykes, road embankments, and earth dams such as the 1964 Niigata Earthquake, the 1989 Loma Prieta Earthquake, the 1993 Kushiro-oki Earthquake and the 1995 Hyogoken-Nanbu Earthquake. Liquefaction of foundation soils resulted in large deformations or failures of embankments supported on silty and sandy deposits. To prevent the damages caused by liquefaction, several types of remediation methods, whose principles are based on compaction, drainage, solidification and inclusions, have been applied to the liquefiable soils [Marcuson et al. 1996 ; PHRI. 1997 ; Adalier et al. 1998]. Recently more experiences are gained to show that treated ground performs satisfactorily during earthquakes[Mitchell et al. 1995 ; 1998]. However, liquefaction is a very complex phenomenon and many things remain to be solved particularly for designing ground improvement methods to minimize the damaging effects of liquefaction.

1-g shaking table tests and centrifuge model tests for embankments resting on liquefiable ground have been conducted to understand the fundamental mechanism of liquefaction and the effectiveness of remediation techniques by many researchers. Koga et al[1990] carried out 1-g shaking table tests of earth embankments founded on saturated sandy ground to investigate the effects of input acceleration characteristics such as wave form, frequency contents and duration time on the response and damages to the model. It revealed that input earthquake motion characteristics strongly affected the soil response and damage extent of an embankment. In sinusoidal excitation, the settlement of embankment was more severe when the frequency was low.

Zheng et al[1995] investigated the mechanism of embankment failure due to liquefaction in centrifuge model tests. The sand ground in free field liquefied, while the sand ground beneath and near embankment did not liquefy. And also it was observed that the sand ground beneath embankment extended horizontally toward the free field due to the weight of embankment which pulled the embankment horizontally and led to the cracking of embankment. Therefore it was concluded that one of effective ways to mitigate the damages of embankment due to liquefaction is to constrain the horizontal extending of the sand ground under embankment. Adalier et all[1998] presented the test results and analyses of liquefaction countermeasures including densification, solidification, a gravel berm, and sheet pile(with tie rods) enclosure. Effects of each countermeasure on excess pore pressure developments and embankment deformations were compared and analyzed. The countermeasures reduced embankment settlement by a maximum of about 50% in case of sheet-pile enclosure. It was added that retrofit techniques to constrain lateral spreading of the underlying liquefied soil are deemed to be particularly effective in preserving overall embankment integrity as indicated in Zheng et al[1995].

1-g shaking table tests for 12 different cases including the cases with no countermeasure, are executed on model embankments resting on liquefiable loose sands to verify the effectiveness of liquefaction countermeasures which are frequently applied in practice and to substantiate the proper arrangements of these countermeasures. Liquefaction countermeasure techniques reviewed in this study are sheet-pile walls, pipe piles, gravel drains, and sand compaction piles.

2. SHAKING TABLE TEST PROGRAM

A test box is made of plexi-glass, whose size is $194^{cm}\times 44^{cm}\times 60^{cm}$(height). The average size of model sand(D_{50}) is 0.55^{mm} and the coefficient of uniformity is 1.6. The maximum and minimum dry density of the sand are 1.6ton/m^3 and 1.4ton/m^3 respectively. Liquefiable loose sand ground is formed under water by pouring the sand through 1mm sieve to reduce the dropping velocity and to maintain it in the loosest state as possible. The relative density of the liquefiable sand ground(D_r) is about 30%. A dense sand layer is prepared by shaking the liquefiable loose sand with intensity of 0.3g for 20 minutes which comes to the relative density of about 90%. Embankment is built of angular gravel of 1-2 cm size with 1:1.5 slopes. A piece of geotextile is placed below the embankment to separate it from the foundation ground but not to influence the behavior of the embankment.

In total, 12 different types of tests are planned as shown in Fig. 1. The first 3 tests are run with no liquefaction countermeasure applied, but with the different thickness of liquefiable loose sand ground ; 40^{cm}, 30^{cm} and 15^{cm}. All the other model tests, where liquefaction countermeasures are applied, are executed on the sand grounds with the loose layer of 30^{cm} underlain by the dense layer of 10^{cm}. The next 4 tests are executed with sheet-piling as a liquefaction countermeasure. In the 3 tests among the 4, the thickness of the sheet piles is varied as 1.2^{mm}, 0.5^{mm}, and 0.3^{mm}, and all are embedded into the dense soil layer. In the 4^{th} test, the sheet pile of 0.3mm thickness is placed above the dense layer.

In case 8, pipe piles are installed up to the bottom of the model box by driving. In cases 9 and 10, gravel drains are constructed at the toe and beneath the embankment respectively. Gravel drains used here have a cross section as shown in Fig. 1(case 9 and case 10). Gavel drains are constructed by drilling the ground with a casing to prevent the collapse of a hole and installing poorly graded fine gravels which are selected to meet the general requirements of filter gravels. The average size of the gravel is 3mm. Sand compaction piles are positioned at the toe and beneath the embankment which are named as cases 11 and 12, respectively. The sand compaction piles are constructed using pipe casings, a vibrating hammer, and a compaction rod. All the model structures are so constructed as to simulate the field situations as much as possible.

12 piezometers, 4 accelerometers , and 2 LVDT's are instrumented as shown in Fig. 1 at case 4,5,6 illustration diagram. Sands near the front wall side of the test box are colored along the vertical and horizontal lines, that are spaced 5cm each, to visualize the patterns of deformations of the ground during and after excitations. Each model is shaken for 5 seconds each time with 4 Hz frequency. The level of cyclic acceleration is increased by 0.05g starting from 0.1g. Between the excitations, the quiet period is maintained for one minute to wait for the dissipation of the excess pore pressures developed at the preceding excitations.

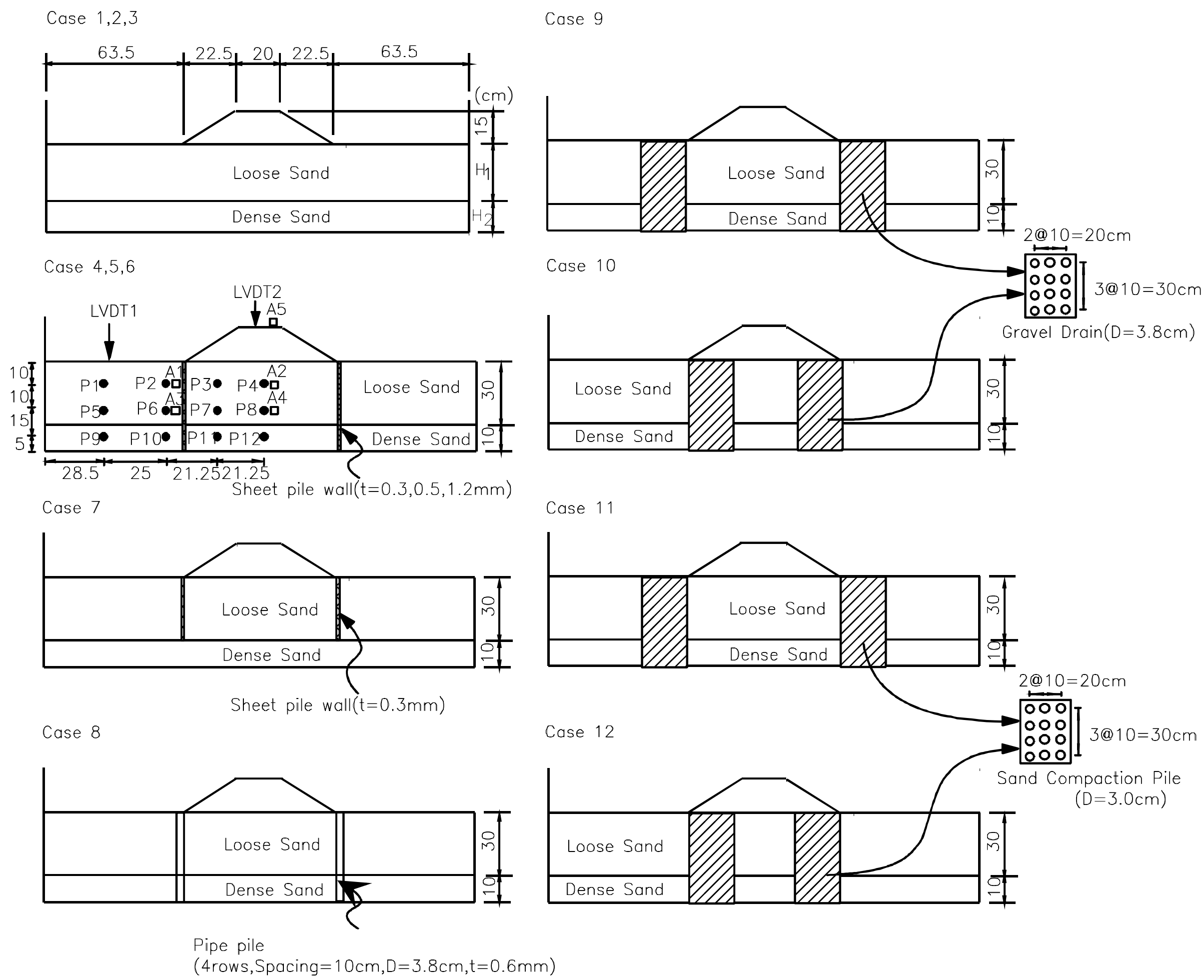
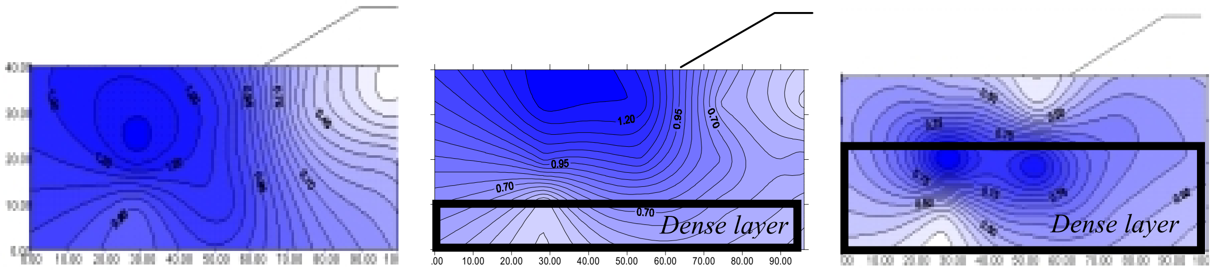


Fig 1: Schematic drawings of each test set-up

3. TEST RESULTS

3.1 Case 1,2,3 (No Countermeasure)

As is seen in Fig.2(a), (b) and (c), liquefaction occurs in the free field, which causes the failure of the embankment by lateral flow of the base ground. The free field liquefaction occurred at 0.15g for both cases 1 and 2, while it occurred at 0.3g for case 3, which reveals that liquefaction may occur more easily as the liquefiable layer becomes thicker. Referring to this result, the formation of the sand ground for the rest model tests is determined to be 30^{cm} of loose layer underlain by 10cm of a dense layer.



(a) Case 1 at 0.15g **(b) Case 2 at 0.15g** **(c) Case 3 at 0.30g**

Fig 2: Contours of excess pore water pressure ratios

3.2 Case 4,5,6,7 (Sheet-Piling)

Fig. 3 shows the contours of the excess pore water pressure ratios of case 6 at 0.15g. Liquefaction initiates at the same level of acceleration (0.15g), in this case, with that of case 2 in the free field. However, since the liquefaction region is contained in a smaller area as compared with Fig. 2(b), the failure of embankment occurred at 0.25g proving the effectiveness of the sheet-piling as a liquefaction countermeasure. These may be confirmed from LVDT measurement data as listed in Table 1, which shows big settlements in the free field at 0.15g, and at the crest of the embankment at 0.25g regardless of the thickness nor the embedment depth of the sheet-piling into the dense layer. From this observation, it is tentatively concluded that the thickness of the sheet piles and the embedment depth of the piles in the dense layer do not influence the liquefaction resisting capacity much.

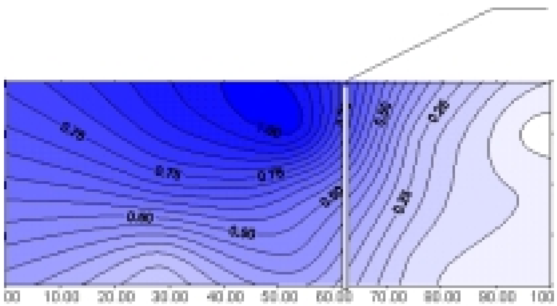


Fig 3: Contours of excess pore water pressure ratios for case 6 at 0.15g

Table 1. LVDT measurements of case 4,5,6 and 7 (unit : mm)

	Free field					Crest of embankment				
	010g	0.15g	0.20g	0.25g	0.30g	0.10g	0.15g	0.20g	0.25g	0.30g
case 4	0.09	>47	0.03	-	-	0.2	4.4	1.6	54	-
case 5	0.09	35	0.4	7.8	-	0.2	3.7	2.0	56	32
case 6	0.14	>43	0.03	0.03	-	0.5	4.0	2.6	58	27
case 7	0.09	46	0.4	4.8	-	-	-	-	60	-

3.3 Case 8 (Pipe-Piling)

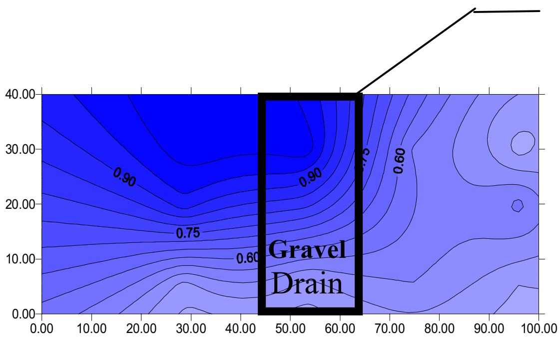
In this particular case(Table 2), initial liquefaction started at 0.2g in the free field. This may occur as a consequence of pipe pile driving which densified loose sands to a certain extent. Embankment failure occurred at 0.35g.

Table 2. LVDT measurements of case 8 (unit : mm)

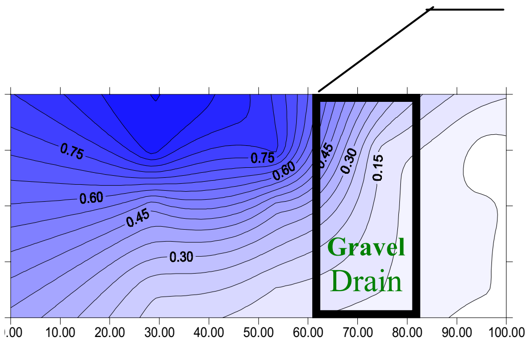
	Free field						Crest of embankment					
	0.10g	0.15g	0.20g	0.25g	0.30g	0.35g	0.10g	0.15g	0.20g	0.25g	0.30g	0.35g
case8	0.1	3.2	30	1.6	0.1	3	0.2	0.4	2.5	2.3	0.8	39

3.4 Case 9,10 (Gravel Drain)

As are shown in Fig. 4, much bigger excess pore pressures are developed in case 9 compared to case 10 at 0.15g. LVDT 2 readings taken at the embankment crest(Table 3) also support this observation showing much bigger settlement of the embankment in case 9(20mm) than case 10(1.4mm) at the acceleration level of 0.15g. Due to the excessive settlement of the embankment occurred at 0.15g in case 9, it is uncertain whether the gravel drains installed at the toe of the embankment would work as a countermeasure against liquefaction in a real situation. Embankment failure occurred at 0.25g in case 10.



(a) Case 9 at 0.15g



(b) Case 10 at 0.15g

Fig 4: Contours of excess pore water pressure ratios

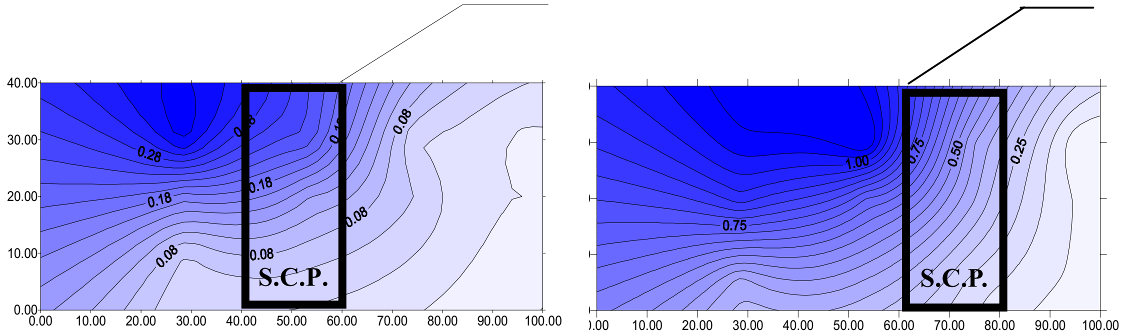
Table 3. LVDT measurements of case 9 and 10 (unit : mm)

	Free field						Crest of embankment					
	0.10g	0.15g	0.20g	0.25g	0.30g	0.35g	0.10g	0.15g	0.20g	0.25g	0.30g	0.35g
case9	0.03	22	0.17	1.05	0.64	7.2	0.6	20	1.3	5.5	5.2	43
case10	0.06	21.8	0.55	24.5	0	-	0.2	1.4	2	49	5.0	-

3.5 Case 11,12 (Sand Compaction Piles)

Comparing Fig. 5(a) with 5(b), it is noticed that the free field of case 12 at 0.3g is partly liquefied, but the free field of case 11 is not at the same g's level. LVDT readings taken in the free field supports this fact as shown in Table 4. In case 12, the crest of the embankment also starts to settle from 0.3g excitation, which increases as the g's level of excitation increases.

For the case 11, the free field is not liquefied until it is excited at the maximum g's level in the test programs as can be seen in Table 4. Meanwhile, the LVDT readings taken at the embankment crest show large settlement of the crest at 0.4g, which is totally attributed to the large displacement of the embankment slope caused by the dynamic instability of the slope itself(photo 1).



(a) Case 11 at 0.3g **(b) Case 12 at 0.3g**

Fig. 5 Contours of excess pore water pressure ratios

Table 4. LVDT measurements of case 11 and 12 (unit : mm)

	Free field							Crest of embankment						
	0.10 g	0.15 g	0.20 g	0.25 g	0.30 g	0.35 g	0.40 g	0.10 g	0.15 g	0.20 g	0.25 g	0.30 g	0.35 g	0.40 g
Case11	0.02	0.02	0.02	0.02	0.8	0.12	4	0.2	0.2	0.2	0.2	0.6	2.4	44
Case12	0.15	0.08	2.2	0.83	43	0.02	-	0.2	0.2	0.2	0.8	5	12.0	18.8

Photo 1. Deformed embankment of case11 after 0.4g excitation

4. CONCLUSIONS

1) Initial liquefaction occurring in a free field is followed by embankment failure as a result of lateral flow of the base ground, and liquefaction occurs more easily as a liquefiable layer becomes thicker.
2) The sheet-piling is proven to be an effective countermeasure against liquefaction. And, it is tentatively concluded that the stiffness of the sheet piles and the embedment depth of the piles into dense layers do not influence the level of liquefaction resisting capacity much.
3) It is uncertain whether the gravel drains installed at the toe of the embankment would work as a proper countermeasure against liquefaction.
4) Sand compaction piles are found to be the most effective tool in mitigating the liquefaction damages, which should be installed at the toe of, rather than right beneath, the embankment for better result against liquefaction.

5. REFERENCES

Adalier, K., Elgamal, A. K., and Martin, G. R.(1998), "Foundation Liquefaction Countermeasures For Earth Embankments," *Journal of Geotechnical and Geoenvironmental Engineering*, *ASCE* Vol. 124, No.6, pp500□517.
Koga, Y. and Matsuo, O.(1990), "Shaking table tests of embankments resting on liquefiable sandy ground," *Soils and Foundations*, Vol. 30, No. 4, pp162-174.
Marcuson, W. F., Hadala, P. F., and Ledbetter, R. H. (1996) "Seismic rehabilitation of earth dams," *J. Geotech. Engrg*., *ASCE*, 122(1), pp7-20.
PHRI. (1997), *Handbook on liquefaction remediation of reclaimed land,* A.A.Balkema, Rotterdam & Brookfield.
Mitchell, J. K., Baxter, C. D. P. and Munson, T. C. (1995) "Performance of improved ground during Earthquakes." *Soil improvement for liquefaction hazard mitigation*, Geotech. special Pub. No. 49, *ASCE*, pp1-36.
Mitchell, J. K., Cooke, H. G, Schaeffer, J. A.(1998) "Design considerations in ground improvement for seismic risk mitigation," *Geotechnical Earthquake Engineering and Soil Dynamics* III, Geotech. special pub. No. 75, ASCE, Vol. 1, pp580 – 613.
Zheng, J., Ohbo, N., Suzuki, K., Suzuki, R., Mishima, N. and Nagao, K.(1995), "Analysis of results of centrifuge tests on seismic behavior of embankment", *First International Conference on Earthquake Geotechnical Engineering*, Vol. 2, pp1069 – 1074.

1170/5/A

LIQUEFACTION OF EMBANKMENTS ON SANDY SOILS AND THE OPTIMUM COUNTERMEASURE AGAINST THE LIQUEFACTION

Young-Ho PARK[1]

Sung-Ryul KIM[2]

Sung-Hwan KIM[3]

Myoung-Mo KIM[4]

ABSTRACT

Shaking table tests for 12 different cases are performed to investigate the behavior of embankments resting on liquefiable sand grounds in a model box, and to compare the effectiveness of several methods of countermeasures in mitigating earthquake induced liquefaction phenomena. The countermeasures employed here are installation of sheet-pile walls, gravel drains, sand compaction piles, and pipe piles. The dimensions of the transparent model box are 194^{cm} long, 44^{cm} wide, and 60^{cm} deep. Saturated sand grounds are prepared for the tests to have a relative density of about 30% by water sedimentation method, and 15cm high embankments are constructed on the model grounds. 12 pore water pressure transducers, 4 accelerometers, and 2 LVDTs are installed throughout the model set-ups to monitor the response of the ground as well as the embankments.

The shaking table, whose size is 2m x 2m, is excited from 0.1 g up to 0.4 g. Sand compaction piles are found to be the most effective in mitigating the liquefaction damages among the methods applied in the model tests, even if all the methods are proven to be effective tool in preventing the occurrence of liquefaction at normal range of excitations. Further, the optimum location of each countermeasure is also determined in this study.

KEYWORDS

[1] *Hyundai Institute of Construction Technology, 102-4 Mabuk-li, Kuseong-myun, Yongin-si, Kyounggi-do, Korea E-mail:yhpark@hyundai.hdec.co.kr*

[2] *Department of Civil Engineering, Seoul National University, San 56-1, Shinlim-dong, Kwanak-gu, Seoul, Korea E-mail:yawnful@chollian.net*

[3] *Korea Highway Cooperation, 293-1 Kumto-dong, Sujeong-gu, Sungnam-si Kyounggi-do, Korea E-mail:shkim@freeway.co.kr*

[4] *Department of Civil Engineering, Seoul National University, San 56-1, Shinlim-dong, Kwanak-gu, Seoul, Korea E-mail:geotech@gong.snu.ac.kr*

liquefaction, countermeasure, ground improvement, optimum design, shaking table testing, embankment, free field, lateral flow, excess pore pressure