Verification of Desaturation Technique as a Liquefaction Countermeasure for Existing Embankments

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Abstract

Highways play a key role of providing not only usual but emergency means of transportation such as during earthquakes. In Shikoku area of Japan, a total of about 10 km highway embankments are expected to be severely damaged due to liquefaction of thick foundation sand during earthquakes. Application of available liquefaction countermeasure techniques had been initially considered for the foundation soil below the existing embankments, but the construction cost resulted to be far beyond the budget. Geotechnical engineers have always had a challenge of applying remedial works for enormous length of existing embankments, which are particularly susceptible to liquefaction-induced damage. In this paper, desaturation method by air injection, a newly developed liquefaction countermeasure technique of dramatically low cost, is introduced. This technique has attracted a considerable amount of attention in recent years as an innovative technique which can be applied to soils beneath the existing structures. The effects of the technique on seismic performance of embankments are examined through a series of centrifuge tests.

Keywords: Liquefaction countermeasure, centrifuge test, earthquake.

1. Introduction

In design practice of earth fills in Japan including highway embankments and river levees, the effect of earthquake-induced foundation liquefaction is not taken into account. For example, although river levees have often been damaged during past large earthquakes in Japan, no earthquake effect was considered up until 1996. This is partly because such damages did not cause any significant loss of human lives and properties, and additionally, it was not difficult to restore the damaged embankments in a short period. In most occasions during the past earthquakes, the temporary embankment and levee restoration works were completed within a week or two.

The Hyogoken-nambu Earthquake in 1995 damaged a 6-m high Yodo river levee. It subsided by as much as 3 -m (Matsuo, 1996), and the highly urbanized hinterland was in real danger of possible overflow of river water. After this earthquake, the Ministry of Construction started a remediation program against a liquefaction-induced failure of vulnerable levees (Public Works Research Institute, 1998). With respect to highway road embankments, however, foundation liquefaction has not been considered yet. As highways play a key role of providing routes for emergency transportation such as during earthquakes, especially in areas of poor road network development, the liquefaction remedial work for the existing highway embankments is very important.

Liquefaction countermeasure techniques have extensively been used to ameliorate liquefaction resistances of loose sand deposits. Most of the techniques, however, are limited to improve foundation soils without any structures. Techniques that can be applied to soils below existing structures are few and particularly expensive (JGS, 1998). A challenge that geotechnical engineers have been facing is the remedial countermeasure works for enormous length of existing embankments and levees which are susceptible to liquefaction-induced damage. In this paper, a newly developed liquefaction countermeasure technique of dramatically low cost, the desaturation method by air injection, is introduced. This technique attracts considerable attention in recent years as an innovative...
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2. Desaturation by air injection as a liquefaction countermeasure

2.1 Factors affecting liquefaction resistance of unsaturated sand

Existence of air in pores in soil is considered to enhance liquefaction resistance. Air in the pores plays a role of absorbing generated excess pore pressures by reducing its volume. The bulk modulus and change in volume of the pore fluid, that is air water mixture, may be the factors dominating this mechanism.

Considering a soil mass of its pore filled with air and water. For a small change in the pore pressure, $Dp$, we obtain the volumetric strain of the soil using the Boyle’s law as,

$$\kappa_v = \frac{\Delta p}{p_0 + \Delta \sigma} = \frac{\Delta \varepsilon}{\sigma'} - \frac{\varepsilon}{\sigma'}$$

where $p_0$ and $e$ denote the absolute pressure of the fluid and the void ratio of the soil mass. In the above equation, compressibility of soil grain and water are ignored. The highest value of the volumetric strain for the soil is achieved when the $Dp$ attains its possible maximum value which is equal to the initial effective confining stress, $\sigma'_c$. This highest value of the volumetric strain is hereafter termed as the potential volumetric strain, $\kappa^*_v$.

Effects of the factors derived above have been investigated through a series of triaxial tests on Toyoura sand specimens at a relative density of 40%. Three testing parameters including the initial effective confining pressure, $\sigma'_c$, the buck pressure, $p_o$, and the degree of saturation, $S_r$, were varied between tests while the void ratio of the specimens was kept constant throughout the test series. It should be noted that the back pressure, $p_o$, is the absolute pressure instead of the ordinary used gauge pressure. It has been confirmed that the liquefaction resistances of the partially saturated sand increase with the initial confining pressures, with the liquefaction resistances being higher for lower $S_r$. Also the liquefaction resistance depends on the back pressure; the liquefaction resistance decreases as the pressure increases (Okamura and Soga, 2006).

The liquefaction resistance ratio, which is the liquefaction resistance of partially saturated sand normalized with respect to that of fully saturated sand, is plotted against the potential volumetric strain in Figure 2(a). All the data lies along a unique curve, confirming that the potential volumetric strain is the determining factor of the effect of degree of saturation on this specific sand at relative density of 40%. Data retrieved from literature as well as that obtained from tests on in-situ sands is also shown in Figure 2(b) in the same manner. The data plotted in this figure was obtained from tests on specimen prepared using different sand at different relative density and at different confining pressures. Despite the different test conditions, all the data lies along the same curve. This indicates that the effect of the degree of saturation on liquefaction resistance is dominated by the factor $\kappa^*_v$. More detail information about the cyclic triaxial tests on the unsaturated sand can be found in Okamura and Soga (2006) and Okamura and Noguchi (2009).

2.2 Centrifuge test

Countermeasure technique discussed in this study is desaturation of soil below existing embankments by air injection (Okamura et al., 2011). Three models were prepared as shown in Figure 3; one is saturated foundation soil model (model 1) and the others are desaturated foundation soil model by air injection (models 2 and 3). The model consisted of a 2 m high well
2.2.1 Air injection

Air was injected in model 2 and 3 at 40g through the injectors set at the bottom of the loose sand layer. Figure 4 indicates time histories of air pressure supplied to the injector, air flow rate and change in pore pressure measured at bottom corner of the container in model 2. The flow rate and water level started rising at \( t = 500 \) sec., indicating that air began flowing into the soil. This timing coincided when the air pressure reached \( P_{\text{inj\,min}} = P_{\text{hyd}} + AEV \), where \( P_{\text{hyd}} \) and \( AEV \) denote the hydrostatic pressure at the depth of the injector and the air entry value of the soil, respectively. The water level rose as the injection pressure increased. After the air injection halted at \( t = 1180 \) sec. the pore pressure settled to the residual pressure 2.5 kPa higher than that before the air injection. The rise in the water level is equivalent to a volume of air in the soil. It was observed by an on-board video camera during the air injection that color of the sand in the desaturated area was changed. This area, shown in Figure 3(b), was consistent to that obtained by detailed eye observation after the centrifuge was stopped after the shaking test. The residual degree of saturation in the desaturated areas was approximately 85% in model 2 and 3.

An excessive injection pressure may loosen the sand around an injector. Figure 4 depicts settlement of the embankment. It was found that the injection pressure did not cause noticeable heave or subsidence to the embankment. The desaturated zones in model 2 and 3 shown in Figure 3 extended upward from the injection ports with the breadth of the zones being wider with decreasing depth. In model 2, air was injected from one injection port on the centerline, while in model 3, two injection ports were used to desaturate wider zone of the foundation soil.

2.2.2 Shaking test

Hereafter in this paper, all the test results are presented in prototype scale, otherwise mentioned. On completion of desaturation process, ample time was allowed so as to drain excess pore fluid until the height of water table coincided to the level 0.4 m below the ground surface. Then one dimensional lateral shaking was imparted along the model long axis using a mechanical shaker while in flight at 40g. Three seismic events, that is weak, moderate and strong shaking events, were imparted to each model. During the weak and the moderate shaking events with a peak input acceleration of 0.7 m/s² and 2.0 m/s², respectively, any significant displacement and excess pore pressures generation were observed in all the models. The acceleration time histories of the strong shaking event have the shape indicated in Fig. 4(a), with peak accelerations of about 2.5
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Figure 3. Centrifuge models tested at 40g

Figure 4. Time histories of air pressure and flow rate. Crest settlement was essentially insensible to air injection (in model scale)

Figure 5. Input acceleration and excess pore pressure responses during strong shaking event.

Figure 6. Maximum excess pore pressure distribution

m/s² and lasted 20 seconds in the prototype scale. Time histories of observed excess pore pressures (EPP) at locations B1 and B3 are shown in Fig. 5. Distribution of the maximum excess pore pressure is depicted in Fig. 6. Note that effective overburden stress, \( s'_v \), indicated in these figures are obtained simply from the depth of soil above the pore pressure cells.

Significant effects of soil desaturation on the EPP can be seen in this figure. At B1, the EPP of the benchmark model (model 1) sharply increased as soon as the shaking event initiated and then leveled off. While for model 2 and 3, in which this location was in the desaturated zone, the EPPs were remarkably low. At location B3, EPP of model 1 also attained its maximum value as high as the initial effective overburden pressure and EPP of model 3 stayed very low throughout the shaking event. It is of interest to note that although the location B3 in model 2 was well out of the desaturated zone, EPP was significantly lower than that of model 1. This was also the case at the location B4 that EPPs...
observed in model 2 and 3 at B4 were apparently lower than model 1 even though this location is in the saturated zone for all the models. Migration of pore fluid might occur and EPP of saturated zones was absorbed by the desaturated zone. It is confirmed that a decrease in degree of saturation of soil below the embankment has a significant effect to restrain excess pore pressure generation and the desaturation effect is not limited within the desaturated zone but extends to the neighborhood.

Settlement of the embankment crest is depicted in Figure 7. Figure 8 shows deformation of models after the test. The crest settlements for model 2 and 3 were less than one-seventh of that for model 1. It can be concluded that countermeasure to soil beneath embankment has significant effects to reduce embankment settlement.

3 Numerical analysis
Deformation analysis of the centrifuge models using the numerical code, ALID (Yasuda et al., 1999), was performed in this study. In ALID, shear rigidity of soils which is degraded due to soil liquefaction is estimated based on the empirical relation between degradation ratio and factor of safety against liquefaction, FL (Yasuda et al., 1999), and liquefaction-induced deformation is evaluated by a self-weight analysis. FL of the centrifuge models was computed according to the method specified by JRA (Japan Road Association, 2002). The liquefaction strength ratio of Toyoura Sand at Dr or 60% to cause double axial strain amplitude of 5% in 20 cycles, RL20, was determined from undrained cyclic triaxial tests (Japanese Geotechnical Society, 2009) as 0.136. The liquefaction strength of the unsaturated zone was obtained by multiplying the liquefaction resistance ratio indicated in Fig. 2. The initial shear rigidity of embankment was decided as \( G_0 = 14.6 \) MPa based on the measured shear wave velocity.

It was found that FL value in whole region below ground water level of all three centrifuge models was lower than unity, which fact was inconsistent with test observations. This discrepancy may be due to the fact that the tests were carried out using a rigid container with relatively narrow width. The constraint effects induced by the container rigid walls resulted in apparent liquefaction strength of the soil being higher. The apparent liquefaction strength ratio determined as follows, therefore, was used in the ALID analysis in this study. Considering the fact that soil at the location B4 in model 1 liquefied at seventh cycles with the average acceleration amplitude of 190 gal, and that the triaxial strength ratio corresponding to \( N = 7 \) cycles is 0.130 as shown in Fig. 9, apparent liquefaction strength ratio may be 3.4 times higher than the triaxial strength.

Figure 9 Liquefaction strength curve (triaxial tests) (Tatsuoka, F. et al. (1984))
Figure 10 shows distribution of FL value of all three models. FL value of whole region in case 1 is lower than unity, while in the desaturated region in cases 2 and 3, FL value is higher than 1.2.

Figure 11 depicts deformation of models for cases 1 through 3 obtained from ALID. For model 1 the liquefied soil flowed outward significantly in the similar way to that observed in the test. In cases 2 and 3 with the desaturated zones, deformation of foundation soil is limited due to the existence of non-liquefied zone.

Figure 12 shows maximum shear strain contours of centrifuge models derived from deformation of the lattice points of the colored sand. In model 1 without desaturation zone, maximum shear strain ranged between 4% and 18%. While for case 2 maximum shear strain was lower than 5% and significant deformation appeared only in the vicinity of embankment toes and surface of level ground. This may indicate that desaturated is not very effective to region where overburden stress is low. Maximum shear strain in the case 3 with wider desaturated zone was lower than 3%.

Figure 13 shows contours of maximum shear strain (ALID)
Cumulative crest settlement is compared in Figure 14. Though the numerical analysis tends to underestimate crest settlement of model tests, the crest settlement decreased with increasing area of desaturation in the same way as the test results.

4 Concluding remarks

Effects of desaturation of liquefaction prone soil below embankment by air injection were studied through centrifuge tests. Models of fully saturated foundation soil with embankment were desaturated in-flight and subjected to base shaking. The injected air distributed in soil below the embankment and residual degree of saturation of the desaturated soil after the injection was estimated approximately 85%. This slight decrease in degree of saturation restricted significantly the excess pore pressure generation during shaking, especially at locations with higher effective overburden pressures. The settlement of embankment was decreased by 90%. Excess pore pressure of soil in saturated zone was found to be absorbed by the adjacent desaturated zone. The effectiveness of desaturation on mitigating crest settlement was confirmed.

Deformation analysis of the centrifuge models using the numerical code ALID was performed to validate the code. Although the deformation obtained from ALID is smaller than that observed in the tests, good qualitative similarity of overall pattern of deformation between model tests and numerical simulation is confirmed.

References


