Effects of counterweight fill on reduction of settlement of main embankment constructed on sandy soil deposits liquefied during earthquakes

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ABSTRACT

When long-stripe structures used as road embankments are constructed on liquefied sandy soil deposits, several countermeasures against liquefaction using soil improvement methods such as sand compaction pile method or deep mixing method are generally applied to the sandy ground directly under the fill slope in the longitudinal direction of the structures. However, such improvement methods are costly. When the site is not restricted to a narrow area and the land price is low, counterweight fill methods are usually applied to soft clayey soil deposits as a static stabilization countermeasure. In the present study, a series of residual deformation analyses during earthquakes was performed in order to clarify the effects of the counterweight fill on the reduction of settlement of the main embankment constructed on liquefied ground, where the counterweight fill was applied to the liquefied ground as a countermeasure against liquefaction without improvement of soil ground described above. Static analyses using the FEM analysis program ‘ALID’ were conducted for three sections of counterweight fill models with different heights and without counterweight fill. Consequently, the earthquake-induced residual settlement of the main embankment, corresponding to the roadway part, decreased when the width and height of the counterweight fill was long and high, respectively. Therefore, reduction in residual deformation may be possible due to the application of the counterweight fill without the soil improvement methods.

Keywords: liquefaction, level 2 earthquake, residual deformation analysis, counterweight fill

1 INTRODUCTION

When loose sandy soil layers lying at a depth greater than the ground water level are liquefied during an earthquake, a large residual deformation occurs in earth structures constructed on the liquefied ground due to the degradation of stiffness and the dissipation of excess pore water pressure in the liquefied layer. Therefore, functions of the earth structures may be lost during the event. For example, assuming that a road embankment on a principal line such as an emergency route constructed on a loose sandy soil layer susceptible to liquefaction is struck by a large earthquake, so-called level 2 earthquake, a certain degree of deformation damage in that road embankment is expected. However, the road embankment has to possess efficiency such that the deformation damage is within the scope of the earth structure being restored with ease after the earthquake, since the road embankment is constructed along an emergency route.13

Since it is forecasted that earthquakes due to active faults with a magnitude of 7.0 or mega earthquakes due to the Nankai Trough located off the Pacific coast of Japan will occur in the near future, examination of the seismic stability considering a level 2 earthquake is also essential in road embankments along the emergency routes in Kyushu Island.

In many cases, countermeasures against liquefaction using soil improvement methods such as the sand compaction pile method or deep mixing method are generally applied to the sandy ground directly under the fill slope in the longitudinal direction of the structures. However, the improvement methods must be applied directly under not only the slope surface but the crown of the embankment. This activity would be extremely costly, if the improvement methods described above were applied to the ground taking into account level 2 earthquakes.

The counterweight fill method could be much less costly than the other countermeasures, if the site is not restricted to a narrow area, the land price is low, there are no particular problems in acquiring the landownership, the banking materials can be obtained at a low price and that improvement method can be an effective countermeasure against liquefaction.

The effectiveness of the counterweight fill method is discussed in this paper, comparing data on the residual deformation due to liquefaction during level 2 earthquakes.
2 ANALYSIS PROCEDURE AND CONDITION

A series of residual deformation analyses used for finite element method were performed to clarify the effectiveness of the counterweight method in reducing the residual deformation of the main part of an embankment subjected to seismic loading of a level 2 earthquake.

2.1 Case and flowchart of analysis

A schematic illustration of the analytical models for examining the effectiveness of the counterweight fill method used as a countermeasure against liquefaction is shown in Fig.1. Three cases were assumed in the analysis, where the road embankment was constructed on the liquefied sandy soil layer in every case. In Case 1, the main embankment without counterweight fill and in Cases 2 and 3, the main embankment with counterweight fill were assumed to be constructed, respectively. There was a difference between the height of the counterweight embankment in Cases 2 and 3, in which it was assumed that the height of the counterweight fill and the main embankment was the same in Case 2 and the height of the counterweight fill was higher than that of the main embankment in Case 3. It was also assumed that the crown of the main embankment was part of the carriageway and both the adjacent crowns were part of the sidewalk on the counterweight fill.

The analysis was performed under two-dimensional conditions. Fig.2 indicates the flowchart of the residual deformation analysis in this study. First, initial stresses acting on the element were calculated, and then seismic response analyses were performed to obtain shear stresses during earthquakes acting under the initial stresses on the elements. The level 2 earthquake motion was input to the base for the seismic design. In the calculation of safety factors against liquefaction, $F_L$-value, the liquefaction strength ratio, $R_{L20}$, obtained from cyclic undrained triaxial tests, were divided by the shear stress acquired from the seismic response analysis. Then, the reduction ratio of shear modulus was read from the figure showing the safety factors against liquefaction, $F_L$-value versus liquefaction strength ratio, $R_{L20}$, relationship of which was published in the guideline on road earthwork of countermeasure against soft soil ground. Furthermore, the residual deformation of the embankment due to liquefaction was obtained in the analysis, considering that the shear modulus was decreased based on the reduction ratio of shear modulus and thereafter also considering the dissipation of excess pore water pressure. A two-dimensional FEM program, ‘ADVANF/Win Ver.4’ was used in the seismic response analysis and a two-dimensional FEM program for liquefaction flow analysis, ‘ALID/Win Ver.5.0’, was used to calculate the initial stress, the $F_L$-value and the residual deformation due to liquefaction.

2.2 FEM analytical model

Fig.3 shows the two-dimensional FEM analytical model in Case 3 as the typical cross section of the ground. The upper figure indicates the entire model and the lower figure shows the main part of the model including the embankment portion, in which the foundation soil ground was composed of horizontal stratifications and the embankment built on the ground. The element used for FEM in Case 3 was about 1m in length of one side, the total number of nodes was 4660 and the total number of elements was 4476. Moreover, the boundary conditions for the response analysis were composed of an energy transmitting boundary in the lateral boundary surface and an elastic foundation with dasypot boundary in the basal plain. And a roller boundary in the lateral boundary surface and a hinge boundary in the basal plain were utilized for the deformation analysis using the program ‘ALID’.

2.3 Ground conditions

Table 1 indicates several kinds of data of the foundation soil ground used for the FEM analyses and
the ground was composed of three parts of alluvial layer, diluvial layer and weathered granite layer. In the three layers, the layers susceptible to liquefaction were the first alluvial sandy soil layer (As1-2), the second alluvial sandy soil layer (As2) and the third alluvial sandy soil layer (As3). The layer of As1-2 was classified as a sand mixed with silt, having the parameters of \( N = 4 \), \( F_c = 38.1\% \) and \( I_p = 10.8 \); the layer of As-2 was classified as a silty sand mixed with shells, having the parameters of \( N = 0-1 \) and \( F_c = 24.3\% \); and the layer of As-3 was classified as a low plasticity sand mixed with silt, having the parameters of \( N = 6 \) and \( F_c = 33.3\% \).

In order to perform undrained triaxial compression tests and cyclic undrained triaxial tests on the alluvial sandy soil layer, undisturbed samples from the As-2 layer with a small \( N \)-value that was assumed to be susceptible to liquefaction were picked up using a thin walled tube sampler. The cohesion, \( c' \), used for the analyses was the value obtained from the undrained triaxial tests and the internal friction angle, \( \phi' \), was the value estimated from the formula shown as \( \phi' = \sqrt{20N+15} \).

Furthermore, the value of cohesion was estimated as zero in the residual deformation analyses considering the degradation of shear modulus and the dissipation of excess pore water pressure. The liquefaction strength ratio, \( R_{L20} \), was evaluated from the cyclic stress ratio at the twentieth number of cycles using the data from the cyclic triaxial tests.

Fig. 4 shows the shear modulus, \( G \), and damping constant, \( h \), versus shear strain, \( \gamma \), relationships used in the analyses, which was evaluated using the values of confining pressure and mean grain size, based on the method of Yasuda and Yamaguchi (1985)\(^5\). However, no tests to obtain the dynamic deformation properties of the soil layers could be performed for economic reasons. Fig. 5 indicates a concept on the variety of the secant shear modulus of the shear stress-shear strain curve obtained from the static shear tests before and after the occurrence of liquefaction. \( G_{0,i} \) was the secant shear modulus at a shear strain of 0.1\% obtained from the static shear test without cyclic loading.

2.4 Input earthquake motion

The input earthquake motion used for the response calculation under seismic conditions was six type of waves for the first grade ground, which was classified as a level 2 earthquake including three waves according to Types I and II, published in the Specification for Highway Bridges Part V Seismic Design (2012)\(^6\). Type I is a large-scale earthquake occurring at the plate boundary in the Pacific Ocean and Type II is a strong local earthquake such as the 1995 Hyogoken-Nambu Earthquake induced by active faults. In the response calculation, the amplitude of the six earthquake motions was multiplied by a regional coefficient of 0.7 and the input modified earthquake motion was entered to the base ground surface under the 2E condition.

Moreover, the residual deformation in Case 3 was the
largest value, when the seismic wave observed at the ground around Kaihoku Bridge during the 2011 off the Pacific coast of Tohoku Earthquake as shown in Fig.6 was used for the input earthquake motion that is classified as level 2 and Type I earthquake for the first grade ground. Therefore, the residual deformation analyses were conducted using the seismic wave shown in Fig.6 as the input earthquake motion, in order to compare the effectiveness of the counterweight fill in reducing the residual deformation. Although the response calculation was also conducted in the case of the coefficient of 1.0, the severity of liquefaction was high in the sandy soil layer below the part of the embankments and the settlement of the counterweight fill increased and that of the main embankment decreased comparing with the results in the case of the coefficient of 0.7.

3 RESULTS OF ANALYSIS

3.1 Seismic response analysis

Fig.7 indicates the contour of the maximum shear strain obtained from the seismic response analysis. The shear strain directly under the embankment reached about 5% and shear strain pretty greater than 5% did not occur. Therefore, it is judged that the response calculation provided a reasonable result.

Fig.8 shows the distribution of the maximum acceleration in the seismic response analysis. The amplification of the acceleration was great in the layer deeper than the liquefied layer and the value of the maximum acceleration reached about 550 Gal. On the contrary, the value of the acceleration was relatively small in the layer shallower than the liquefied layer.

Table 1. Several kinds of data on the foundation soil ground

| Stratum      | Soil      | GL(m) | H      |平均N-値 |γ' | σ' | Fc (%) | D30 (mm) | D10 (mm) | b | c, c' | ϕ, ϕ' | Rl20 | Vs (m/s) | G0 (kN/m²) | G0,i (kN/m²) | γ   |
|--------------|-----------|-------|--------|---------|---|---|-------|---------|---------|---|------|-------|------|---------|---------|------------|---|
| Alluvial layer | Sandy soil | -0.70 | 0.70   | 0.70   | 0.70 | 0.70 | 0.70 | 0.70 | 0.70 | 0.70 | 0.70 | 0.70 | 0.70 | 0.70 | 0.70 | 0.70 |
| Dāulval layer | Sandy soil | -1.30 | 1.30   | 1.30   | 1.30 | 1.30 | 1.30 | 1.30 | 1.30 | 1.30 | 1.30 | 1.30 | 1.30 | 1.30 | 1.30 | 1.30 |
| Weathered granite layer | Sandy soil | -3.00 | 3.00   | 3.00   | 3.00 | 3.00 | 3.00 | 3.00 | 3.00 | 3.00 | 3.00 | 3.00 | 3.00 | 3.00 | 3.00 | 3.00 |

*: It was judged that the soil was a low plastic sandy soil by touching the soil samples, although no liquid and plastic limit tests were conducted.

GL: Depth (m), H: Thickness of layer (m), γ': Effective overburden pressure at the center of thickness (kN/m²), Fc: Fine content ratio (%), D30:10% diameter on the grain size diagram (mm), b: Plasticity index, c, c': Cohesion (kN/m²), ϕ, ϕ': Internal friction angle (degree), Rl20: Liquefaction strength ratio, Vs: Shear wave velocity (m/s), G0: Initial shear modulus (kN/m²), G0,i: Shear modulus at the shear strain of 0.1%, ν: Poisson's ratio
small in the layer shallower than the liquefied layer and the embankment, because the amplification of the acceleration was restrained under the influence of the liquefied layer. Similar trend on the behavior of the maximum response acceleration was observed in the analyses using the other Type I earthquake motions.

Fig.9 indicates the contour of the $F_L$-value in which the part colored in red shows $F_L < 1.0$ and liquefaction occurred in the red sphere. The $F_L$-value just under the counterweight fill was about 0.8 to 1.0 and relatively high, compared with about 0.6 to 0.8 just under the main embankment, because the intensity of liquefaction was reduced due to the confining effect of the fill.

3.2 Residual deformation analysis

Fig.10 indicates the contour of the settlement observed in Cases 1 to 3. The portion where the settlement was greater than 25 cm is colored in red because the allowable value of the residual settlement in the road embankment was 25 cm during earthquakes. The allowable settlement was 25 cm based on the Earthquake-Resistant Performance Diagnostic Criteria (2008) considering the passable function of a road just after the occurrence of earthquakes. Namely, the difference in levels smaller than 25 cm can be repaired relatively easily and early by urgent restoration works. The difference in level is replaced with the settlement, because the settlement is always larger than the difference in level. The residual deformation of the main embankment in Case 3 was smaller than those in Cases 2 and 1 and deformation in Case 2 was smaller than that in Case 1. Therefore, the residual deformation can be reduced by the counterweight fill without applying the soil improvement methods. The settlement of the main embankment with counterweight fill in Cases 2 and 3 was smaller than 25 cm and was within the allowable...
value in every point. Moreover, the settlements of the main embankment with the higher and the lower counterweight fill in Cases 3 and 2 were about 16 cm and 23 cm, respectively.

Fig.11 indicates the displacement vector obtained from the residual deformation analysis in Case 3. The displacement was larger in the counterweight fill part than in the main embankment part in both the horizontal and vertical directions, and the horizontal displacement in the counterweight fill portion occurred significantly in both the sides with no fills.

4 CONCLUSIONS

In the present study, the effects of the counterweight fill on reducing the residual deformation of the main embankment due to liquefaction of the foundation soil under level 2 earthquakes were investigated by static FEM analysis. Consequently, the counterweight fill can reduce the settlement of the main embankment without any countermeasures.

However, whether the liquefaction-induced residual deformation during earthquakes actually shows behaviors similar to the results observed in this study is a topic requiring further investigation. Residual deformation analyses calculating transient behaviors are necessary to investigate the process of deformation and the final residual deformation accurately.

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