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ANALYSES OF LIQUEFACTION-INDUCED DEFORMATION OF GORUNDS AND STRUCTURES BY A SIMPLE METHOD

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ABSTRACT

The authors proposed a simple method to estimate liquefaction-induced deformation of grounds and structures. Adaptability of the method to the settlement of footings was studied. Centrifuge tests for the footing of transmission tower were selected to demonstrate the adaptability of the method. Analyses were carried out under the same conditions of the centrifuge tests. By comparing the analyzed results with the tested results, it was concluded that the effect of thickness of liquefied layer and soil density on the settlement of footing can be evaluated well by the simple method. Effect of countermeasures on the settlement of footing also could be evaluated well.

INTRODUCTION

Liquefaction of loose sandy ground often causes severe damage to structures. The surest way to avoid damage is by improving the sandy soil so it will not liquefy. However, soil improvement is expensive and it is not easy to apply to existing structures. Furthermore, if the design input earthquake motion is very strong, it is difficult to prevent the occurrence of liquefaction by some soil improvement methods. For this situation, it is desirable to develop alternative, more rational, design methods in which the occurrence of liquefaction is accepted but the structure is designed to survive it without damage.

In these design methods, deformation of structures in liquefied ground must be estimated. The authors have developed a simple estimation method for the liquefaction-induced deformation. Adaptability of this method to ground flow and river levees have been demonstrated in previous papers. In this paper, outline of the method and adaptability to ground flow and river levees are introduced briefly. Then adaptability of the method to the settlement of footings is discussed.

OUTLINE OF PROPOSED METHOD

The authors have developed a simple method named "ALID (Analysis for Liquefaction-induced Deformation)" to estimate liquefaction-induced deformation of grounds and structures (Yasuda et al., 1999). Figure 1 shows the flow chart of the analysis. Finite element method is applied twice as follow:

(1) In the first step, the deformation of the ground is calculated by the finite element method using the shear modulus before earthquake.

(2) The finite element method is applied again by using the decreased shear modulus due to liquefaction with the

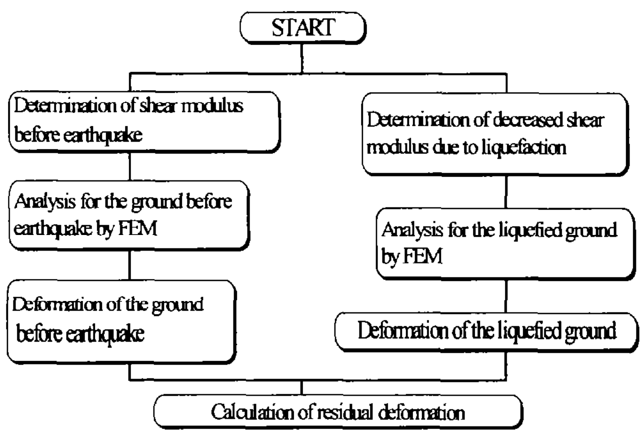


Fig. 1 . Flow chart of the analysis.

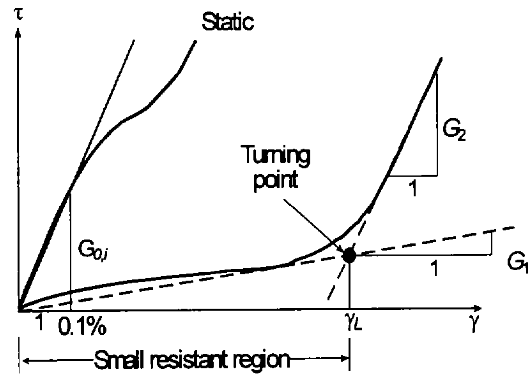


Fig. 2. Definition of G_{0i}, G_1, G_2 and γ_L

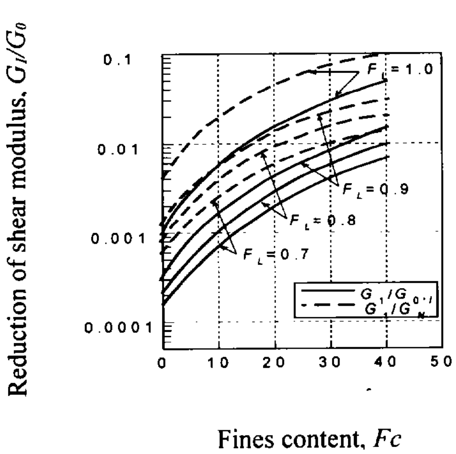


Fig. 3. Summary of test results.

condition of no volume change.
(3) The difference in the deformation measured by the two analyses is supposed to equal the residual ground deformation.

The authors conducted torsional shear tests to obtain the reduction rate of shear modulus due to liquefaction (Yasuda et al., 1998). A prescribed number or prescribed amplitude of cyclic loadings was applied to samples in undrained condition. Safety factor against liquefaction, F_L, which implies severity of liquefaction was controlled by the number of cycles or amplitude of the cyclic loadings. After that, a monotonic loading was applied under undrained condition.

In the liquefied specimen, shear strain increased with very low shear stress up to very large strain as schematically shown in Fig. 2. Then, after a resistance transformation point (turning point), the shear stress increased comparatively rapidly with shear strain, following the decrease of pore water pressure. The amount of strain up to the resistance transformation point is called the "reference strain at resistance transformation, γ_L" as shown in Fig. 2. Stress-strain curves before and after the reference transformation point can be presented approximately by a bilinear model with G_1, G_2 and γ_L :

$$\tau = G_1 \gamma \qquad \text{for } \gamma < \gamma_L \qquad (1)$$

$$\tau = G_1 \gamma_L + G_2(\gamma - \gamma_L) \qquad \text{for } \gamma \geqq \gamma_L \qquad (2)$$

where G_1 and G_2 are the shear moduli before and after the reference transformation point, respectively. To know the reduction rate of shear modulus due to liquefaction, the rate of shear modulus G_1/G_0, which is the ratio of shear modulus after and before liquefaction, was calculated. Relationships among the shear modulus ratio, G_1/G_0 and fines content less than 75 μm, Fc, in the range of F_L =0.7 to 1.0 were summarized as shown in Fig. 3.

ADAPTABILITY OF THE PROPOSED METHOD TO GROUND FLOW AND RIVER LEVEES

Analyses for liquefaction-induced ground flow

The proposed method "ALID" was first applied to estimate liquefaction-induced ground flow (Yasuda et al., 1999). Two type of ground flow which occurred during past earthquakes were selected to demonstrate the adaptability of the method. As the first type of flow, deformation of the ground behind a quay wall which occurred at Uozakihama in Kobe during the 1995 Hyogoken-nambu (Kobe) earthquake, was selected.

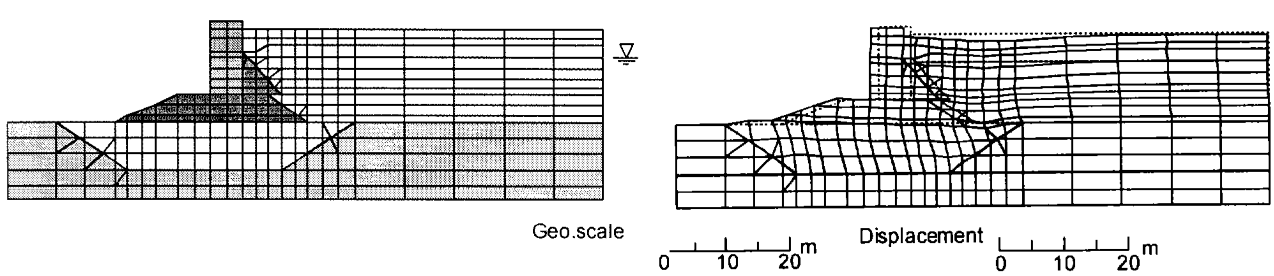


Fig. 4. Deformation of the ground behind a quay wall in Kobe (Yasuda et al., 1999)

Horizontal displacement of the wall was 2.0 m and the flow extended to more than 100 m from the wall during the earthquake. In the analysis, rate of shear modulus of liquefied layer was evaluated based on the data obtained by torsional shear tests on liquefied soil which is same as the rate shown in Fig.3. And the rate of shear modulus for non-liquefied layer, which overlaid the liquefied layer was assumed as ten times of the rate of the liquefied layer. Figure 4 shows the analyzed result. Horizontal displacement of the wall was 1.2 m and the flow extended more than 100 m. Analyzed results agreed fairly well with the actual damage.

As the second type of ground flow, the damage occurred in Noshiro City during the 1983 Nihonkai-chubu earthquake, was selected. The flow was induced on a very gentle slope with the gradient of about 1 %. The maximum displacement on the ground surface was about 5 m. Analyzed average displacement was fairly coincided with the actual displacement.

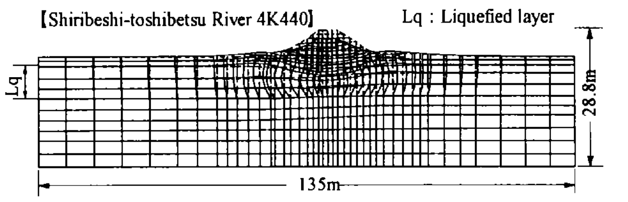
<u>Analyses for the settlement of river levees</u>

In the second step, the proposed method was applied to river levees (Yasuda et al., 2000a). The river levees selected for the analyses were seven damaged or non-damaged levees during the 1993 Hokkaido-nansei-oki earthquake and 1995 Hyogoken-nambu (Kobe) earthquake. The maximum settlement of the damaged bank was 2.6 m. Detail soil investigations were conducted at these sites. Reduced shear moduli of non-liquefied layers, which overlaid the liquefied layers, were assumed to be ten times larger than the reduced moduli of the liquefied layers same as the analysis for the ground flow. As the reduction rate of shear moduli in the levee body were not measured, the following four reduction rates were assumed in order to study the range of possible values:

Case A: The reduction rate for the levee body is same as the rate of the non-liquefied layer.
Case B: Shear modulus of the levee body does not decrease, namely G_l/G_N=1.0.
Case C: G_l/G_N=1/5 based on the tests on unsaturated soils.
Case D: G_l/G_N=1/10.

Figure 5 shows the analyzed results for the levee at Site No.9 of Shiribeshi-toshibetsu River under the condition of F_L=0.7 and Case D. The levee body settled and the liquefied soil under the levee body was pushed out, as shown the heave of the ground at toe in Fig. 5.

Figure 6 compare the analyzed settlements and actual settlements for seven river levees. Roughly speaking, the analyzed settlements increased with the actual settlements. Among the four cases, the analyzed settlements under the assumption of Case D agree fairly well with the actual settlements. Therfore, it can be said that the assumption of Case D for the reduction rate of shear modui in the levee body is appropriate among the four assumptions.



G_l/G_N=1/400(A_{s1}layer), G_l/G_N=1/600(A_{s2}layer), G_l/G_N=1/60(upper non-liquefiled layer), G_l/G_N=1/10(embankment), E=28N, K=const., F_L=0.7

Fig. 5. Deformation of the river levee at Site No.9 of Shiribeshi-toshibetsu River (Yasuda et al., 2000a)

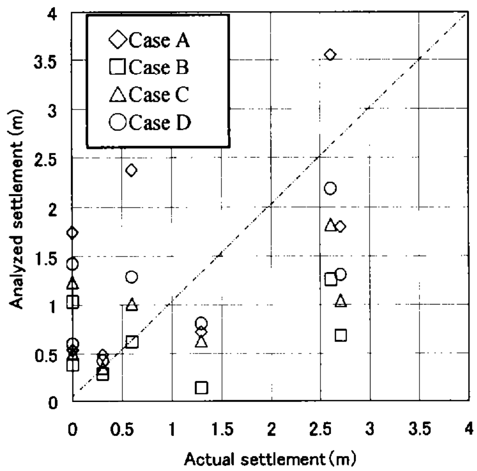


Fig. 6. Relationship between actual and analyzed settlements (Yasuda et al., 2000a).

ANALYSES OF SETTLEMENT FOR THE FOOTING OF POWER TRANSMISSION TOWER

In the third step, the proposed method was applied for the settlement of footings of transmission tower. Kawasaki et al. (1998) conducted many dynamic centrifuge tests to demonstrate the mechanism of settlement, and to find the parameters that influenced the settlement of an isolated footing for a power transmission tower. Figure 7 shows the model of the footing. Based on the test results, the following equation was derived to estimate the settlement of footings:

$$S = S_0 \times C1 \times C2 \times C3 \times C4 \times C5 \times C6 \times D1 \times D2 \times D3 \quad (3)$$

Where, S: Settlement, S_0: Settlement under particular standard conditions, $C1$: Factor for thickness of liquefiable layer, $C2$: Factor for thickness of non-liquefiable layer, $C3$: Factor for density of ground, $C4$: Factor for grain size, $C5$: Factor for amplitude of acceleration, $C6$: Factor for number of cycles of loading $D1$: Factor for width of footing, $D2$: Factor for load intensity, and $D3$: Factor for penetration depth.

Then the authors applied the proposed method "ALID" for the same models to demonstrate the adaptability of the analytical method. (Yasuda et al., 2000b). Figure 8 shows the results of

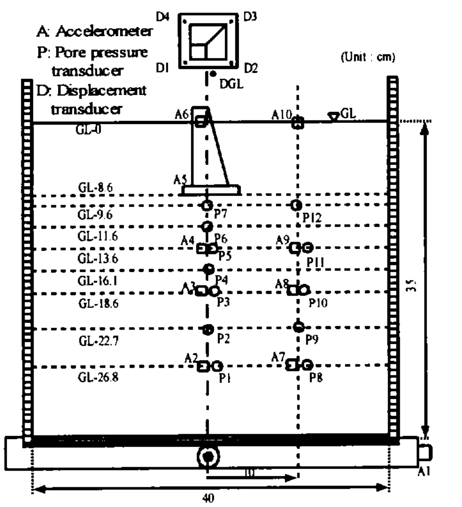


Fig. 7. Model footing and ground for centrifuge test (Kawasaki et al., 1998).

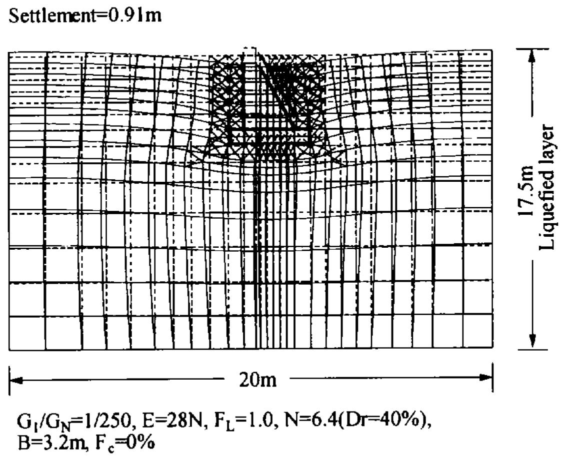


Fig. 8. Deformation of footing and ground in standard condition

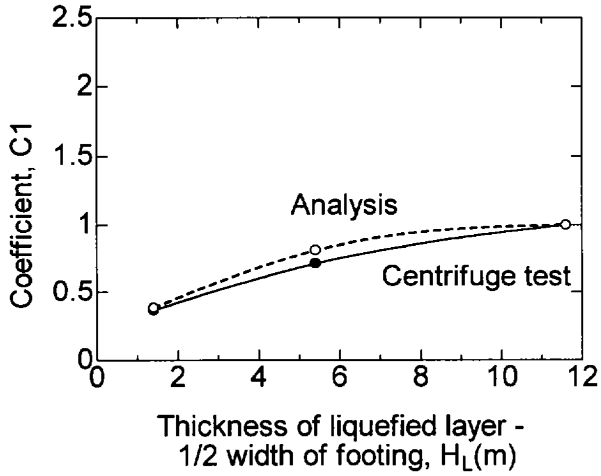


Fig. 9. Coefficient for thickness of liquefied layer

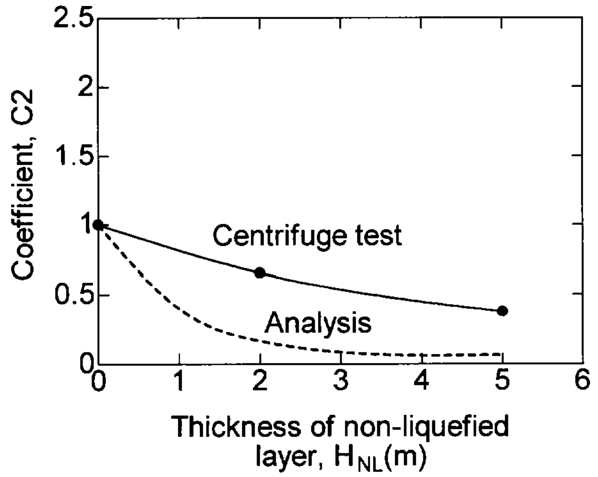


Fig. 10 Coefficient for thickness of non-liquefied layer

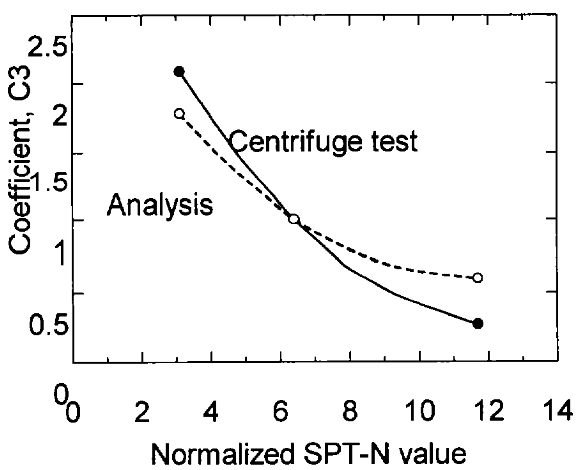


Fig. 11. Coefficient for SPT N-value

the analysis for the particular standard condition. The footing settled and the liquefied soil under the footing was pushed out. The pushed out soil moved upward. Analyzed settlement for the standard condition was 0.91 m. On the contrary tested settlement was 0.35 m. There are two possibilities to explain the difference: ① this analysis overestimates the settlement, and ② centrifuge test underestimates the settlement. The authors tried to find out the actual settlement of damaged towers during the 1964 Niigata earthquake to compare the centrifuge tests. However, it was difficult also to estimate the actual settlement. Therefore it is difficult to judge whether the absolute value of the analyzed settlement is appropriate or not.

In addition to the analysis under particular standard condition, several more analyses were carried out under the same conditions of centrifuge tests. Then the effects of the above mentioned factors were compared with the coefficient obtained by the tests. Figs.9 to 12 compare the tested and

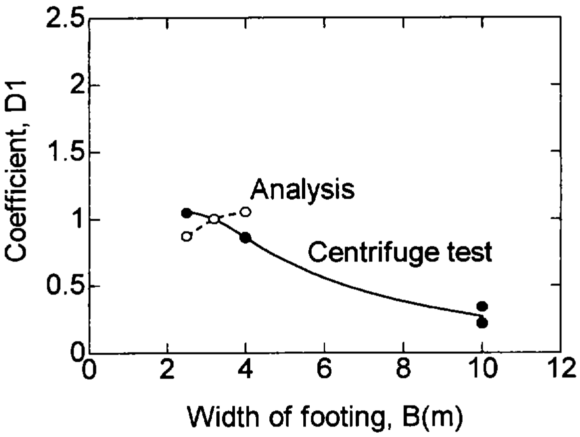


Fig. 12. Coefficient for width of a footing

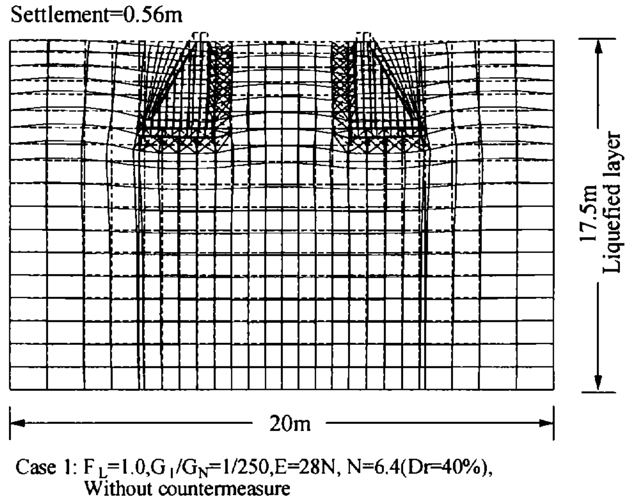


Fig. 13. Deformation of footings and ground in Case 1

analyzed coefficients for *C1*, *C2*, *C3* and *D1*. These figures imply the following tendencies:

1) The coefficient on thickness of liquefied layer, *C1*, and the coefficient on SPT *N*-value, *C3*, agreed well.

2) In the coefficient on thickness of non-liquefied layer, *C2*, the absolute value was different but the tendency was same.

3) Effect of width of a footing was different. According to the actual settlement of buildings and oil tanks during past earthquake, the settlement decreased with the width of foundation. This implies that the effect of width of footing can not be analyzed well by this method. One reason may be that the non-liquefied zone under the foundation is not considered in the analysis. Further study is necessary on the effect of width of foundation.

Centrifuge tests and analyses were carried out also for four-footings model with and without countermeasures. Five cases of tests and analyses were conducted:

Case 1: without countermeasure,

Case 2: connect top of four footings by a concrete slab,

Case 3: connect top of four footings by a concrete slab, and replace the ground under the slab by gravel of ρ_t=2.0 t/m^3 and SPT *N*-value=30,

Case 4: connect top of four footings by a concrete slab, and surround by sheet piles of E=2.5 × 10^6 N/cm^2, and

Case 5: connect top of four footings by a concrete slab, and improve the ground under the footings by densification for ρ_t=1.9 t/m^3 and SPT *N*-value=30.

Analyzed deformation of the footings and the ground for Cases 1 and 4 are shown in Figs.13 and 14, respectively (Uda et al., 2000). In Case 1, the liquefied soil just under footings moved to outer and inner directions and the footings settled and tilted as shown in Fig.13. Once top of footings are connected by a concrete slab and the soil under the slab is replaced by gravel, movement of liquefied soil to inner direction and tilting of footings were prevented, resulting to reduce the settlement of footings. Surrounding the footing by sheet piles prevented the movement of liquefied soil to outer direction as shown in Fig.14. Densification of the ground under the footings decreased the settlement.

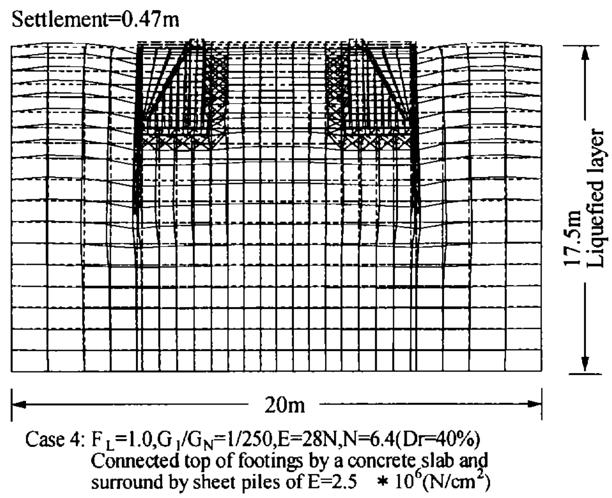


Fig. 14. Deformation of footings and ground in Case 4

Centrifuge tests also showed the effectiveness of these countermeasures. Figure 15 compares the reduction rate of settlement due to countermeasures. The order of effectiveness of countermeasures in tests and analyses was the same, though the rate was slightly different. This implies that the proposed is valid for the evaluation of effectiveness of countermeasures.

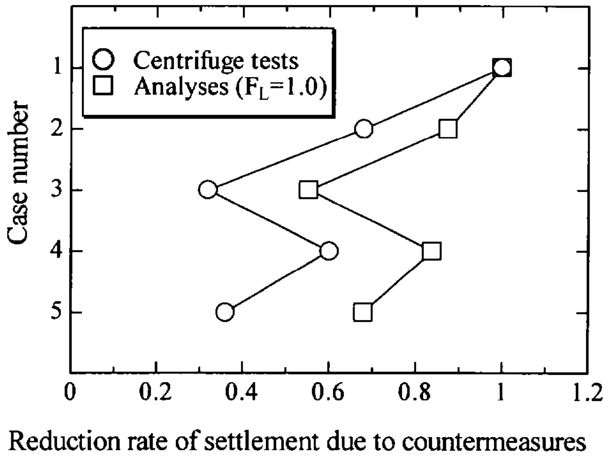


Fig. 15. Reduction rate of settlement due to countermeasures

CONCLUSIONS

Analyses of settlement of footings in liquefied ground were conducted by the proposed simple method named "ALID", and following conclusions were derived by comparing with centrifuge test results:

(1) Effect of thickness of liquefied layer and soil density on the settlement of footing can be evaluated by the simple method. However, effect of width of footings on the settlement cannot be evaluated well.

(2) Effect of countermeasures on the settlement of footing can be evaluated well by the simple method.

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