RESIDUAL DEFORMATION OF CAISSON, SHEET PILE AND GROUND BY SIMPLIFIED ANALYSIS

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SUMMARY

Residual deformation analyses were carried out to demonstrate the effect of input soil parameters on the liquefaction-induced ground flow behind sea walls. Two types of sea walls, a caisson type wall and a sheet pile type river revetment, which were damaged during the 1995 Hyogoken-nambu earthquake and the 1964 Niigata earthquake were selected for the analyses, respectively. Analyzed displacements of the ground behind the walls were roughly equal to the actual displacements if appropriate parameters are applied.

INTRODUCTION

The authors have developed a simplified method to evaluate the liquefaction-induced large deformation (Yasuda et al., 1999). The code of the developed method is named "ALID" (Analysis for Liquefaction-induced Deformation). In the method, the authors assumed that the residual deformation would occur in liquefied ground due to the reduction of shear modulus. The authors have tried to apply this method to several structures and grounds. In this paper, the authors applied the method to two types of sea walls that moved and the liquefied ground behind the walls flowed during the 1995 Hyogoken-nambu (Kobe) earthquake and the 1964 Niigata earthquake. In the companion paper (Higuchi, Yasuda et al. 2000), several shaking table tests also were carried out to demonstrate the mechanism of the ground flow behind the same walls.

2. OUTLINE OF THE PROPOSED METHOD

2.1 Concept of the proposed method

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Figure 1 shows the concept of the stress-strain curves which are used in the proposed method. Line *l* denotes a backbone curve at the beginning of the earthquake. Point A is supposed to be initial state of a soil element in the ground. When excess pore water pressure generates, material properties such as shear strength and elastic moduli change. Suppose that the backbone curve moves from *l* to *m* due to liquefaction, then strain should increases in order to hold the driving stress. Since driving stress, however, decreases according to the change of geometry, actual strain increment from state *l* to *m* is from A to C. Namely, strain increment caused by the change of material property is γ_C-γ_A. Ground flow stops when new material property comes to balance with a new driving stress, which is shown as point C in the figure. In the proposed method here, two paths : (1) A to C through B and (2) O to C through B, are assumed though the actual path is A to C because of simplification.

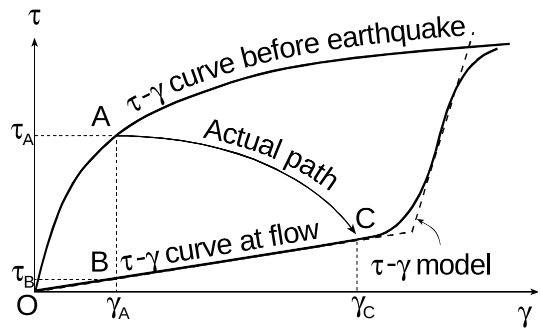


Fig.1 Concept of stress-strain curves

If the two paths are assumed, two procedures to analyze the liquefaction-induced deformation, (1)stress relaxation method and (2) self weight method, are available. Both methods have merits and demerits. In the following analyses, the self weight method is employed with linear stress-strain relationships.

2.2 Modeling of the ground

In the modeling of the liquefied soil layers, safety factor against liquefaction, F_L of liquefiable layer is evaluated at first by some method. Then the rate of reduction of shear modulus due to liquefaction is estimated by laboratory tests or a proposes relationship shown later. Shear modulus of the upper unliquefiable layers had better to be reduced because frequently cracks are induced in the unliquefiable layers during the liquefaction-induced flow. Shear modulus ten times larger than that in the liquefiable layer was applied in the following analyses based on several back analyses during past earthquakes. Subsidence of the ground surface occurs by two mechanism : geometrical change due to the flow and densification due to dissipation of excess pore water pressure. The subsidence by the first mechanism is evaluated automatically in this analysis. Some additional analyses are necessary to evaluate the subsidence due to the second mechanism.

2.3 Flow chart of the analysis

Figure 2 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 condition of no volume change.
(3) The difference in the deformation measured by the two analyses is supposed to equal the residual ground deformation.

2.4 Determination of input soil data

Reduction of shear modulus due to liquefaction was evaluated based on cyclic torsional shear tests on Toyoura sand and Masa which was take in Kobe. Reduction of shear modulus thus evaluated for liquefied Toyoura sand and Masa were about 1/500 and 1/300, respectively (Yasuda et al., 1999). Analyses were carried out also under different values of reduction rate.

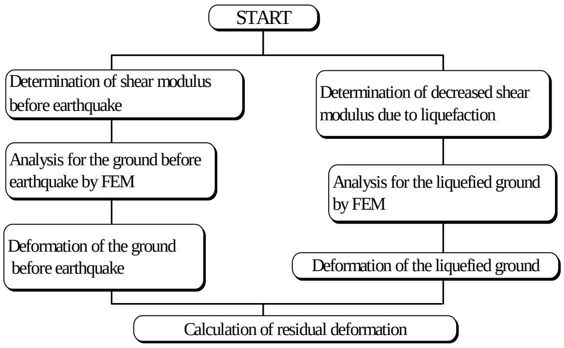


Fig.2 Flow chart of analysis

In addition the data above mentioned, the authors have conducted tests for many soils and summarized the reduction rate with fines content and severity of liquefaction as shown in Fig.3 (Yasuda. et al., 1999)

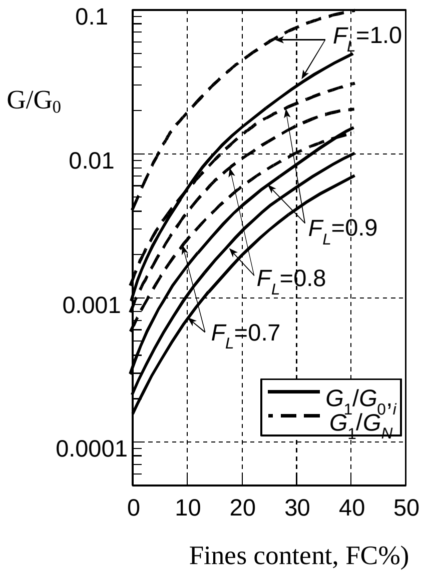


Fig.3 Summary of test results

3. Models for the analyses

Two types of sea walls were selected for the analyses:

(1) A caisson type quay wall at Uozaki-hama in Kobe City: The wall moved towards sea and the ground behind the wall flowed during the 1995 Kobe earthquake. Horizontal displacement of the wall was 2.0m and the flow extended to more than 100m form the wall. Figure 4 shows the model for Uozaki-hama.

(2) A sheet pile type river revetment at Showa Bridge in Niigata City: The revetment tilted towards the river and the ground behind the wall flowed during the 1964 Niigata earthquake. Horizontal displacement near the wall was about four meters and the flow extended to about 200m. Figure 5 shows the model for Showa Bridge.

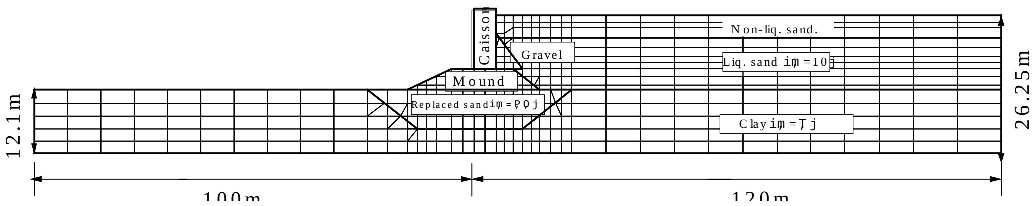


Fig.4 Model for Uozaki-hama

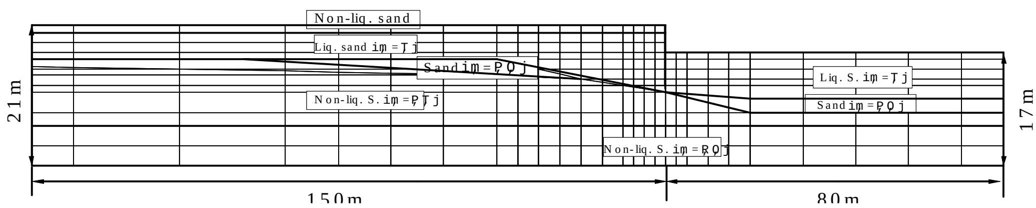


Fig. 5 Model for Showa Bridge

In the analyses, shear modulus before the earthquake were estimated by SPT N-values by the formula $E=28N$, where E is Yong's modulus in kgf/cm^2 and N is SPT N-value. Poisson's ratio before the earthquake was assumed as 0.33. Safety factor against liquefaction, F_L was evaluated by the method introduced in the Specification for Highway Bridges (1996). In the analyses for Uozaki-hama, shear modulus rate (reduction rate) with liquefaction G_1/G_N for reclaimed sand layer under ground water table,

reclaimed sand layer upper than ground water table and replaced sand, were assumed as follows:
a) reclaimed sad layer lower than ground water table, which liquefied during the earthquake: 1/100 and 1/300
b) reclaimed sad layer upper than ground water table, which did not liquefy during the earthquake: ten times of the rate of the liquefied layer, and
c) replaced sand, which did not liquefy but large shear strain was induced due to large initial shear stress: twice and ten times of the rate of the liquefied layer.
The rate of 1/300 for the liquefied layer was decided by cyclic torsional tests as mentioned before. Conditions for the analyses are shown in Table 1. Joint elements were used between the wall and the gravel, between the wall and the mound, and between the mound and replaced sand.

In the analyses for Showa Bridge, shear modulus rate were assumed as follows:
a) loose sand layer with 5 of SPT *N*-value lower than ground water table, which liquefied during the earthquake: 1/100, 1/500 and 1/1000
b) loose sand layer upper than ground water table, which did not liquefy during the earthquake: ten times of the rate of the liquefied layer.
The rate of 1/500 for the liquefied layer was selected because the test results for Toyoura sand, which is the similar sand in Niigata, was 1/500 as mentioned before. Table 2 shows the condition for the analyses.

Table 1 Soil condition for Uozaki-hama

Case No.	Liq. Layer	Non-liq. Layer	Replaced s.
U-1	1/100	1/10.	1/100
U-2	1/100	1/10.	1/50
U-3	1/100	1/10.	1/10.
U-4	1/300	1/30.	1/300
U-5	1/300	1/30.	1/150
U-6	1/300	1/30.	1/30.

Table 2 Soil condition for Showa Bridge

Case No.	Liq. Layer	Non-liq. Layer
S-1	1/100	1/10.
S-2	1/500	1/50
S-3	1/1000	1/100

Joint elements were used between the wall and the ground.

4. Analyzed results

4.1 Caisson type wall

Figure 6 shows deformation of the caisson type wall and the ground for Case No.U-1 and U-6. The wall moved and the ground flowed towards the sea as same as the damage at Uozaki-hama during the 1995 Kobe earthquake. However, the caisson did not tilt towards the sea because the effect of inertia force cannot be considered in the analysis. Figure 7 compares the distribution of horizontal displacement on the ground surface in different soil condition. Flowed area and displacement at a distance from the wall increased with the decrease of shear modulus of the liquefied layer and the replaced sand. Black circles show the measured displacement at Uozaki-hama. Analyzed displacement in Case No.U-6 agreed with the actual displacement. Therefore, it can be said that the cyclic torsional shear test results can be applied directly for the analysis, and the appropriate shear modulus rate of the upper non-liquefiable layer and the replaced sand was about ten times of the rate of the liquefied layer in this model.

4.2 Sheet pile type wall

Figure 8 shows deformation of the sheet pile type wall and the ground for case No.S-1 and S-2. The wall bent and tilted, and the ground behind the wall flowed towards the river as same as the damage at Showa Bridge during the 1964 Niigata earthquake. Figure 9 compares the distribution of horizontal displacement on the ground surface in different conditions. Flowed area and displacement at a distance from the wall increased with the decrease of shear modulus of the liquefied and non-liquefied layers.

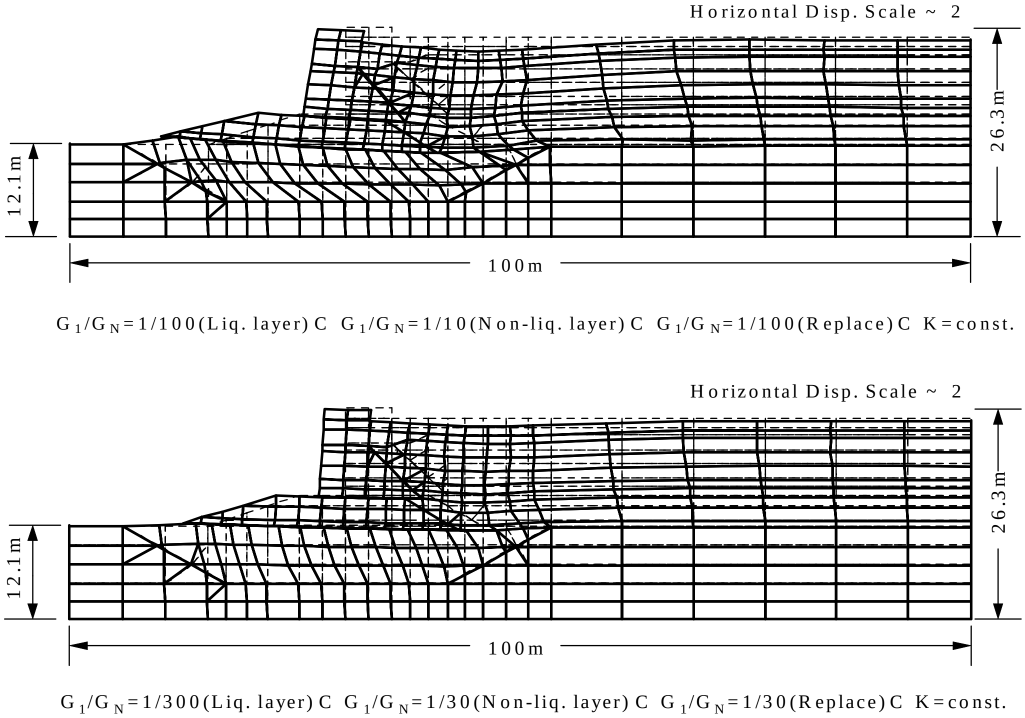


Fig.6 Analyzed results for the model at Uozaki-hama

Black circles show the measured displacement behind Showa Bridge. Analyzed displacement in case No.S-2, in which torsional shear test results was applied, agreed fairly with the actual displacement at the distance of around 30m. However, the displacement at far distance was not coincided.

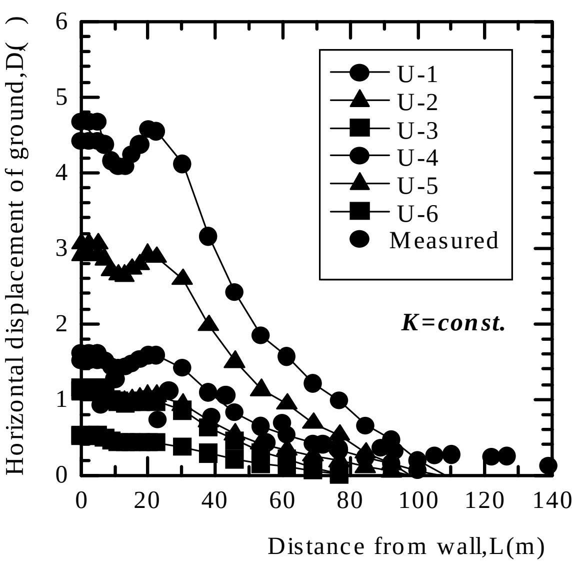


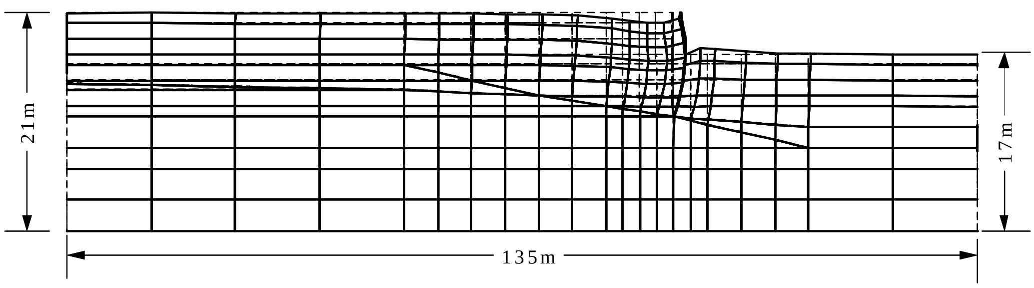
Fig.7 Distribution of displacement on the ground surface for the model at Uozaki-hama

5. CONCLUSIONS

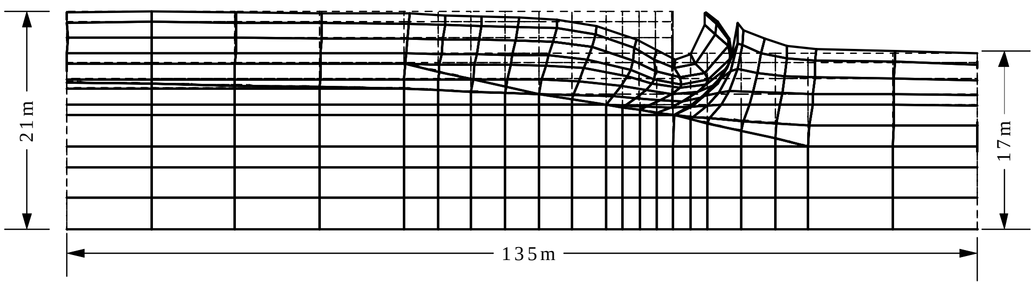
Several residual deformation analyses were carried out to demonstrate the effect of input soil parameter on the liquefaction-induced ground flow behind two types of sea walls, and the following conclusions were derived:

(1) A simplified analytical method by considering the reduction of shear modulus due to liquefaction could simulate the displacement of the ground behind a sea wall. .

(2) Analyzed deformation is sensitive to the shear modulus rate of liquefied and non-liquefied soil layers.



G_1/G_N=1/100(Liquefied layer) , G_1/G_N=1/10(Non-liquefied layer upper water table.) , K=const.



G_1/G_N=1/500(liquefied layer) , G_1/G_N=1/50(Non-liquefied layer upper water table) , K=const.

Fig.8 Analyzed results for the model at Show Bridge

ACKNOWREDGEMENT

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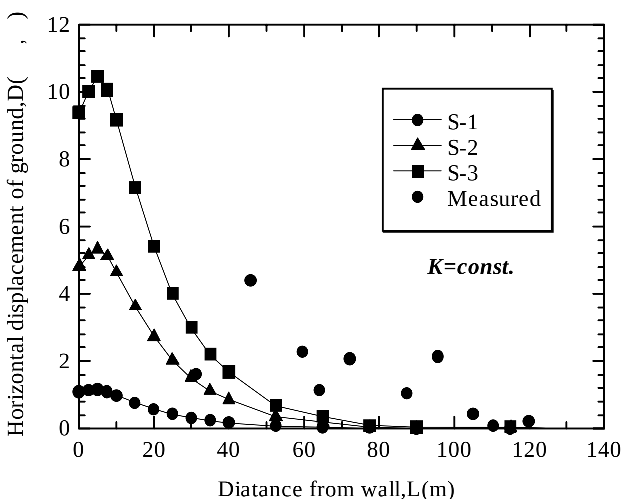

Fig.9 Distribution of displacement of the ground surface for the model at Showa Bridge

RFERENCES

1. Higuchi, K., Yasuda, S. et al. (2000), "Shaking table tests on the mechanism of liquefaction-induced ground flow behind quay walls," *12th WCEE*.
2. The Japan Road Association (1996), "Specification for Highway Bridges."
3. Yasuda, S., Yoshida, N., Adachi, K., Kiku, H. and Gose, S. (1999), "A simplified analysis of liquefaction-induced residual deformation," *Proc. of the 2nd International Conference on Earthquake Geotechnical Engineering.*