RESIDUAL DEFORMATION ANALYSIS OF SHEET PILE QUAY WALL AND BACKFILL GROUND AT SHOWA-OHASHI SITE BY SIMPLIFIED METHOD

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SUMMARY

A method is presented to compute liquefaction-induced residual deformation of quay wall and its backfill ground. This method is composed of equivalent linear dynamic response analysis and linear static analysis. The shear modulus in the static analysis is computed by multiplying three reduction factors into undamaged shear modulus. They are reduction factors by considering the effect of gravity, earthquake, and liquefaction; the first factor comes from the gravity-caused strain, the second factor from the nonlinear stress-strain relationship during earthquake, and the third factor due to liquefaction. These factors are computed according to stress, strain, etc. of ground by linear static or equivalent linear dynamic analyses. The Showa-Ohashi site where significant lateral movement of quay wall was observed during the 1964 Niigata earthquake is analyzed. The sheet pile moved 3.4m to 7.4 m towards the sea, which agrees with the observed damage.

INTRODUCTION

The Technical Committee on Flow Deformation and Permanent Displacement of Ground and Earth Structures during Earthquakes, Japanese Geotechnical Society, started its activities for three years from April, 1995. The committee took charge of simultaneous model tests and analyses on waterfront structure during earthquakes among its activities [Kanatani and Yoshida, 1998] . As objects, the committee selected the caisson type quay wall of Kobe port which was damaged by 1995 Hyogo-ken Nanbu earthquake and sheet pile type quay wall on the left bank of Shinano River (Showa-Ohashi site) which suffered from damages by 1964 Niigata earthquake.

We used a simplified method (herein referred to as FLUSH-L method) combining programs of total stress method for analyzing damages on the sheet pile type quay wall at Showa-Ohashi site. The paper describes its method and analyzed results.

ON FLUSH-L METHOD

Outline of FLUSH-L method

The FLUSH-L method was proposed in 1996 as shear modulus estimating method for liquefied ground based on F_L-value and preliminary surveying method for damage degree resorting to it [Shibata et al., 1997b]. Its procedure can be outlined as a) compute F_L-value of each element of ground using 2-D equivalent linear dynamic

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analysis program FLUSH [Lysmer et al.,1975] based on finite element method, b) estimate the shear modulus of ground at a liquefaction based on F_L-value, c) using that ground shear modulus, analyze a linear earthquake response for obtaining a section force produced on sheet pile, etc. in liquefied ground, d) likewise, using a decreased shear modulus of ground, do self-weight analysis for obtaining a residual deformation produced by gravity acting on liquefied ground and further the corresponding section force of sheet pile, etc., and e) add the maximum section force in the duration time by seismic force and section force by gravity for computing a maximum section force.

For estimating the shear modulus G of ground based on F_L-value, let us refer to G/G_0-F_L curve determined separately. Here, G_0 is the initial shear modulus computed from for example shear wave velocity on PS logging, and G the shear modulus of liquefied ground. Since the decreasing ratio of shear modulus G/G_0 at a liquefaction of real ground is unknown, it was substituted by a value analyzed by FLIP program [Iai et al., 1992] which considers a liquefaction phenomenon based on the effective stress method. We examined the relationship between F_L value of each element of ground by FLUSH and decreasing ratio of shear modulus (G/G_0) at the same position by FLIP on a cross section of quay wall A (- 12 m) of steel-pipe-type section's sheet pile of Osaka Port. It showed a certain trend. A similar trend was recognized on cross sections of other quay walls of Osaka Port. For analyses heretofore by FLUSH-L method for Osaka Port, therefore, we used G/G_0-F_L curve obtained by study of quay wall A mainly. We retained 1/10 for any decreasing ratio of shear modulus of non-liquefied ground. Fig. 1 shows the structure of quay wall A. As replacement sand and filling soil, well graded sandy soil was used.

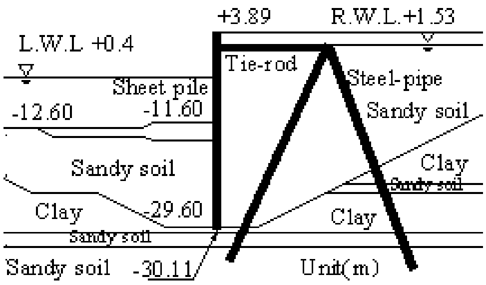

Figure 1: Cross sectional view of quay wall A of Osaka port

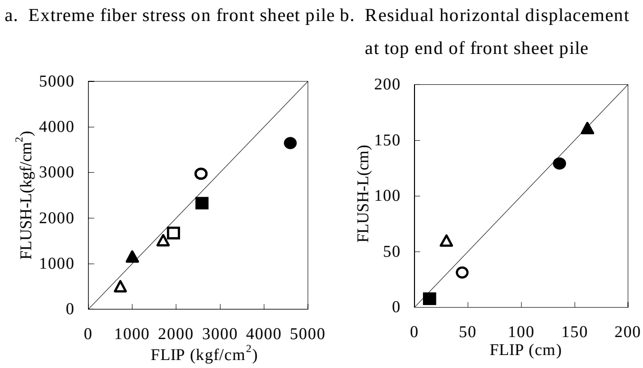

Figure 2: Comparison of results by FLUSH-L method and FLIP for quay wall A (■□), coastal levee B (●○) and revetment C (▲△) at Osaka Port ■●▲: Current cross section. □○△: Improved cross section. Seismic wave for input: Hachinohe wave (max. acceleration 250 gal).

Fig. 2 compares results obtained by FLUSH-L method and FLIP for cross sections of current status and those which have been improved to cope with a liquefaction of three kinds of sheet pile type revetments (quay wall A, coastal levee B, revetment C) of Osaka Port [Shibata et al., 1997a]. For analyses, we used as a seismic wave for input a "Hachinohe wave" (observed at 1968 Tokachi-oki earthquake) with its maximum acceleration adjusted to 250 gal. In these diagrams, values estimated by each of two ways show a fairly good agreement.

Modification of FLUSH-L method

The estimation by the method described above is not accurate enough in case of strong earthquakes or if a stress is put on the estimation of shear modulus of non-liquefied layer. So, the method was modified. I.e., it was improved so that we can estimate the decrease of shear modulus of ground induced, respectively, by a) gravity (failure by active earth pressure, etc.), b) seismic force and c) liquefaction because the accuracy might be compromised if the decrease of shear modulus of ground were attributed to FL-value only while it may also be caused by other than the liquefaction [Ozutsumi et al., 1997].

For example, the shear modulus of ground of the revetment area is decreasing by a gravitational action. Let us express by G_1 the decreased shear modulus of soil caused by gravity. If an earthquake has occurred at this status, the shear modulus of ground will further drop dependent on the strain amplitude even if it does not liquefy. Let us express by G_2 a decreased shear modulus. If it liquefies, the shear modulus will further decrease. Let us express it by G. If the decreasing ratios (G_1/G_0, G_2/G_1, G/G_2) of shear modulus caused by such factors can be estimated, shear modulus G of soil of liquefied ground can be computed by:

$$G = G_0 \times \underbrace{(G_1/G_0)}_{\substack{\text{decreasing ratio}\\ \text{of shear modulus}\\ \text{by gravity}}} \times \underbrace{(G_2/G_1)}_{\substack{\text{decreasing ratio of}\\ \text{shear modulus by}\\ \text{seismic force}}} \times \underbrace{(G/G_2)}_{\substack{\text{decreasing ratio of}\\ \text{shear modulus by}\\ \text{liquefaction}}} \quad (1)$$

G can easily be estimated if these decreasing ratios can be determined from analyzed results by linear or equivalent linear analysis program. FLUSH-L method estimates each decreasing ratio using parameters given in Table 1. It would be best if we could know the relationship between these parameters and decreasing ratios of shear modulus on real ground. We will use analyzed values from FLIP program instead.

Table 1: Factors of decreasing of ground shear modulus and parameters for estimating the decreasing ratio

Shear modulus decrease induced by:	Decreasing ratio	Parameter for estimation
(1) Gravity	G_1/G_0	Shear stress ratio τ_{max}/σ_{m0}' resulting from linear self-weight analysis
(2) Seismic force	G_2/G_1	Decreasing ratio of shear modulus G_{new}/G_0 resulting from equivalent linear dynamic analysis
(3) Liquefaction	G/G_2	Liquefaction safety factor F_L value

Fig. 3 is a diagram for showing a typical relationship between shear stress ratio (τ_{max}/σ_{m0}') of linear self-weight analysis and FLIP's self-weight analysis. So long as the shear stress ratio remains small enough, FLIP responds as per linear elasticity and, therefore, both of them show a good agreement. Note that, for FLIP, the high limit of shear stress ratio is $\sin\phi_f'$ (ϕ_f': angle of internal friction) but the linear analysis has no high limit. Use of the relationship illustrated here allows you to guess the shear stress ratio by FLIP from linear self-weight analysis. Once the shear stress ratio by FLIP is known, you can estimate the gravity-induced decreasing ratio of shear modulus (G_1/G_0) resorting to:

$$G_1/G_0 = 1 - (\tau_{max}/\sigma_{m0}')/\sin\phi_f' \quad (2)$$

which is based on the nature of hyperbolic model characterizing the constitutive equation of FLIP.

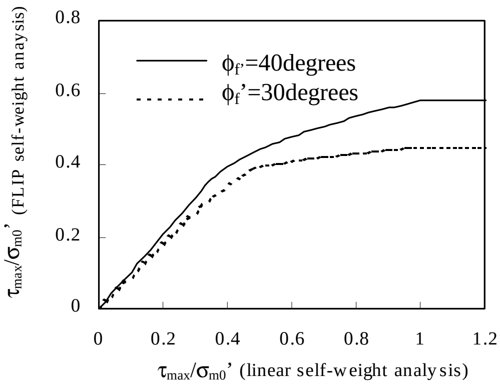

Figure 3: Shear stress ratio resulting from self- weight analysis by FLIP versus shear stress ratio resulting from linear self-weight analysis

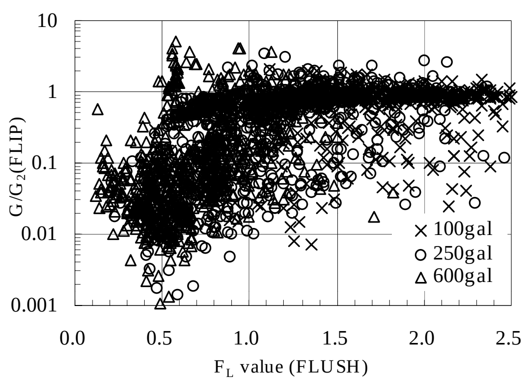

Figure 4: Quay wall A at Osaka Port: G/G_2 versus F_L

As a seismic force-induced decreasing ratio of shear modulus (G_2/G_1), we used a value of decreasing ratio (G_{new}/G_0) of shear modulus resulting from an equivalent linear dynamic analysis. As for liquefaction-induced decreasing factor of shear modulus (G/G_2), we obtained a decreasing ratio of shear modulus by liquefaction (G/G_2) of each soil element from analyzed results of FLIP for quay wall A of Osaka Port and associated it with F_L value obtained by FLUSH. This relationship is shown in Fig. 4. The plotted points in the figure correspond to each soil element of the analyzed model. Both G and G_2 in G/G_2 on the ordinate of the figure are shear moduli of ground after shaking by FLIP. G results from an analysis where a rise of excess porewater pressure is taken into account, and G_2 from another analysis where it is not. Therefore, G/G_2 can be considered as a liquefaction-induced decreasing ratio of shear modulus. Referring to this relationship, we determined G/G_2-F_L curve as shown by the thick full line in Fig. 5.

Let us see the texture of replacement sand and filling soil on quay wall A of Osaka Port. The fine fraction content ratio is 10 to 30%, and the gravel content 30 to 60%. The aspect of ground of Showa-Ohashi site, on the other hand, which is natural sedimentary ground mainly composed of sand is different from quay wall A. Therefore, it is natural to guess that the liquefaction characteristics are different between the two sites. So, we analyzed Showa-Ohashi site according to FLIP also [Sawada et al., 2000] and created a graph the same as in Fig. 4. The result is shown in Fig. 6. When we compare the two graphs, the decreasing ratio of shear modulus G/G_2 for Showa-Ohashi site reduces abruptly as soon as F_L value is below 1.0. Based on this relationship, we prepared G/G_2-F_L curve shown by a thin full line in Fig. 5 and used it for analyses of this time.

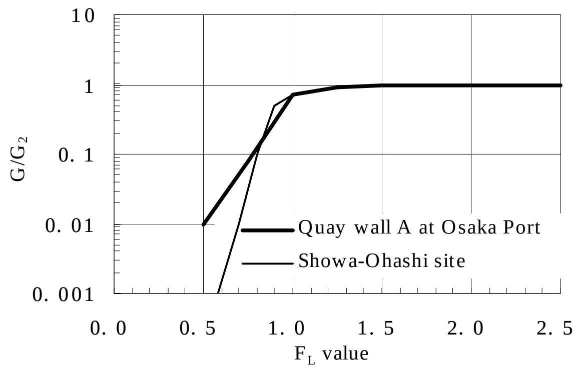

Figure 5: Curves of G/G_2 versus F_L

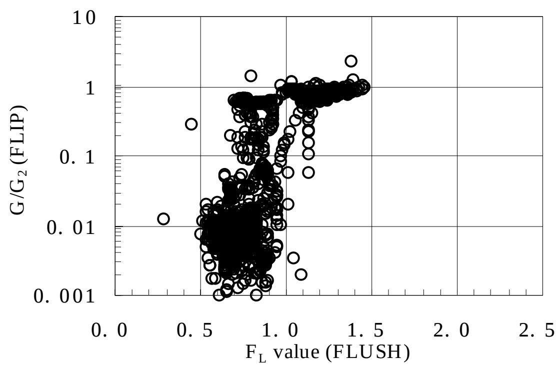

Figure 6: Showa-Ohashi site: G/G_2 versus F_L

Fig. 7 shows the calculation procedure of the FLUSH-L method. Note that, unlike the effective stress method which is generally adopted, the FLUSH-L method is characterized in a) that you need not set complicated parameters for expressing the liquefaction characteristics, b) that the problem of no convergence of calculation hardly occurs, and c) that a short calculation time suffices.

start

a. Calculate initial effective confining pressure (σ_{m0}') and shear stress ratio (τ_{max}/σ_{m0}') (linear self-weight analysis)

b. Estimate the gravity-induced decreasing ratio of shear modulus G_1/G_0 (parameter: shear stress ratio)

c. Compute shear stress of soil at earthquake (FLUSH equivalent linear analysis)

d. Estimate the seismic force-induced decreasing ratio of shear modulus G_2/G_1 (parameter: G_{new}/G_0 from equivalent linear result)

e. Calculate liquefaction safety factor F_L (use Ishihara method)

f. Estimate the liquefaction-induced decreasing ratio of shear modulus G/G_2 (parameter: liquefaction safety factor F_L value)

g. Estimate the ground shear modulus combining 3 factors $G= G_0 \times (G_1/G_0)\times(G_2/G_1)\times(G/G_2)$

h. Compute maximum section force of member by seismic force at liquefaction (FLUSH linear analysis)

i. Calculate residual displacement and member section force (after shaking) at liquefaction (linear self-weight analysis)

j. Calculate maximum member section force (add section forces in h and i)

end

Figure 7: Calculating procedure for FLUSH-L method

Analyses of Showa-Ohashi site

At the time of 1964 Niigata earthquake, Showa-Ohashi bridge (in Niigata City, Japan) over Shinano River fell down. The ground of the sheet pile type revetment area on the left bank of that site moved by about five meters toward the center of the river [Hamada et al., 1986]. We performed the deformation analysis by the FLUSH-L method for the site. The analyzed model and analyzing conditions we used conform to the directions of the committee.

Soil layer division and finite element mesh

Fig. 8 shows the scope of analyzed model and soil layer divisions. According to the conditions of simultaneous analyses, a soil layer having SPT N-value of 50 which appears at GL - 21 m behind the quay wall was retained as a basement layer and its top level was retained as a bottom boundary of the analyzed model. In front of the quay wall, the top level of this soil layer is shallower and, accordingly, the bottom boundary line is provided with a gradient. Considering the difference in confining pressure, each soil layer is divided into several parts and different material numbers are allocated to them.

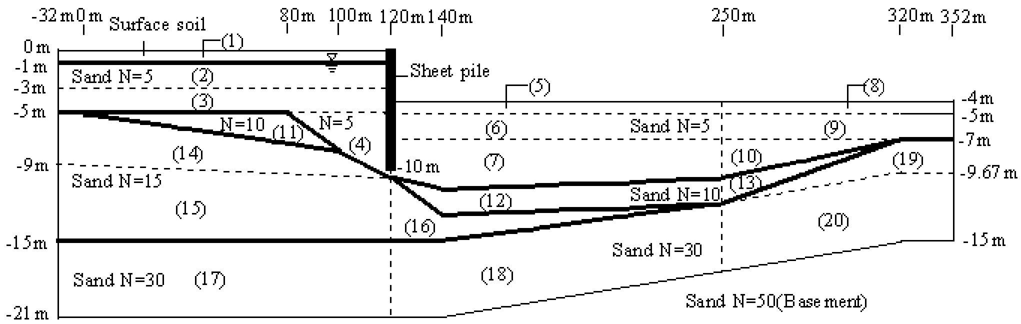

Figure 8: Soil Layer divisions (ordinate is magnified 10-fold with respect to abscissa). (1) to (20) are material numbers.

In analyzing the earthquake response by FLUSH, the viscous boundary was assigned to the bottom, and the horizontal roller to the side. The lowermost level of sheet pile is GL - 10 m under the condition of simultaneous analyses, which means that it would share a nodal point with element of soil layer having SPT N-value of 15. On the present analyzed model, however, the level was restricted down to GL - 9 m so that the nodal point will not be shared by element of soil layer having SPT N-value of 15. Fig. 9 shows finite element mesh and soil layer divisions in the sheet pile revetment area.

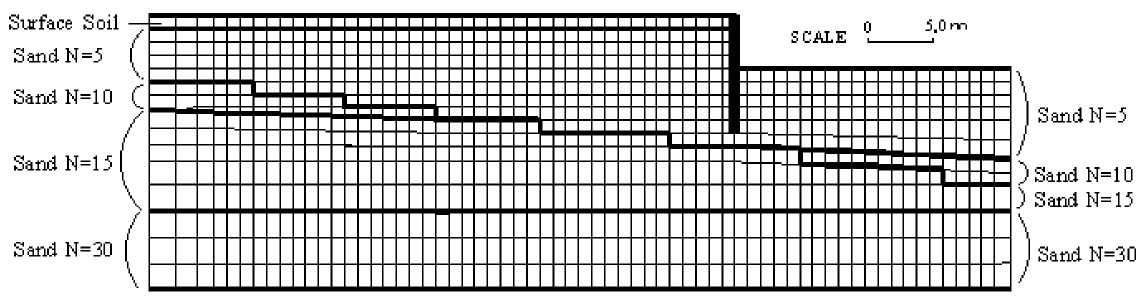

Figure 9: Finite element mesh divisions of major section.

Material properties

Initial shear modulus

Shear wave velocity Vs of each soil layer is given as shown in Table 2 as a condition of simultaneous analyses. It is assumed that Vs is the value at the center of each soil layer referring to the layer system at 40 m behind the revetment. The corresponding initial shear modulus which can readily be obtained from Vs and density is retained as "reference initial shear modulus G_{ma}". We obtained effective overburden pressure σ_v' at the center of each soil layer and, assuming that coefficient of earth pressure at rest k_0 = 0.5, computed "reference mean effective stress σ_{ma}'". These values are also given in Table 2. Applying a relation that the initial shear modulus is proportional to 0.5th power of mean effective confining pressure to G_{ma} and σ_{ma}', we will have initial shear modulus at any mean effective confining pressure σ_m' (expression (3)). Using this relation, we computed the initial shear modulus for each material number.

$$G_0 = G_{ma} \times (\sigma_m' / \sigma_{ma}')^{0.5} \tag{3}$$

Table 2: Material properties of soil

Soil layer	Shear wave velocity	Reference mean effective confining pressure	Reference initial shear modulus	Wet unit weight	Max. damping coeff.	Angle of internal friction	Liquefaction resistance ratio
	Vs m/s	σ_{ma}' tf/m^2	G_{ma} tf/m^2	ρ_t tf/m^3	h_{max}	ϕ_f' degree	$R1_{20}$
Surface layer	110	0.68	2222	1.8	0.24	30.0	—
N=5	140	2.70	3800	1.9	0.24	38.0	0.22
N=10	180	4.89	6282	1.9	0.24	41.0	0.23
N=15	200	8.55	8163	2.0	0.24	41.0	0.30
N=30	240	13.84	12343	2.1	0.24	42.0	—

Nonlinear characteristics of soil

By the FLUSH-L method, an equivalent linear dynamic analysis is made by FLUSH. As its strain dependence curve, we used curves (expressions (4) to (6)) normally employed by the FLUSH-L method. The curves are based on a hyperbolic model. Since, under conditions of simultaneous analyses, it is a convention to use expressions of Yasuda and Yamaguchi [Yasuda and Yamaguchi, 1996] as strain dependence curves, these expressions were also employed for analyses.

$$G / G_0 = 1/(1+x) \tag{4}$$

$$h / h_{max} = x/(1+x) \tag{5}$$

$$x = G_0 \gamma /(\sigma_{m0}' \sin\phi_f'), \quad h_{max} = 0.24 \tag{6}$$

Angle of internal friction

At a point 40 m behind the quay wall, we obtained effective overburden pressure σ_v' at the center of each soil layer. From it and SPT N-value of relevant soil layer, we computed relative density Dr resorting to Meyerhof's expression (expression (7)). Then we computed the angle of internal friction referring to the relation [Zen et al., 1990] between relative density and angle of internal friction obtained by a test using sand of Akita Outer Port. Note that, for surface soil only, the values are slightly reduced with a view to reducing its deformation confining effect. These values are also given in Table 2.

$$Dr(\%) = 21 \times \sqrt{N/(\sigma_v'+0.7)} \quad (\sigma_v' \ldots \text{kgf/cm}^2) \tag{7}$$

Liquefaction characteristics

Under the conditions of simultaneous analyses, the liquefaction resistance ratio (τ_1/σ_{m0}') at 5th and 20th cycles is designated. Since the FLUSH-L method uses Ishihara method [Ishihara, 1976] for computing the liquefaction safety factor (F_L-value), only the value at the 20th cycle was referred to (see Table 2).

Material properties of sheet pile wall

Table 3 gives specifications per meter of sheet pile wall width. The sheet pile wall was modeled in terms of linear beam element.

Table 3: Material properties of sheet pile wall per meter of wall width

Cross sectional area	Moment of inertia of area	Section modules
153.0 cm^2/m	8740 cm^4/m	874 cm^3/m

Analytical conditions

As a seismic wave for input, we employed in accordance with simultaneous analyses a wave somewhat modified of NS component observed by Akita Prefectural Government at 1964 Niigata earthquake. We adopted 120 gal (2E) of maximum acceleration of seismic wave for input and inputted it to the viscous boundary of the analyzed model bottom. Sample interval Δt in the time domain is 0.01185 second which is modified wave recording interval and the number of sample points is 2048, supplementing trailing 0. The high limit of frequency involved is 10 Hz. Fig. 10 shows the time history of seismic wave for input.

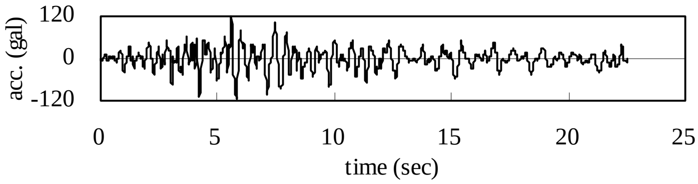

Figure 10: Seismic wave for input (modified wave from one observed by Akita Prefectural Government at 1964 Niigata earthquake)

Analyzed results

Circumstances of liquefaction

Fig. 11 shows the distribution of F_L values in the major section. According to this figure, liquefied layers are almost limited to soil layers where SPT N-value is 5. F_L value of this soil layer is 1.0 or greater immediately behind the top of sheet pile wall and smaller than 1.0 in other parts and is 0.6 to 0.8 in fair part of the revetment area particularly. As will be described later, the existence of parts where F_L value is 1.0 or greater affects the bending moment of sheet pile wall. The current FLUSH-L method computes F_L value based on results of equivalent linear dynamic analysis to be performed first. With such a method, the processes of spreading liquefaction by re-distributing the stress could not be expressed. In the first equivalent linear analysis, the shear modulus before liquefaction is kept by the ground. Supported by that ground, the sheet pile wall suppresses a shear deformation of ground located behind. That may be the reason why F_L value was above 1.0. If a seismic response analysis has been analyzed at a status where the ground is liquefied with its shear modulus decreased, the ground immediately behind the sheet pile wall may probably liquefy.

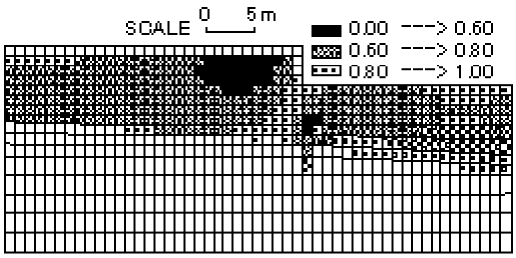

Figure 11: F_L value distribution (major section)

Figure 12: Residual deformation (major section)

F_L value according to expressions of Yasuda and Yamaguchi shows almost the same distribution although it is estimated smaller in the surface layer of riverbed in front of quay wall. Then the shear modulus decreased, thereby reducing the passive resistance and, as stated later, increasing the lateral movement. The difference in F_L value in the surface layer of riverbed may be attributable to difference in confining pressure dependency of nonlinear characteristics of soil.

Aspect of deformation

Fig. 12 shows the residual displacement of major part. The lateral movement is 3.4 m, and the settlement 2.1 m at the surface immediately behind the sheet pile wall. According to expressions of Yasuda and Yamaguchi, they were 7.4 m and 6.2 m, respectively. In Fig. 12, the residual deformation is limited to a soil layer corresponding to N = 5 in the vicinity of quay wall. In Fig. 13, the horizontal residual displacement distribution at the soil surface and riverbed is indicated by full line, and the vertical displacement by dotted line. These figures indicate that the extent where the ground is deformed on account of revetment offset is 20 and 10 m behind and before the revetment, respectively.

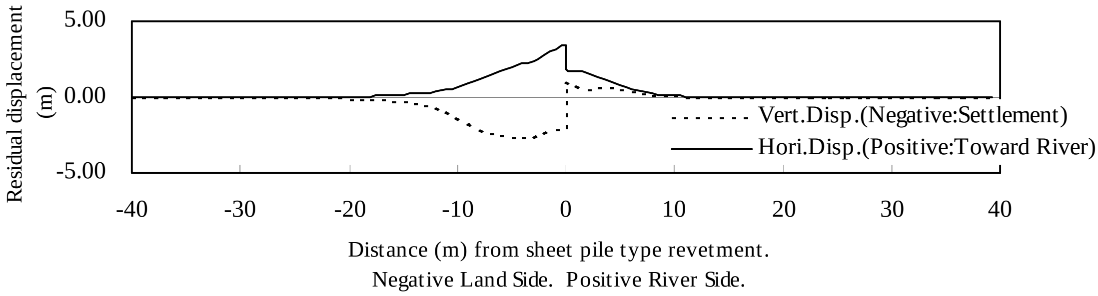

Figure 13: Distribution of horizontal and vertical residual displacements on soil surface and riverbed

Other response values

On the surface soil immediately behind the sheet pile wall, the maximum horizontal acceleration was 276 gal, and the maximum vertical acceleration 205 gal. The maximum bending moment per meter of sheet pile wall width is 13 tfm/m which is rather small. A large moment seems to have not occurred because the ground immediately behind the sheet pile wall did not locally liquefy, keeping the shear modulus to a certain degree as already stated.

CONCLUSIONS

Using the FLUSH-L method, the seismic response was analyzed taking into account the liquefaction of Showa-Ohashi site. With the FLUSH-L method, G/G_2-F_L curve is used for estimating the liquefaction-induced decreasing ratio of shear modulus. Because there were no curves that were applicable to natural sedimentary sandy ground, analysis by FLIP was also made and, referring to their results, G/G_2-F_L curve was determined. The horizontal residual displacement of revetment resulting from analyses with this curve agreed with the really measured value. The adequacy of extent where lateral movement occurs to be studied hereafter.

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