COMPARATIVE STUDY ON INTERACTIVE RESPONSE ANALYSES BETWEEN SOIL AND PILE FOUNDATION UNDER LIQUEFACTION

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SUMMARY

The behavior of pile foundation under a severe earthquake is an interesting issue but it usually entails numerous analytical and experimental works. Two-dimensional (2-D) effective stress analysis, for example, is one of the promising methods to observe their dynamic behavior, however, it requires complicated procedures for modeling and calculation. As a result, many researchers to date have reported studies on evaluation methods for degradation in shear modulus due to earthquakes in order to more simply predict the behavior of foundation during earthquakes. This paper aims to propose a simple and practical analyzing method for pile foundation installed in sand and silt layers where liquefaction is likely to occur. An equivalent shear modulus during seismic motion is introduced, which was defined from the result of one dimensional (1-D) effective stress analysis considering the rise in excess pore water pressure, and 2-D total stress analysis was adopted with using this equivalent shear modulus. The equivalent shear modulus proposed by the authors is helpful for a very severe earthquake where the maximum shear strain of soil becomes more than 1% despite it giving smaller response for a relatively weak earthquake than that obtained from the 2-D effective stress analysis. In addition, the applicability of the proposed method is verified by varying the amplitude of acceleration of incidental earthquake.

INTRODUCTION

Accompanying the progress in computer technology, analyzing techniques have improved during the past several decades. One of the most promising methods for analysis of liquefaction is the effective stress analysis [Ishihara and Towhata, 1980]. This method solves two-phase problem of FEM usually based on a constitutive law for non-linear behavior of soil and there is a wide variety of constitutive laws proposed by many researchers [Finn, Lee and Martin, 1977] that are being practically applied. Among them, the S-D model, proposed by Cubrinovski [Cubrinovski, 1993] and Ishihara [Ishihara, 1993], can evaluate the response of the soil under various conditions of stress and density over a wide range by using the state index I_S. Since pile foundation is generally applied for thick soil layers covering bedrock and their initial confining stress greatly varies along the depth, the S-D model which allows consistent modeling for both upper and lower layers with different confining stresses possible was judged appropriate for this study.

The authors applied the S-D model to a one-dimensional problem and the characteristics of S-D model were reported [Togashi, Nakafusa and Goto, 1997]. In this paper, the S-D model was adopted for a 2-D model in order to observe the behavior of pile foundation considering liquefaction. Since 2-D effective stress analysis requires much work and time for modeling and calculation, the authors propose an alternative method using total stress with degrading shear modulus based on the 1-D effective stress method. With this method, the deteriorated shear modulus of soil used in the total stress analysis is obtained from the usual G - γ relation as a value of G at the maximum strain given by the 1-D effective stress analysis. These two analyses give close

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solutions to each other for a relatively strong earthquake where the shear strain of soil becomes more than 1%. In the case of a relatively weak earthquake, however, the response is smaller than that obtained by the effective stress method. The applicability of this proposed method based on the results by varying the maximum amplitude of acceleration of input earthquake is discussed at the end of this paper.

METHOD OF ANALYSES

The Effective Stress Model (S-D Model):

The S-D model proposed by Cubrinovski and Ishihara was adopted as the sand model for the effective stress analysis. In the conventional sand model, different index properties have to be set depending on the density of the soil whereas in the S-D model, the response of the soil can be correctly evaluated for various conditions of stress and density over a wide range by using the state index I_S. That is, the physical properties of soil can be correctly reflected by the state index I_S. Moreover, the S-D model has the advantage that if the parameters for one condition of the soil are set, the model can be used for different conditions of initial confining stress or relative density. The state index I_S is an index based on the test results of many tri-axial compression test specimens and hollow cylindrical test specimens in the drained and undrained conditions. It is determined from the relationship between void ratio and mean effective stress. The state index I_s is calculated from Eq. (1) using the QSS-line (Quasi Steady State) and UR-line (Upper Reference) on the e-p plane (defined by the void ratio and mean effective stress) as shown in Figure 1. Point A indicates the condition of sand, point B is a point on the quasi steady state line and point C is a point on the upper reference line.

$$Is = \frac{\Delta e_c}{\Delta e_{qss}} \tag{1}$$

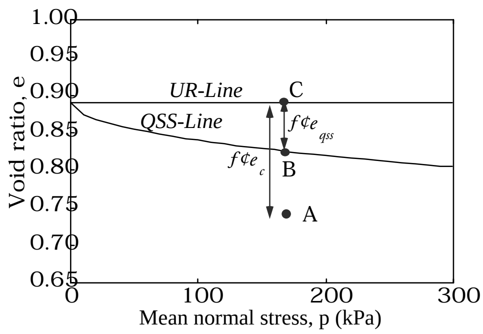

Figure 1: Definition of the state index I_s

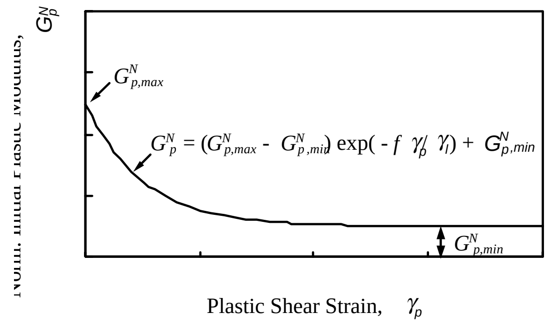

Figure 2: Degradation in *G*

The S-D model adopts a hyperbolic relationship for the plastic component of strains as the stress-strain model. The relation between shear stress τ and effective stress p' is ultimately defined by the following equation (2).

$$\frac{\tau}{p'} = \frac{G_p^N \gamma_p \left(\frac{\tau}{p'}\right)_f}{\left(\frac{\tau}{p'}\right) + G_p^N \gamma_p} \tag{2}$$

τ : shear stress, p' : effective stress, G_p^N : normalized initial plastic modulus by p', γ_p : plastic shear strain

G_p^N is assumed as a function of strain shown in Figure 2 and is expressed by equation (3).

$$G_p^N = \left(G_{p,\max}^N - G_{p,\min}^N\right)\exp\left(-f\frac{\gamma_p}{\gamma_l}\right) + G_{p,\min}^N \tag{3}$$

γ_l : limit strain, f : exponential parameter

The state index I_s is applied to define the variables introduced above. The relations among these variables are shown in equations (4) through (6).

$$\left(\frac{\tau}{p'}\right)_f = \alpha_1 + \beta_1 I_s \tag{4}$$

$$G_{p,\max}^N = \alpha_2 + \beta_2 I_s \tag{5}$$

$$G_{p,\min}^N = \alpha_3 + \beta_3 I_s \tag{6}$$

The constants of α_1 to β_3 can be calculated to concur with the experimental results for soil specimens.

Alternative Method Proposed:

In both 1-D and 2-D analyses, the equivalent linearization analysis and effective stress analyses (S-D model) were carried out, and the effects of the rise in excess pore water pressure on structures were qualitatively evaluated. However, the effective stress analysis requires iteration for convergence of shear modulus and pore water pressure at each time step as a trade-off of its accuracy. To resolve this issue, simplified technique is desirable for practical usage such as for design of actual structures.

The authors analyzed the dynamic response using a total stress analysis with equivalent shear modulus that stands for deteriorated shear resistance due to excess pore water pressure. In the case of pile foundation structures, the behavior of the pile is mainly determined by the deformation of soil layers so that the 1-D effective stress analysis still gives good coincidence with the 2-D analysis. The equivalent shear modulus was estimated through the usual G - γ relations with γ_{max} obtained by 1-D effective stress analysis. That is, the maximum and minimum values of shear strain, γ_{max} and γ_{min} obtained from 1-D effective stress analysis, and the modified H-D model equation were used for setting the soil properties considering rise and accumulation of excess pore water pressure. The method of calculating the shear modulus G_{ep} and damping coefficient h_{ep} after considering the rise and accumulation of excess pore water pressure is given in Eq. (7) and Eq. (8).

$$\frac{G_{ep}}{G_0} = \frac{1}{\left(1 + \frac{(|\gamma_{\max}| + |\gamma_{\min}|)}{2 \times \gamma_{0.5}}\right)} \tag{7}$$

$$h_{ep} = h_{\max}\left(1 - G_{ep}/G_0\right) \tag{8}$$

Where

G_0 : Initial shear modulus of soil

$\gamma_{0.5}$: Reference strain of soil

h_{max} : Maximum damping coefficient of soil

Assuming the equivalent shear modulus for each sand layer, 2-D linear analysis was carried out. Figure 3 shows the flow of the alternative method.

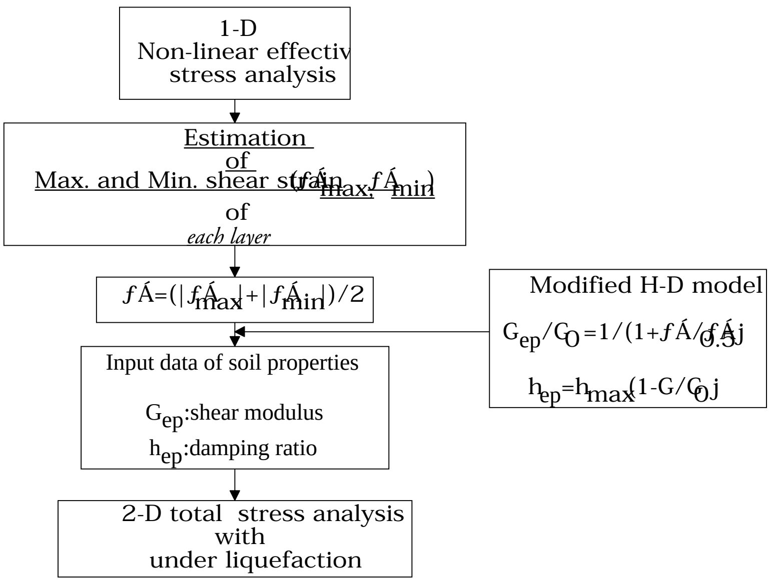

Figure 3: Flow of the Alternative Method

SOIL AND STRUCTURAL CONDITIONS

Soil Condition

Table 1 shows the configuration of layers and properties of soil such as shear wave velocity V_s and shear modulus G_0 of the soil used in the research. The soil model has alternate layers of silt, sand, and gravel up to a depth of 68 m and soft rock at depth below 68 m. The underground water level is 6 m below the ground surface.

Sampling was carried out, and liquefaction tests and dynamic deformation tests were conducted for the silt, sand and gravel layers.

Two types of seismic waves, S2 and S1, were used as input earthquake motions in the analysis. S2 represents extreme conditions, return period as long as 50,000 years with maximum acceleration of 400 gal recorded at about 8.5 seconds. S1 stands for a moderately strong earthquake with 10,000-year return period and the maximum acceleration of about 300 gals. Figures 4 and 5 show the time histories of input earthquake motions.

Table 1 Soil configuration and soil properties

GL (m)	Soil type	γ	V_s	G_0
-4.0	Fill	17.7	170.0	52000
-10.0	Sand 2	19.6	220.0	97000
-12.0	Gravel 2	20.1	270.0	149000
-24.0	Silt 4	16.2	180.0	54000
-28.0	Sand 1	18.1	240.0	107000
-38.0	Silt 3	15.7	190.0	58000
-53.0	Silt 2	16.2	220.0	79000
-58.0	Gravel 1	20.1	270.0	149000
-63.0	Silt 1	17.2	240.0	101000
-68.0	Gravel 1	20.1	350.0	251000
-78.0	Mudstone	17.2	480.0	402000
-88.0	Mudstone	17.2	500.0	441000

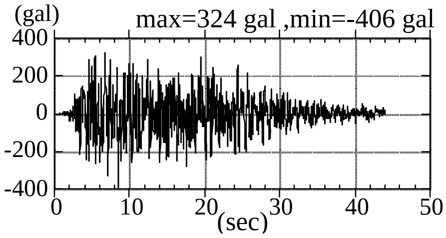

Figure 4: Input earthquake (S2)

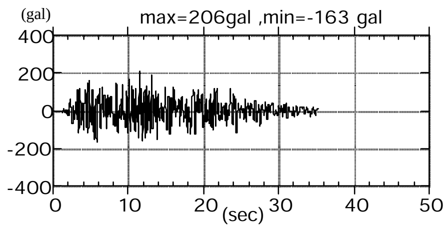

Figure 5: Input earthquake (S1)

Structural Condition

The structure analyzed is as shown in Figure 6. A concrete box culvert for cooling water is continuously supported by steel pipe piles in the longitudinal direction. The longitudinal length of culvert is set as 4 m, which coincides with the longitudinal distance of piles for all of the calculation cases. Figure 7 shows the 2-D model for the effective stress method.

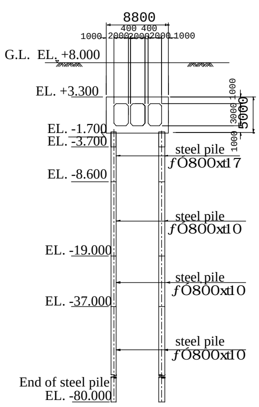

Figure 6: Structural model

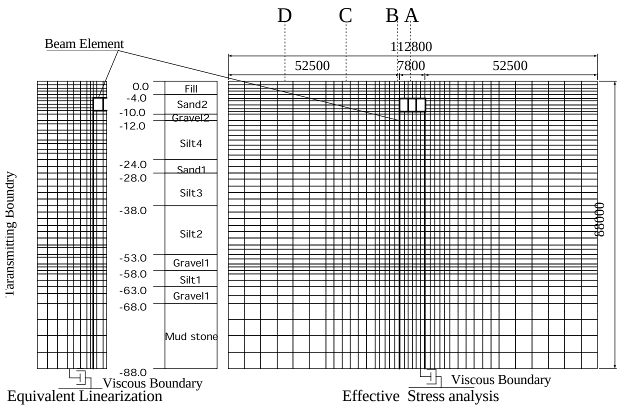

Figure 7: Model for analysis

RESULTS BY THE EFFECTIVE STRESS METHOD

Both 1-D and 2-D analyses by the effective stress method were carried out. Figure 8 shows the contour diagram of excess pore water ratio during the S2 wave and the final deformation after the earthquake is illustrated in Figure 9. As shown in these figures, high pore water pressure ratio was recorded in the sand layer located 24 m under the ground level. Existence of the structure slightly affects the distribution of excess pore water pressure ratio since the ratio at a same depth keeps almost constant regardless of the horizontal location. The history of excess pore water pressure ratio in the sand layer and its stress path are shown in Figure 10 and Figure 11, respectively. From these figures, it can be seen that the shear resistance of sand is constant even though the excess pore water pressure ratio rises to almost 1.0. Such behavior of sand may be called a cyclic mobility phenomenon. The excess pore water pressure ratio of silt also rises to around 0.4. The shear resistance of silt gradually decreases with the rise of pore water pressure ratio and the strain becomes large during the earthquake. In both cases, evaluation of excess pore water pressure ratio is obviously important for prediction of the deformation of soil and piles.

In order to discuss the applicability of the results of 1-D analysis to 2-D problem, the maximum shear strains along several vertical lines, lines A through D (see Figure 7) in the 2-D model were compared. As shown in Figure 12, no significant difference in the distribution of strain was found because pile foundation is so flexible that the deformation of piles are dominated by the motion of soil layers surrounding the structure. Since the 1-D analysis without the structure gives almost the same distribution to that along line D, which is far enough from the structure, 1-D analysis gives reliable results to estimate the deformation of pile foundation. This fact is very important for establishing the validity of the alternative method introduced in the following section.

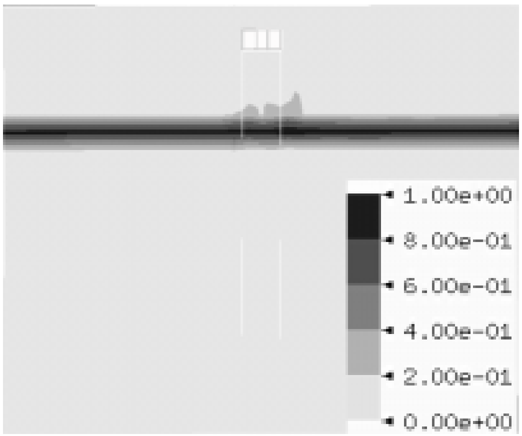

Figure 8: Contour of excess pore water pressure ratio time=10.0sec

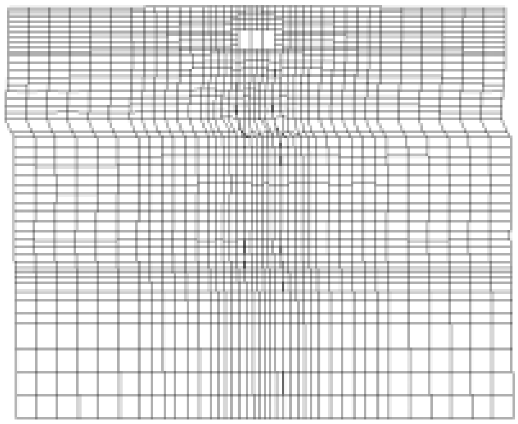

Figure 9: Final deformation after the earthquake

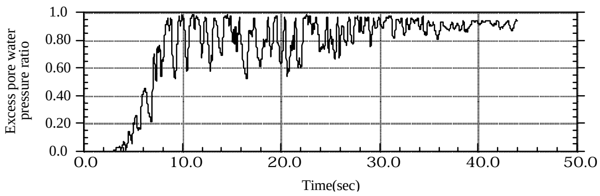

Figure 10: Pore water pressure ratio of sand

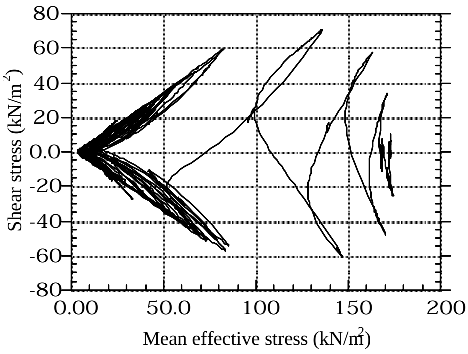

Figure 11: Stress path of sand

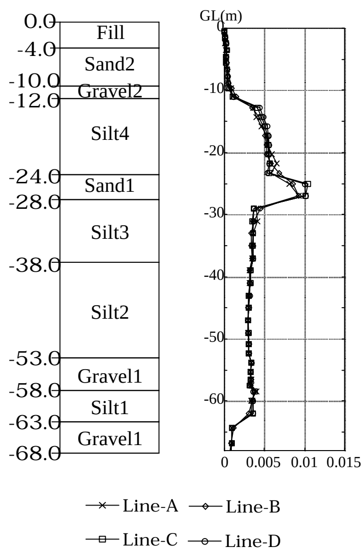

Figure 12: Maximum shear strains along vertical lines

COMPARISON WITH THE PROPOSED METHOD

Based on the calculation results shown above, giving consideration to the effect of excess pore water pressure on the whole behavior is essential for the model. This chapter introduces the results by the alternative method proposed and some comments on its validity.

Dynamic Response by S2 Wave:

The soil properties G_{ep} and h_{ep}, which consider the rise and accumulation of excess pore water pressure, were used in the analysis. Figure 13 shows the distribution of sectional forces and relative displacements of steel pipe pile obtained from total stress analysis, together with the sectional forces obtained from 2-D effective stress analysis and equivalent linearization method. From this figure, it is observed that the distribution of response forces of the proposed method practically envelops the distribution of the sectional forces of effective stress analysis. The relative displacements by the proposed method almost coincides with those by the effective stress method. This shows that the correct sectional force can be obtained by using G_{ep} and h_{ep} as the values of soil properties even in the total stress analysis.

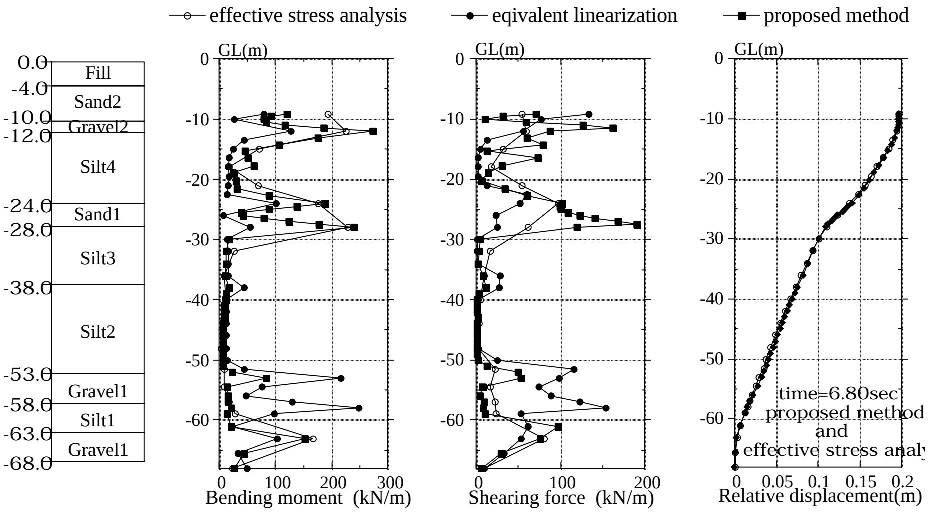

Figure 13: Distribution of sectional forces obtained from different methods of analysis for S2WAVE

Dynamic Response by S1 Wave:

Similarly, S1 wave was also applied to this model. The calculation results are shown in Figure 14. The alternative method yielded solutions close to those obtained by the 2-D effective stress method but compared with the case by S2 wave, the bending moment and relative displacement calculated were slightly underestimated.

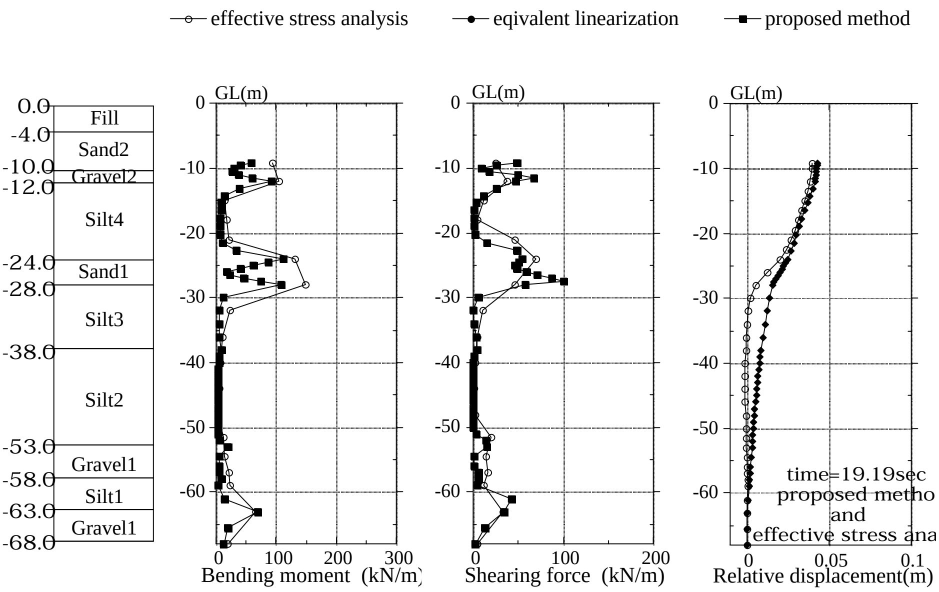

Figure 14: Distribution of sectional forces obtained from different methods of analysis for S1WAVE

Validity of the Proposed Method:

For practical design of structure, being able to easily estimate the maximum sectional force is very helpful. The comparison of alternative method with 2-D effective analysis implies that this method may be a prospective procedure for prediction of maximum sectional forces. According to the comparison of calculation results between S2 and S1 waves, the validity of the alternative method seems to depend on the resultant maximum shear strain because the stronger the input earthquake becomes, the smaller is the difference from the results obtained from 2-D effective stress analysis. Figure 15 shows the distribution of shear strain obtained from both the 2-D effective stress method and the proposed method by varying the maximum acceleration of input earthquake. It can be said that the equivalent shear modulus obtained by γ_{max} has slight difference from that used in the effective stress method if the maximum shear strain is so large that the shear modulus is stable and close to minimum value for G. On the other hand, if the γ_{max} is moderate, it becomes difficult to assume an appropriate value of the equivalent shear modulus because G varies drastically with γ on the G - γ relation.

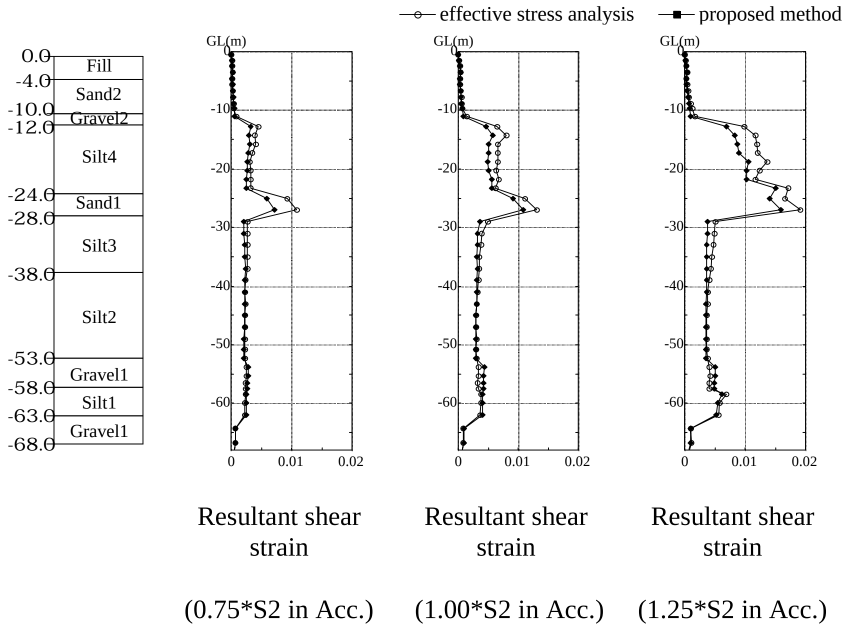

Figure 15: Distribution of shear strain

CONCLUSIONS

The conclusions of this study are summarized as follows;

1. No significant disturbance in pore water pressure was seen in the vicinity of the pile foundation in the 2-D effective stress model. Even 1-D effective stress analysis could sufficiently evaluate the stiffness degradation due to liquefaction.

2. The equivalent shear modulus obtained by G - γ relation with the result of 1-D effective stress method, stands for an appropriate shear resistance for the total stress analysis.

3. In the case of considerably strong earthquake, the proposed method gives close values of sectional forces to those by obtained from 2-D effective stress analysis, which are sufficiently reliable enough for practical design.

4. If the amplitude of input earthquake is moderate and resultant shear strain of soil still remains within medium range of in G - γ relations, the proposed method tends to underestimate the response.

At the end of paper, the authors are grateful to Dr. Cubrinovski for providing invaluable assistance on the analysis by the S-D model.

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