I －B 383 DYNAMIC ANALYSIS OF UNDERGROUND STRUCTURES AT LIQUIFIED SITES

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1.INTRODUCTION

Earthquake induced liquefaction usually cause catastrophic damages to underground facilities as evidenced during the Northridge and Great Hanshin earthquakes. Because of its potentially damaging effects, the modeling and prediction of dynamic response of buried structures at liquefied sites has become a major topic of interest. However, due to the complex nature of the liquefied soils and the soil-structure interaction effects, it is very expensive to perform either 2D or 3D nonlinear effective stress FEM analysis. Therefore in this paper, authors propose an equivalent linear analysis method using the 2D-FEM computer program FLUSH[i] [1]. The above FEM code can produce an approximate nonlinear solution by incorporating the strain dependent shear modulus and damping ratio [1]. Therefore, large shear deformations which occur in soils during strong earthquakes can be accommodated. Further, to assess the proposed method, it was used to predict the dynamic response of Wildlife Site – an instrumented site where liquefaction occurred during the 1987 Superstition Hills earthquake.[1]

2.METHOD OF ANALYSIS

The most important aspect to make a simple constitutive model for soil which is susceptible to liquefaction is to separate its stress-strain behavior before and after liquefaction. Therefore the proposed method has to be divided into three parts; preliminary analysis, pre-liquefaction analysis and post-liquefaction analysis. In the first phase, the conventional dynamic response analysis has to be carried out to determine the shear stress time history. From the maximum shear stress (equivalent sinusoidal amplitude is 65% of maximum) one can calculate the safety factor (F_l) to identify the elements which undergo liquefaction. That is, $F_l = R_{20}/(0.65\,R_{\max})$, refer Fig. 2 for R_{20} and $R_{\max}$ can be obtained from Fig. 1. Where shear stress ratio $R_i = \tau_i / \sigma'_v$. To evaluate the time for onset of liquefaction, define a new factor of safety [3] $F_l^*\ (1/2N_1 + 1/2N_2 + \cdots + 1/2N_l)$ which is in a sense reciprocal of F_l. Further, for each and every peak shear stress ratio (R_i), see Fig. 1, the corresponding time (t_i) and number of cycles to cause liquefaction (N_i), see Fig. 2, can be found. The time (t_l) which corresponds to the peak shear stress ratio (R_l) is considered as the time for onset of liquefaction. Where, R_l is the peak shear stress ratio at which the soil element get liquefied $(F_l^* \geq 1)$. Once the liquefaction is triggered, soil mechanically exhibits properties very similar to incompressible fluid with the same density as soil. Since, the characteristics of soil before and after the liquefaction differs drastically, it is rational to separate the analysis. To facilitate individual analysis for pre and post liquefaction, the input acceleration time history has to be separated at the time when liquefaction is triggered (t_l). Before the liquefaction, soil behaves in the typical manner therefore the pre-liquefaction analysis is identical to the conventional one. The liquefied soil is assumed as an visco-elastic body with a very small modulus of shear relative to that of unliquefied element (1/500 → 1/1000) and damping ratio nearly in the range of 32 to 34. It should be mentioned at this point that the above mentioned abrupt drop in shear modulus is assumed to take place when the shear strain is 1%. Further, Poisson ratio ($\nu \approx 0.48$) has to be re-estimated [3] considering the incompressibility of liquefied element. Therefore new data set has to be prepared for the post liquefaction analysis in view of the above considerations [3] for liquefied elements.

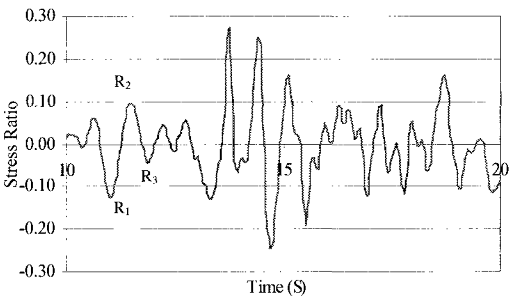

Fig. 1: Shear Stress Ratio for Sub Layer No. 4

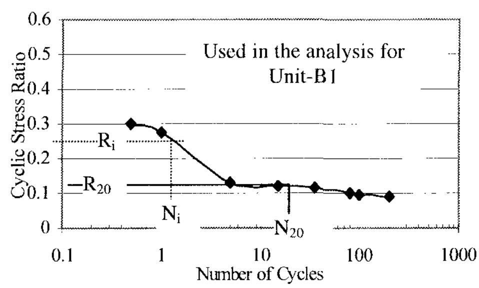

Fig: 2 Cyclic Strength of Soil

3.VALIDATION OF ALGORITHM

The 1D model shown in Fig.3 was used in the analysis of Wildlife Site. The maximum shear modulus (G_{max} in tf/m^2) and unit weight (γ_t in tf/m^3) are shown in

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Fig. 3. Poisson ratio was assumed as 0.35 for all layers (before the liquefaction). Unit-A, has sufficiently high clay content to be liquefaction resistant. Moreover, most of this unit is located above the water table. Unit-B1 consists of a loose moderately sorted sandy silt and Unit-B2 consists of loose to medium dense silty sand to very fine sand. The input motion used for this analysis was recorded at a depth of 7.5 m but in this analysis it was given at the bottom of Unit-B2 (7.0 m). The factor of safety against liquefaction (F_l) calculated from the preliminary analysis for Unit-B1 (Sub layers. 4 and 5) is about 0.82 whereas F_l for Unit-B2 is about 1.2. Thus, only Unit-B1 was liquefied. At the site, significant pore pressure rises were observed in Unit-B1 and Unit-B2. However, only Unit-B1 developed 100% excess pore pressure during this earthquake. The time for onset of liquefaction (t_l) was estimated as described in the previous section and found that Unit-B1 liquefies approximately after 18 seconds. From the field observations it was concluded [2] that Unit-B1 liquefies roughly after 17 seconds. The predicted maximum surface acceleration is 208 cm/s^2 and occurs at 14 seconds, and the observed maximum acceleration is 200 cm/s^2 and at time 14 seconds. It can be noted that maximum acceleration occurs before the onset of liquefaction. The computed and measured maximum relative displacements (datum point at 7.0 m) are 14 cm and 12 cm respectively; obviously this happens after the onset of liquefaction. The predicted and observed maximum (local) relative displacements before the liquefaction are 2.5 cm and 1.5 cm respectively. However, computed maximum (local) acceleration at the ground surface in the post liquefaction session is very much smaller than the observed maximum acceleration (local), this can be explained [3]. The maximum shear strain distribution computed from this analysis is shown in Fig. 4. From the above comparison, it is apparent that the numerical results obtained from this analysis are in reasonable accord with the observation. In this paper, the proposed approach is verified for a 1D model but this can be extended for a 2D model (with buried structures) with out any hesitation [3].

4.CONCLUDING REMARKS

The results presented in this paper clearly indicate that equivalent linear analysis approach can also predict some vital information such as maximum relative displacement and maximum stress which are necessary for the design of underground utilities. However, the accuracy of this analysis method depends highly on soil properties used in the analysis, such as cyclic strength of the soil and strain dependent shear modulus and damping ratio of liquefied elements.

5.REFERENCES

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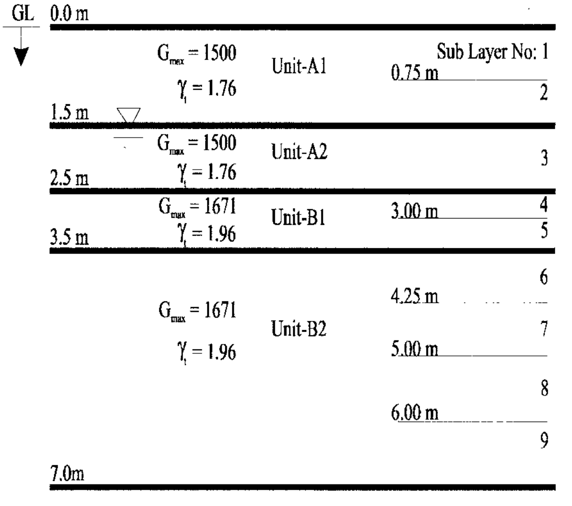

Fig. 3: Soil Condition and Layering

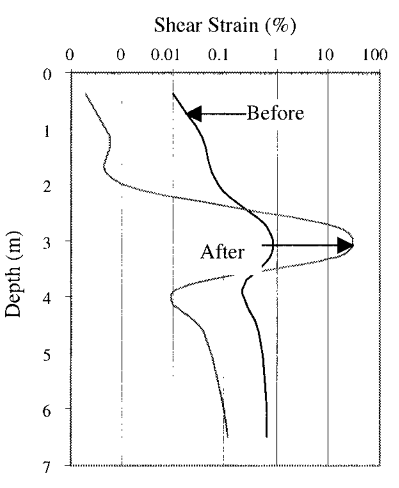

Fig 4: Maximum Strain Distribution

[i] This analysis was carried out using micro-FLUSH (Windows-95 version) which was produced by "Jishin Kogaku Kenkyusho, Inc." in Tokyo. The authors also greatly appreciate the key idea given by Dr.T.Udaka (President, Jishin Kogaku Kenkyosho).