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Ground model for seismic response analysis in Hualien project of large scale seismic test

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ABSTRACT An international joint research program called "HLSST" is proceeding. HLSST is Large-Scale Seismic Test (LSST) to investigate Soil-Structure Interaction (SSI) during large earthquake in the field in Hualien, a high seismic region in Taiwan. In this site a 1/4-scale model building was constructed, and the model building and the foundation ground were extensively instrumented to monitor structure and ground response. To accurately evaluate SSI during earthquakes, geotechnical investigation and forced vibration test were performed during construction process namely before /after base excavation, after structure construction and after backfilling. This paper describes the change of the shear wave velocity (Vs) measured by the field test. Discussion is made on the effect of overburden pressure during the construction process on Vs in the neighboring soil and, further on the numerical soil model for SSI analysis.

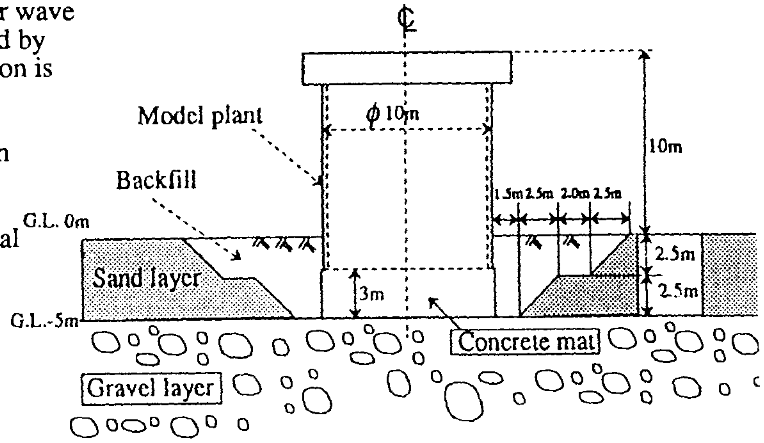

Fig. 1 Outline of model building

1.INTRODUCTION

The mechanical soil properties of the foundation soil, especially just around and beneath a structure, play an important role to evaluate the SSI effect. As the stress of the foundation ground changes due to the construction process, the soil property is expected to change leading to the change in SSI characteristics. Therefore, the soil properties of foundation ground should be determined under the actual in-situ condition after the completion of a structure. However the mechanical property of foundation ground has not usually been measured in such a way, but assumed by the geotechnical investigation results before the model construction.

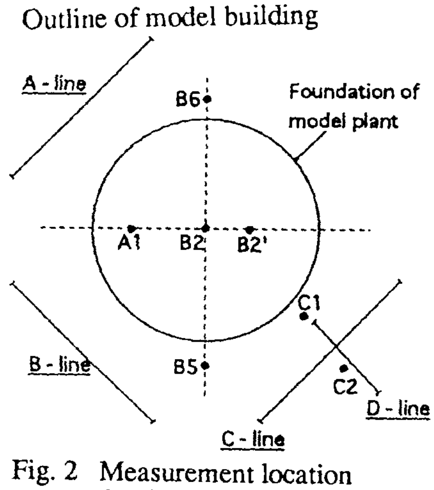

Fig. 2 Measurement location by down-hole-logging

In this respect to accurately evaluate SSI during earthquakes, systematic geotechnical investigations have been carried out by the authors for a 1/4-scale model building constructed in Taiwan as an international joint research program called "HLSST" (Tang et al.1992). The geotechnical investigation has been carried out both in field and laboratory. The field tests consist of borings, large penetration tests, PS-loggings and seismic refraction surveys. The laboratory test program includes triaxial tests using undisturbed samples taken by the freezing sampling technique and other physical and mechanical property tests. To evaluate the change of Vs of the model building foundation ground during the construction process, the field tests were conducted in four phases: before excavation (stage 1), after excavation (stage 2), after model construction (stage 3), and after backfill (stape 4).

This paper describes the results of PS-logging conducted in the four phases. Furthermore, the change of Vs during construction process is discussed and soil property for numerical analysis is proposed.

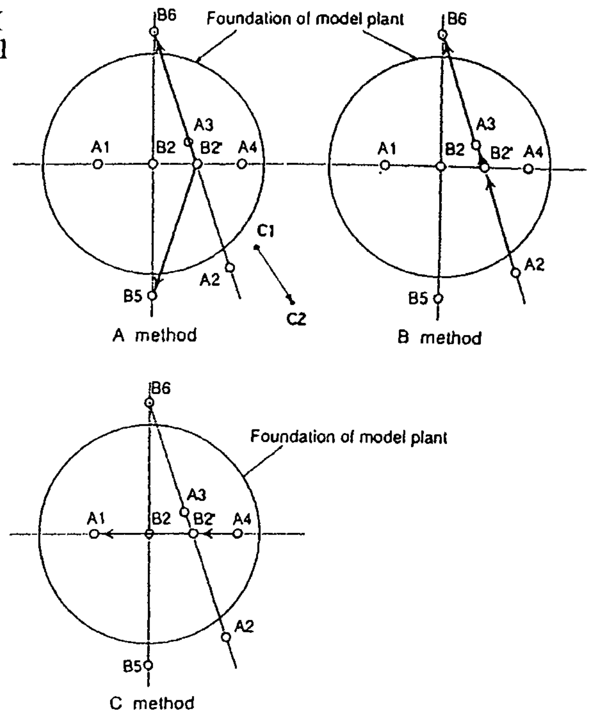

Fig. 3 Measurement location by cross-hole-logging

2.OUTLINE OF INVESTIGATION

2.1 Site Characteristics (Kokusho et al. 1993)

Fig.1 shows the outline of the model building. The model is a cylindrical building (f=10m, h=15m) constructed on a gravel layer at G.L.-5.0m beneath a surface sand layer. The backfill was compacted by a crushed stone of middle particle size. In this site, the Large Penetration Test (LPT) instead of standard penetration test (SPT) were performed, because gravelly soil layer was so dense as to decrease the reliability of SPT whose driving energy is much smaller than LPT. N_{LPT}, the penetration resistance of LPT, is the blow counts required for a 30cm drive by freely dropping a 100kg hammer from 150cm in height. The relationship between N_{LPT} and N value by SPT is given in the following form (Yoshida et al. 1988) of: $N = 1.5\ N_{LPT}$ (sandy soil) $N = 2.0\ N_{LPT}$ (gravelly soil) N_{LPT} of the sand layer was ranged from 5 to 10 that were empirically equivalent to 7.5 to 15 of N. N_{LPT} of the gravel layer was 20 to 50 corresponding to 40 to 100 of N, indicating it was dense or very dense.

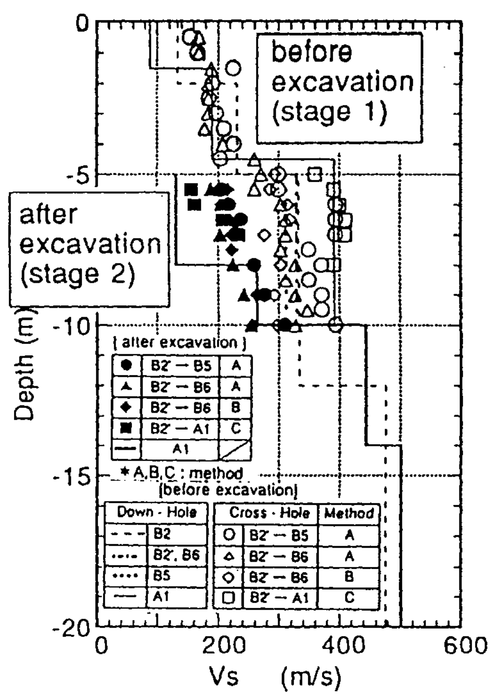

Fig. 4 Vs distribution before and after excavation (stage 1 and stage 2)

2.2 PS-logging (Kokusho et al. 1993, Kudo et al. 1994)

(1) Down-hole-logging : Fig.2 shows the locations of the boreholes for the down-hole-logging, which were carried out at A1, B2, B2', B5, and B6 before the excavation (stage 1), at A1 after the excavation (stage 2), and at B2' after backfill (stage 4). In the same figure the measuring lines for the refraction survey for the backfill soil is shown.

(2) Cross-hole-logging : Fig.3 indicates the wave-paths incorporated in the cross-hole-logging. The cross-hole-logging was performed using A, B and C methods. The A method means that S-wave generated at B2' was received at B5, A3, and B6. Similarly by the B method, S-wave generated at A2 was received at B2', A3, and B6, and the C method indicates that S-wave generated at A4 was received at B2' and A1. The S-wave velocities were measured by these methods before (stage 1) and after the excavation (stage 2) and after model construction (stage 3) except for the A method from B2' to A3. After backfill (stage 4), only the measurement by A method from B2' to A3 was done. Furthermore, C1 and C2 in Fig.2 are the holes for down-hole and cross-hole measurements for the backfill soil.

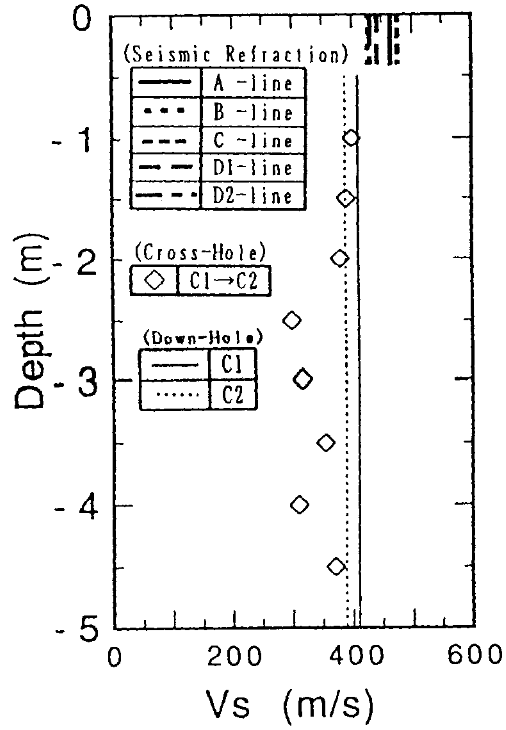

Fig.5 Vs distribution of backfill

3.TEST RESULTS AND DISCUSSION

3.1 Result of PS-logging

Vs obtained by the down-hole and cross-hole-loggings before and after the excavation (stage 1 and stage 2) are summarized in Fig.4. According to the results by the down-hole-logging, the sandy soil layer may be characterized as two layers; the upper layer with 2m thickness has Vs=90-130m/s, and the lower layer has Vs=190-230m/s between G.L.-2 to -5m. From the some results by the down-hole-logging, the gravelly soil layer may also be classified into two layers : the upper layer of Vs=310-390m/s from G.L.-5m to

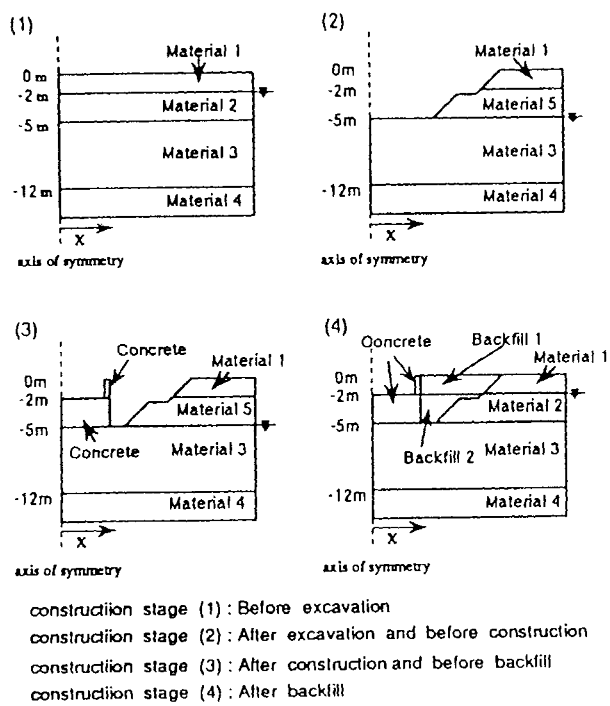

Fig.6 Analytical model for stress change due to model construction process

G.L.-12m , the lower layer of Vs=480m/s from G.L.-12m to G.L.-20m. Based on these results, the shear wave velocities for a free field which is free from the effect of the stress change due to the construction of the model building are determined and incorporated in the final analytical soil model as shown in Fig.9.

In Fig.4 the considerable reduction of Vs can be noticed between the two measurements before and after the excavation. Fig.5 shows Vs of backfill by down-hole logging, cross-hole logging and seismic refraction of short distance measured along the lines shown in Fig.2. In this figure Vs by down-hole is smaller than Vs by seismic refraction and bigger than Vs cross-hole, though Vs is almost 400m/s for all the depth of the backfill indicating that backfill was compacted very dense.

3.2 Discussion on Overburden Effect

The significant change of Vs was observed during the model construction process as shown in Figs.4. The confining stress-dependency of Vs of soil samples was measured by dynamic triaxial tests using undisturbed samples taken by the freezing sampling technique. The relationship between Vs and the effective confining stress,s_c', based on the laboratory tests, is expressed in the following form ; $Vs \propto s_c'^{0.3}$

Next, the change of the mean effective stress in the ground due to the model construction was obtained by an elastic FEM analysis. Fig 6 shows the analytical models. Then Vs was calculated by using the above relation based on the stress change and the initial Vs measured for the gravel layer by the average of the S-wave before excavation.

Figs.7 and 8 indicate measured and calculated Vs after excavation and after backfill, respectively as compared with the measured Vs obtained by the cross-hole method and the conventional down-hole as well as moving average down-hole methods (Kudo et al. 1993,1994). As shown in these figures, the calculated Vs values show a satisfactory agreement with those measured by PS-logging. Consequently, it may be concluded that the change of Vs of the foundation ground during the model construction process is mainly dependent on the unloading and loading of overburden pressure due to the construction process.

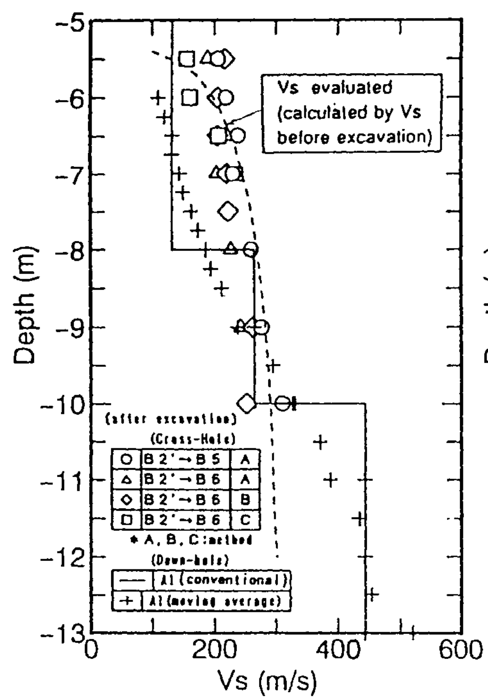

Fig.7 Comparison of measured and calculated Vs after excavation (stage 2)

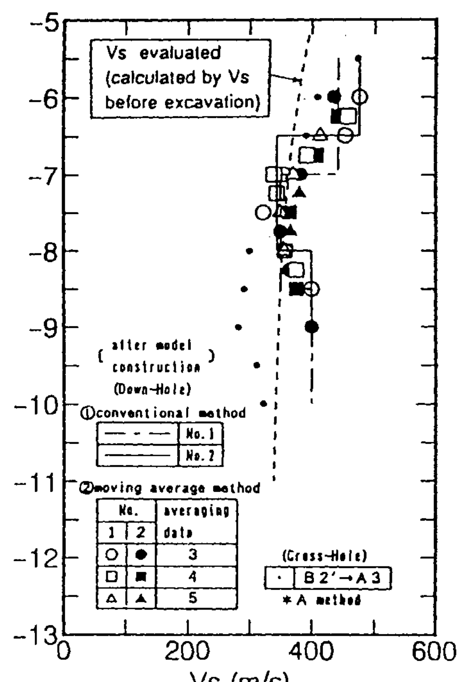

Fig.8 Comparison of measured and calculated Vs after backfill (stage 4)

3.3 Analytical soil model of the foundation ground

Fig.9 shows the final soil model to be incorporated in the SSI analysis for the completed construction after backfill (stage 4). Taking account of the previously mentioned test results, the ground just beneath the model foundation named Gravel 1 (G.L.-5.5~-12m) was modeled as a single soil unit with a uniform soil property. Vs of this unit was determined as 383m/s by averaging measured values after backfill. On the other hand Vs of this soil unit before backfilling (stage 3) was judged to be 317m/s based on the cross-hole logging results . Other parts of the model were made four layers : Sand 1, Sand 2, Gravel 2 and Gravel 3 according to the free field measurements before excavation. The backfill is divided into Backfill 1 and Backfill 2 at the ground-water level.

The Poisson's ratio ν were calculated based on the P and S wave velocities, soil density rt were calculated by soil samples taken by the freezing sampling technique, and hysteric damping ratio h were measured by laboratory cyclic triaxial tests.

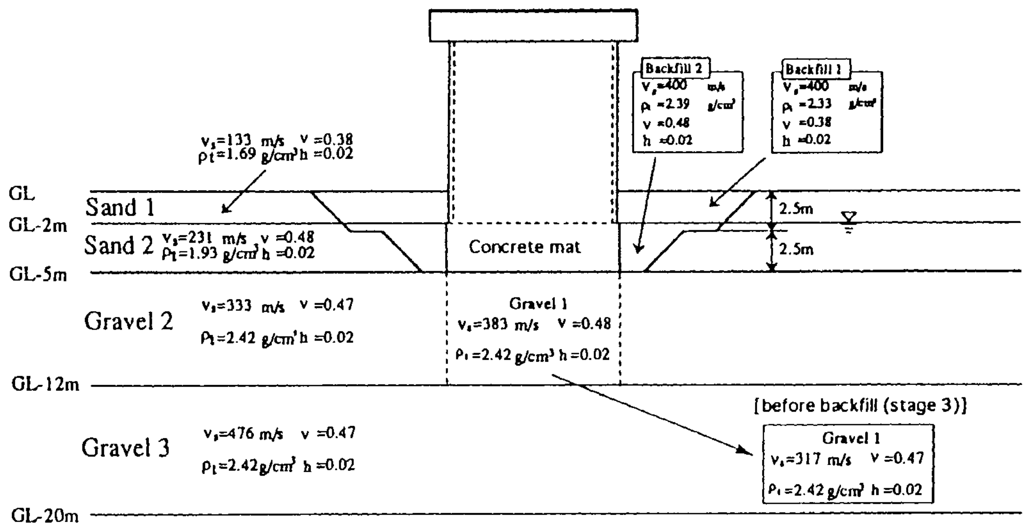

Fig. 9 The soil model after backfill (stage 4)

CONCLUSION

Findings obtained by this study are summarized as follows:

(1) The change in Vs can be accurately detected by the wave-logging method in the ground just beneath the model building during construction process.

(2) Vs of the foundation ground changes during the model construction process due to the unloading and loading of overburden pressure. Based on the above-mentioned in-situ wave measurements the foundation ground around the model building has been idealized for SSI numerical analysis.

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