MODELING ANISOTROPIC BEHAVIOR OF GRAVELLY LAYER IN HUALIEN, TAIWAN

YUKIHISA TANAKA

ABSTRACT

The Hualien Large-Scale Seismic Test (HLSST) program is under way to investigate soil-structure interaction during earthquakes in Hualien, a high-seismicity region in Taiwan. Since anisotropic characteristics of the gravelly layer affect the results of earthquake observation and forced vibration tests, the gravelly layer was modeled by an orthotropic elastic body in this paper to enhance the accuracy of the dynamic simulation analysis. The following conclusions are drawn. (1) The gravelly layer was modeled by an orthotropic elastic body whose model parameters were partly determined using results of earthquake observation. (2) The azimuth dependency of the shear wave velocity measured by the cross hole velocity logging can be expressed by the orthotropic elastic body model. (3) It can be explained by the orthotropic elastic body model that values of shear wave velocity measured by down-hole velocity logging were divided into two groups. (4) The initial shear moduli of undisturbed samples from the gravelly layer evaluated by cyclic triaxial tests agree with the calculated results by the orthotropic elastic body. (5) The anisotropic behavior of the model building during forced vibration tests can be expressed successfully by the orthotropic elastic body.

Key words: anisotropy, (cyclic triaxial test), (forced vibration test), (gravelly soil), (initial shear modulus), (velocity logging) (IGC: D7)

INTRODUCTION

An international joint program, the Hualien Large-Scale Seismic Test (HLSST) program has been under way since 1990 to investigate soil-structure interaction during large earthquakes. A 1/4-scale model building was constructed on the gravelly soil layer at this site. Forced vibration tests (FVT) of model building before and after backfilling, and earthquake observation of the soil-structure system were conducted. Extensive geotechnical investigations including velocity logging and sampling by in-situ freezing technique were conducted for accurate evaluation of FVT and earthquake-induced responses of the model building and surrounding ground.

In this paper, the anisotropic behavior of the gravelly layer seen in earthquake observation and forced vibration tests is modeled by the orthotropic elasticity theory.

In this paper, “undisturbed samples” means samples obtained by the in-situ freezing technique.

OUTLINE OF HLSST PROGRAM

Some results of the HLSST have already been reported (Kokusho et al., 1993; Morishita et al., 1993; Tanaka et al., 1994; Ueshima et al., 1994; Okamoto et al., 1995a; Okamoto et al., 1995b; Yamaya et al., 1995; Ueshima et al., 1995a; Ueshima et al., 1995b; Hanazato et al., 1996; Kokusho et al., 1997; Ueshima and Okano, 1996; Okamoto et al., 1998; Tanaka et al., 1998a; Tanaka, 1998b; Tanaka and Okamoto, 1998c; Tanaka, 1999; Tanaka et al., 2000). Thus this report focuses on understanding the unusual characteristics of the gravelly layer described later.

The HLSST is under way in Hualien, a high-seismicity region in Taiwan (Fig. 1). Figure 2 shows a cross section of the cylindrical 1/4-scale model building and surrounding ground. The model building was constructed on the

---

Fig. 1. Location map of Hualien site

---

b Senior Research Engineer, Geotechnical and Earthquake Engineering Department, Central Research Institute of Electric Power Industry. Manuscript was received for review on November 15, 1999. Written discussions on this paper should be submitted before January 1, 2002 to the Japanese Geotechnical Society, Sugayama Bldg. 4F, Kanda Awaji-cho 2-23, Chiyoda-ku, Tokyo 101-0063, Japan. Upon request the closing date may be extended one month.
surface of the gravelly layer at G.L. — 5 m after excavating the surface sandy layer. The model building was backfilled with compacted crushed stone. The construction was divided into four stages:
Stage 1: Before excavation.
Stage 2: After excavation and before constructing model building.
Stage 3: After constructing model building and before backfilling.
Stage 4: After backfilling.
Velocity logging was conducted at each stage, while FVTs were conducted at Stages 3 and 4. The FVTs conducted at Stage 3 and Stage 4 are named FVT1 and FVT2, respectively.

OUTLINE OF SITE INVESTIGATION AND LABORATORY TEST

Field tests consisted of borings, large penetration tests, velocity loggings and seismic refraction surveys. The laboratory test program included triaxial tests using undisturbed samples and other mechanical and physical property tests.

Figure 3(a) shows the locations of boreholes for the site investigation and Fig. 3(b) shows the locations of three boreholes, DHA1, DHA2 and DHA3, for the down-hole array of accelerometers for earthquake observation conducted by the Electric Power Research Institute (EPRI) of U.S.A. Figure 4 shows the vertical arrangement of the accelerometers in DHA1, DHA2 and DHA3.

Geological Characteristics of Hualien Site

According to the report by Honsho (1990), the sandy layer from G.L. to G.L. — 5 m in Fig. 2 is the terrace deposit and the gravelly layer under the sandy layer is the Milun Formation, which belongs to the Pleistocene of the Quaternary period. Detailed descriptions have been reported previously (Tanaka, 1998b; Tanaka and Okamoto, 1998c; Tanaka et al., 2000).

Velocity Logging

The shear wave velocity of the gravelly layer and the sandy layer was measured by in-situ velocity logging. Velocity logging at Stage 1 was conducted by the down-hole method and the cross-hole method.

In measurement of shear wave velocity by the down-hole method, the SH wave was generated by hitting a wooden plank on the ground surface horizontally. It
should be noted that the azimuth of the hitting and the azimuth of the received acceleration were not recorded in the down-hole logging.

In measurement of shear wave velocity by the cross-hole method, the SV-wave was generated by hitting a metal bar vertically. The metal bar was connected to a plate which was pressed against the wall of the borehole. Detailed descriptions of the velocity logging have been reported previously (Kokusho et al., 1993; Kudo et al., 1994; Okamoto et al., 1995a; Okamoto et al., 1995b; Kokusho et al., 1997).

**Sampling by In-Situ Freezing Technique**

Sampling by the in-situ freezing technique was conducted in Stages 1 and 2. Detailed descriptions of the sampling procedure have been reported previously (Tanaka et al., 1998a; Tanaka et al., 2000).

**Laboratory Tests**

Cyclic triaxial tests at small strain amplitude and shear wave velocity tests were carried out to measure shear modulus. Details of these tests have been reported previously (Tanaka et al., 1994; Tanaka et al., 1998a; Tanaka et al., 2000).

**TEST RESULTS**

**Results of Large Penetration Tests**

Figure 5 shows the results of the Large Penetration Test (LPT) (Kaito et al., 1971; Tanaka et al., 2000). According to Fig. 5, the penetration resistance of LPT, $N_L$ values, of the sandy layer (G.L. 0 to $-5$ m) ranges from 5 to 10, whereas that of the gravelly layer is larger than 20. The base mat of the model building was on the surface of the gravelly layer at G.L. $-5$ m.

---

**Fig. 5.** Soil profile and results of large penetration test (quoted from Kokusho et al. (1993) and modified)
Results of Velocity Logging and Cyclic Triaxial Test

Figures 6(a) and 6(b) show the distribution of shear wave velocity at Stage 1 obtained by cross-hole velocity logging and down-hole velocity logging, respectively. In Fig. 6(a), shear wave velocity converted from initial shear moduli measured by cyclic triaxial tests (Tanaka et al., 1998a; Tanaka et al., 2000) are also plotted. Initial shear moduli by cyclic triaxial tests were calculated by dividing undrained Young’s moduli at shear strain amplitude of about $5 \times 10^2$ by 3. As described previously (Tanaka et al., 1998b; Tanaka et al., 2000), Figs. 6(a) and 6(b) indicate the following:

i) The shear wave velocity of the gravelly layer from G.L. $-5$ m to $-12$ m measured by velocity logging varies widely from 300 to 400 m/s irrespective of methods of measurement, whereas the scattering of shear wave velocity of the sandy layer from G.L. $-2$ m to $-4.5$ m measured by velocity logging is relatively small.

ii) Shear wave velocity by cyclic triaxial tests agrees approximately with shear wave velocity by velocity logging in the sand layer from G.L. $-2$ m to $-4.5$ m, whereas shear wave velocity by cyclic triaxial tests is smaller than shear wave velocity by velocity logging in the gravelly layer from G.L. $-5$ m to $-12$ m.

Results of Identification of Shear Wave Velocity of Ground by Inversion Analysis Using Earthquake Acceleration Records

Figure 6(c) presents the shear wave velocity identified by the one-dimensional inversion analysis using acceleration records observed in boreholes, DHA1, DHA2 and DHA3 and shown in Figs. 3(b) and 4 (Ueshima and Okano, 1996; TEPCO Group and Kajima Research Institute, 1996; Chen et al., 1996). All the earthquakes used for the inversion analysis were so small that nonlinearity of the ground could be ignored. Figures 6(a), 6(b) and 6(c) show that the shear wave velocity of the sandy layer from G.L. $-3$ m to $-5$ m is almost independent of the azimuth of the acceleration records and agrees with the results of velocity logging. However, according to Fig. 6(c), the shear wave velocity of the gravelly layer below G.L. $-5$ m shows obvious azimuth dependency.

Ueshima and Okano (Ueshima and Okano, 1996) defined the strong axis as the axis along which the maximum shear wave velocity is identified, while the weak axis is defined as the axis along which the minimum shear wave velocity of the gravelly layer is identified. In this paper, the weak and strong axes are called $U_1$-axis and $U_2$-axis, respectively. The azimuth of the $U_1$-axis is 100 (deg) east from magnetic north, while the $U_2$-axis is 10 (deg) east from magnetic north (Fig. 3(a)). Thus, they intersect at right angles.

FVT Results

The method of FVT has been reported previously (Morishita et al., 1993; Yamaya et al., 1995). Open circles in Figs. 7(a), 7(b), 7(c) and 7(d) denote the resonance

![Graphs showing comparison between observed and simulated results of FVT1 at Stage 3](image)

Fig. 7. Comparison between observed results and simulated results of FVT1 at Stage 3: (a) Response in y-direction during excitation in y-direction, (b) Response in x-direction during excitation in y-direction, (c) Response in y-direction during excitation in x-direction, (d) Response in x-direction during excitation in x-direction
curves measured in FVT1. The y-direction and x-direction are shown in Figs. 3(a) and 3(b). In FVT1, a shaker was placed on the roof floor of the model building. The resonance curves in Figs. 7(a) and 7(d) have clear double peaks. Since the model building has no backfill at Stage 3 and the model building is almost axisymmetric, the double peaks are mainly attributable to the characteristics of the gravelly layer beneath the model building.

FEATURES OF HLSST RESULTS

The results of velocity logging, earthquake observation and FVT1 indicate unique behavior of the gravelly layer and the model building as follows:

Phenomenon 1: Shear wave velocity measured by the velocity logging varies widely in the gravelly layer (Figs. 6(a) and 6(b)).

Phenomenon 2: Shear wave velocity of the gravelly layer identified by inversion analysis shows obvious azimuth dependency (Fig. 6(c)).

Phenomenon 3: Some resonance curves by FVT1 have clear double peaks (Figs. 7(a) and 7(d)).

The cause of these phenomena is discussed in the next part.

CAUSE OF AZIMUTH DEPENDENCY OF GRAVELLY LAYER

Morishita et al. (1993) and Yamaya et al. (1995) showed that the response orbit of the model structure during FVT1 can be divided into two components along two principal orthogonal axes, $D_1$, $D_2$. They also showed that the resonance curve for a principal axis has a single peak and that the effect of the orthogonal component is negligibly small. In the same manner as Ueshima and Okano (1996), if the strong axis is defined as the axis along which the maximum shear wave velocity is identified and the weak axis is defined as the axis along which the minimum shear wave velocity is identified, $D_1$-axis and $D_2$-axis are the weak axis and the strong axis, respectively. Thus, each resonance curve of FVT1 might have double peaks because resonance occurs at two frequencies corresponding to the two kinds of stiffness of the gravelly layer. The azimuth of $D_1$-axis is 105 (deg) east from magnetic north, while that of $D_2$-axis is 15 (deg) east from magnetic north (Morishita et al., 1993; Yamaya et al., 1995). The difference between the azimuth of $D_1$-axis and $U_1$-axis is 5 (deg), meaning that they are substantially the same; the azimuths of $D_2$-axis and $U_2$-axis are also substantially the same. This suggests that the phenomena 2 and 3 mentioned above are attributable to the same cause.

Morishita et al. (1993) and Yamaya et al. (1995) claimed that phenomenon 3 mentioned above is attributable to the effect of soil inhomogeneity at the surface of the excavated ground. However, the azimuth dependency of the gravelly layer is seen not only near G.L. $-5\, \text{m}$, which corresponds to the bottom of the concrete mat of the model building, but also throughout the gravelly layer, as shown in Fig. 6(c). Consequently, if phenomena 2 and 3 have a common cause, it is not the effect of soil inhomogeneity at the surface of the excavated ground, but the anisotropic nature of the overall gravelly layer.

Generally, anisotropy of soils is classified into stress-induced anisotropy and inherent anisotropy (Oda and Kazama, 1993). The former is caused by the anisotropic stresses acting on the soil, and the latter is caused by structural characteristics of the soil generated during sedimentation. If the azimuth dependency of the shear wave velocity in the gravelly layer is attributable to stress-induced anisotropy, the horizontal earth pressure at rest must vary with azimuth. However, since the ground surface at the HLSST site is almost horizontal, the horizontal earth pressure at rest is not thought to vary so much with azimuth.

Moreover, since the horizontal earth pressure of the gravelly layer just beneath the model building at Stage 3 is almost completely induced by the self-weight of the model building, the horizontal earth pressure at Stage 3 does not vary much with azimuth if the gravelly layer is not inherently anisotropic. Therefore, I conclude that the azimuth dependency of the shear wave velocity of the gravelly layer at Stage 3 is mainly induced by the inherent anisotropy of the gravelly layer.

If the bedding plane of the gravelly layer is horizontal, phenomena 2 and 3 should not occur because of the inherent anisotropy. Thus, the bedding plane of the gravelly layer at the HLSST site should not be horizontal.

As described previously, the gravelly layer at the HLSST site is called the Milun Formation of the Pleistocene origin. According to Ho (1988), the bedding plane in the Milun Formation is inclined to the horizontal by 30 (deg) at most.

Figure 8 shows an outcrop of the Milun Formation near the seashore about 2.0 km east of the HLSST site. Inclination of bedding planes can be seen. Since the angle of view of the photograph is the west, layers in the Milun Formation seems to be inclined to the south.

Undisturbed samples of the gravelly layer at G.L. $-5\, \text{m}$ to $-6\, \text{m}$ were obtained from the excavated ground by the in-situ freezing technique (Tanaka et al., 1998a). In this case, a clod of frozen soil about 1.5 m in diameter was lifted out of the ground and cores of 30-cm diameter were taken (Figs. 3(a) and 9). Figure 9 is a photograph of the clod of frozen soil. Figure 10(a) is a sketch of the gravel particles in Fig. 9. Moreover, Fig. 10(b) shows the results of statistical analysis of the inclination of the gravel particles in Fig. 10(a). Figure 10(b) shows that about 90% of the $\beta_0$ values are positive, meaning that the gravel particles are inclined southwest. Since the direction of the long axes of the gravel particles is thought to be parallel to the bedding plane, Fig. 10(b) means that the bedding plane of the gravelly layer is inclined to the horizontal plane.
MODELING ANISOTROPIC BEHAVIOR OF GRAVELLY LAYER AS ORTHOTROPIC ELASTIC BODY

Modeling Azimuth Dependency of Initial Shear Modulus as Orthotropic Elastic Body

Generally, an orthotropic elastic body has parallel planes on which mechanical characteristics are isotropic. Thus, the plane is named an isotropic plane in this paper. Assuming that the $x_1$ axis and $y_1$ axis are parallel to the isotropic plane and that the $z_1$ axis is perpendicular to the isotropic plane as shown in Fig. 11(a), the stress-strain relationships of an orthotropic body can be expressed as follows (Wardle and Gerrard, 1972):

$$
\begin{bmatrix}
\varepsilon_{x_1} \\
\varepsilon_{y_1} \\
\varepsilon_{z_1} \\
\gamma_{x_1y_1} \\
\gamma_{y_1z_1} \\
\gamma_{x_1z_1}
\end{bmatrix}
= [C_0]
\begin{bmatrix}
\sigma_{x_1} \\
\sigma_{y_1} \\
\sigma_{z_1} \\
\tau_{x_1y_1} \\
\tau_{y_1z_1} \\
\tau_{x_1z_1}
\end{bmatrix}
$$

(1a)

where,

$$
[C_0] =
\begin{bmatrix}
1 & -v_{nh} & -v_{nh} & 0 & 0 & 0 \\
\frac{v_{nh}}{E_h} & \frac{1}{E_h} & \frac{-v_{nh}}{E_h} & 0 & 0 & 0 \\
\frac{v_{nh}}{E_h} & \frac{-v_{nh}}{E_h} & \frac{1}{E_h} & 0 & 0 & 0 \\
0 & 0 & 0 & 1 \frac{1}{G_v} & 0 & 0 \\
0 & 0 & 0 & 0 \frac{1}{G_v} & 0 & 0 \\
0 & 0 & 0 & 0 & 0 \frac{1}{G_h}
\end{bmatrix}
$$

(1b)

$$
E_h = \frac{1}{2(1 + \nu_{x_1y_1})} \frac{1}{G_h}
$$

(1c)

Coordinate transformation was conducted on Eq. (1a). Then, the stress-strain relationships of the orthotropic elastic body at the $x$-$y$-$z$ coordinates shown in Fig. 11(b) were derived as follows:
where,

\[
\begin{bmatrix}
\varepsilon_{xx} \\
\varepsilon_{yy} \\
\varepsilon_{zz} \\
\gamma_{xy} \\
\gamma_{xz} \\
\gamma_{yz}
\end{bmatrix} = [C] \cdot 
\begin{bmatrix}
\sigma_{xx} \\
\sigma_{yy} \\
\sigma_{zz} \\
\tau_{xy} \\
\tau_{xz} \\
\tau_{yz}
\end{bmatrix}
\]  \hspace{1cm} (2a)

\[
[C] = 
\begin{bmatrix}
\gamma_{11} & \gamma_{12} & \gamma_{13} & \gamma_{14} & \gamma_{15} & \gamma_{16} \\
\gamma_{12} & \gamma_{22} & \gamma_{23} & \gamma_{24} & \gamma_{25} & \gamma_{26} \\
\gamma_{13} & \gamma_{23} & \gamma_{33} & \gamma_{34} & \gamma_{35} & \gamma_{36} \\
\gamma_{14} & \gamma_{24} & \gamma_{34} & \gamma_{44} & \gamma_{45} & \gamma_{46} \\
\gamma_{15} & \gamma_{25} & \gamma_{35} & \gamma_{45} & \gamma_{55} & \gamma_{56} \\
\gamma_{16} & \gamma_{26} & \gamma_{36} & \gamma_{46} & \gamma_{56} & \gamma_{66}
\end{bmatrix}
\]  \hspace{1cm} (2b)

Furthermore, \([D]\) matrix was defined as follows:

\[
[D] = [C]^{-1} = 
\begin{bmatrix}
d_{11} & d_{12} & d_{13} & d_{14} & d_{15} & d_{16} \\
d_{12} & d_{22} & d_{23} & d_{24} & d_{25} & d_{26} \\
d_{13} & d_{23} & d_{33} & d_{34} & d_{35} & d_{36} \\
d_{14} & d_{24} & d_{34} & d_{44} & d_{45} & d_{46} \\
d_{15} & d_{25} & d_{35} & d_{45} & d_{55} & d_{56} \\
d_{16} & d_{26} & d_{36} & d_{46} & d_{56} & d_{66}
\end{bmatrix}
\]  \hspace{1cm} (3)

Equations (2a) and (2b) express the stress-strain relationships of an orthotropic elastic body with inclined isotropic planes (Fig. 11(b)). Shear moduli, \(G_{xy}\) and \(G_{xz}\) in Fig. 11(b) are expressed as follows:

\[
\frac{1}{G_{xy}} = C_{44} = -\sin^2\xi \cdot A + \frac{\sin^2\beta + \cos^2\beta}{G_h + G_v}
\]  \hspace{1cm} (4a)

\[
\frac{1}{G_{xz}} = C_{55} = -\cos^2\xi \cdot A + \frac{\sin^2\beta + \cos^2\beta}{G_h + G_v}
\]  \hspace{1cm} (4b)

where,

\[
A = \frac{\sin^2\beta \cdot \cos^2\beta}{G_h + G_v} - \frac{(\cos^2\beta - \sin^2\beta)^2}{G_h + G_v} - 4 \cos^2\beta \cdot \sin^2\beta \cdot \left(\frac{1 + 2\gamma_{vh}}{E_v} + \frac{1}{E_h}\right).
\]  \hspace{1cm} (4c)

According to Eqs. (4a) and (4b), \(G_{xy}\) and \(G_{xz}\) have extreme values when \(\xi = 0\) or \(\xi = 90\) as follows:

\[
\frac{1}{G_{xy}(\xi = 0^\circ)} = \frac{1}{G_{xy}(\xi = 90^\circ)} = \frac{\sin^2\beta}{G_h + G_v} + \frac{\cos^2\beta}{G_h + G_v}
\]  \hspace{1cm} (5a)

\[
\frac{1}{G_{xz}(\xi = 0^\circ)} = \frac{1}{G_{xz}(\xi = 90^\circ)} = \frac{\sin^2\beta + \cos^2\beta}{G_h + G_v} - A.
\]  \hspace{1cm} (5b)

When one of the right hand side of Eqs. (5a) and (5b) is maximum, the other is minimum. Whether Eq. (5a) corresponds to maximum or minimum depends on the sign of \(A\) in Eq. (5b). Thus, it is necessary to determine the sign of \(A\).

When one of the right hand side of Eqs. (5a) and (5b) is maximum, the other is minimum. Whether Eq. (5a) corresponds to maximum or minimum depends on the sign of \(A\) in Eq. (5b). Thus, it is necessary to determine the sign of \(A\).

According to Fig. 10(b), the values of \(\beta_0\) are in the region of 24 (deg) ~ 35 (deg) when the percent finer by frequency, which corresponds to the vertical axis, is in the region of 40% ~ 60%. Figure 9 is a photograph taken about 50 (deg) west of magnetic north, which is 30 (deg) east of the \(U_1\)-axis (Fig. 3(a)). If the bedding plane corresponds to the isotropic plane of the orthotropic elastic body, \(\beta\) can be calculated from \(\beta_0\) when the azimuth of the inclination of the bedding plane is assumed. The calculated \(\beta\) is in the region of 27 (deg) ~ 39 (deg) assuming that the bedding plane is inclined to the \(U_2\)-axis, and in the region of 42 (deg) ~ 55 (deg) assuming that the bedding plane is inclined to the \(U_3\)-axis. Since the bedding plane in the Milun Formation is said to be inclined to the horizontal plane by 30 (deg) at most (Hö, 1988), it seems reasonable to assume that the bedding plane is inclined to the \(U_2\)-axis. In this case, the following equations are obtained.

\[
A > 0
\]  \hspace{1cm} (6a)

\[
\frac{1}{G_{\min}} = \frac{\sin^2\beta + \cos^2\beta}{G_h + G_v}
\]  \hspace{1cm} (6b)

\[
\frac{1}{G_{\max}} = -A + \frac{1}{G_{\min}}
\]  \hspace{1cm} (6c)

Furthermore, Eqs. (5a) and (5b) can be rewritten using Eqs. (6a) and (6b).
Determination of $\beta$

As described previously, the value of $\beta$ is possibly in the region of 27 (deg) - 39 (deg). Thus, $\beta = 30, 35$ and 40 (deg) is assumed for the calculation described later in this paper.

Determination of $v_{bh}$ and $v_b$

In dynamic problems, saturated sandy soils and gravelly soils of the HLSST site seem to deform under undrained condition. Thus, it would be an appropriate approximation to assume that volumetric strains of soils, $\varepsilon_v$, are constantly zero. When $\sigma_x$ is applied to the orthotropic elastic body, the following equation is derived by assuming that $\varepsilon_v = 0$ in Eqs. (1a) and (1b).

$$v_{bh,0} = 0.5$$

Similarly, when $\sigma_x$ or $\sigma_y$ is applied to the orthotropic elastic body, the following equation is derived by assuming that $\varepsilon_v = 0$ in Eqs. (1a) and (1b).

$$v_{b,0} = 1 - \frac{1}{2} \frac{E_b}{E_v}$$

In reality, the values of $v_{bh}$ and $v_b$ are smaller than those calculated by Eqs. (8) and (9) because of the compressibility of pore fluid and soil particles. Moreover, if $\varepsilon_v = 0$ is assumed, hindrance will occur during numerical simulation analysis of FVT1 which will be described later in this paper.

In engineering practice, assuming that the elastic deformation characteristics of the ground are isotropic, Poisson's ratio of the ground is calculated by $v_i$ and $v_p$. Thus, in this paper, assuming $v_i = 0.35$ m/s and $v_p = 1.620$ m/s, which are average values obtained by in-situ velocity logging, Poisson's ratio is calculated to be 0.48. Thus $v_{bh}$ is assumed to be 0.48, while $v_b$ is calculated by multiplying 0.96 (= 0.48/0.5) by $v_{bh,0}$.

Determination of $G_i$ and $G_b$

Substituting the values of $\beta$, $E_b/E_v$, $v_{bh}$, $v_b$, $G_{max}$ and $G_{min}$ into Eqs. (6b) and (6c), $G_i$ and $G_b$ are calculated from Eqs. (6b), (6c) and (1c).

List of Cases of Parametric Analyses

Table 1 is a list of cases of parametric analyses conducted in this paper.

**COMPARISON BETWEEN EXPERIMENTAL RESULTS AND CALCULATED RESULTS USING THE ORTHOTROPIC ELASTIC BODY**

**Shear Wave Velocity by the Velocity Logging**

Shear wave velocity measured by the cross-hole method and the down-hole method is calculated theoretically using the orthotropic elastic model.

Generally, the velocity of plane waves, $v$, can be calculated as solutions of the following equation (Kolsky, 1963).
Table 1. Cases for parametric analyses

<table>
<thead>
<tr>
<th>Cases</th>
<th>Assumed values</th>
<th>Values determined by inversion analyses</th>
<th>Calculated values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\beta$ (deg)</td>
<td>$E_0/E_0$</td>
<td>$r$</td>
</tr>
<tr>
<td>Case 1</td>
<td>30</td>
<td>0.5</td>
<td>0.96</td>
</tr>
<tr>
<td>Case 2</td>
<td>30</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Case 3</td>
<td>35</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>Case 4</td>
<td>35</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Case 5</td>
<td>40</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>Case 6</td>
<td>40</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

$$0 = \left( B_1 - \rho v^2 \right) \left( B_4 - B_5 \right)$$

\[
B_1 = d_{11} l^2 + d_{66} m^2 + d_{55} n^2 + 2 d_{16} m l + 2 d_{15} m n + 2 d_{15} n l \quad (10b)
\]

\[
B_2 = d_{22} l^2 + d_{22} m^2 + d_{44} n^2 + 2 d_{24} m l + 2 d_{24} m n + 2 d_{24} n l \quad (10c)
\]

\[
B_3 = d_{33} l^2 + d_{44} m^2 + d_{55} n^2 + 2 d_{34} m l + 2 d_{34} m n + 2 d_{34} n l \quad (10d)
\]

\[
B_4 = d_{23} m^2 + d_{23} n^2 + (d_{12} + d_{66}) l m + (d_{13} + d_{66}) l n + (d_{15} + d_{66}) m n \quad (10e)
\]

\[
B_5 = d_{23} m^2 + d_{23} n^2 + (d_{12} + d_{66}) l m + (d_{15} + d_{66}) m n + (d_{13} + d_{66}) l n + (d_{15} + d_{66}) m n \quad (10f)
\]

Equation (10a) is a cubic equation for $\rho v^2$. Equation (10a) has three positive roots for any elastic solid. In general, these three roots are different from each other and correspond to three distinct plane waves. Moreover, when the three positive roots are different from each other, the vibration directions of the three distinct plane waves are mutually perpendicular (Kolsky, 1963).

When a plane wave propagates along the x-axis in Fig. 11(b), Eq. (10a) has three distinct roots. One of the roots corresponds to a compressional wave (P-wave) whose predominant direction of vibration is the x-direction. The other two roots correspond to two shear waves. In cross-hole logging, the shear waves propagate along the x-axis with vertical vibration. Thus, shear wave velocity measured by cross-hole logging corresponds to $v_{\text{s,CHL}}$.

Figures 13(a), 13(b) and 13(c) show the relationships between $v_{\text{s,CHL}}$ and $\xi$. In Figs. 13(a), 13(b) and 13(c), solid lines and dashed lines denote the relationships between calculated $v_{\text{s,CHL}}$ and $\xi$. In Figs. 13(b) and 13(c), measured $v_{\text{s,CHL}}$ agrees with calculated $v_{\text{s,CHL}}$ when $\xi$ is about 150 (deg), whereas measured $v_{\text{s,CHL}}$ is larger than calculated $v_{\text{s,CHL}}$ when $\xi$ is about 40 (deg) or about 110 (deg). Nevertheless, the overall trend of the measured $v_{\text{s,CHL}}$ can be expressed theoretically in Figs. 13(b) and 13(c). In contrast to Figs. 13(b) and 13(c), calculated values of $v_{\text{s,CHL}}$ of Cases 1 and 2 in Fig. 13(a) are much larger than measured values when $\xi$ is about 40 (deg) or about 150 (deg).

In the down-hole method, a plane wave propagates along the z-axis in Fig. 11(b). In that case, Eq. (10a) has three distinct roots. One of the roots corresponds to a compressional wave (P-wave) whose predominant vibration direction is the z-direction. The other two roots correspond to two shear waves, $v_{\text{s,DLH}}$ and $v_{\text{s,DLH}}$. The values of $v_{\text{s,DLH}}$ or $v_{\text{s,DLH}}$ are plotted in Fig. 14 together with the measured results. A closer look at Fig. 14 seems to reveal that the measured values of shear wave velocity seem to be divided into two groups: data of about 330 m/s and datum of about 390 m/s. The former value and the latter value seem to correspond $v_{\text{s,DLH}}$ or $v_{\text{s,DLH}}$ respectively. In Cases 3, 4, 5 and 6, the calculated $v_{\text{s,DLH}}$ values agree well with measured values, while the calculated $v_{\text{s,DLH}}$ values are smaller than the measured values. In Fig. 14, all the calculated values of $v_{\text{s,DLH}}$ of Cases 1 and 2 are much larger than measured values.

Table 1 shows that the $G_0/G_r$ values of Cases 1 and 2 are 6.96 and 13.92, respectively. These values are larger than those of Cases 3, 4, 5 and 6. According to a literature survey, the maximum measured value of $G_0/G_r$ of sands and clays is 3, which is smaller than the values in Cases 1, 2, 4, and 6. However, according to the inversion analysis of the gravelly layer from G.L. - 5 m to - 12 m, the ratio of maximum shear wave velocity to minimum shear wave velocity is in the region of 1.5 - 1.7 (Fig. 12). Thus, the ratio of maximum shear modulus to minimum shear modulus, $G_{\text{max}}/G_{\text{min}}$, is in the region of 2.2 - 2.9. Considering that the bedding plane of the gravelly layer is inclined to the horizontal plane, it is not unreasonable to assume that the value of $G_0/G_r$ is larger than 2.2 - 2.9. Thus, it can not be said that the orthotropic elastic model is unreasonable because the values of $G_0/G_r$ in Table 1
are larger than 3. However, in this paper, because the parameters of Cases 1 and 2 do not seem to be correct in that the values of $G_h/G_v$ are too large, these parameters are not used for calculation in the later part of this paper.

Though the calculated results of Cases 3, 4, 5 and 6 in Figs. 13(b), 13(c) and 14 agree approximately with the measured results, a closer look at Fig. 13(b), 13(c) and 14 seems to reveal the following facts:

i) Calculated values of $v_{s,CH}$ of Cases 3, 4, 5 and 6 are discernibly smaller than measured values by the cross-hole method.

ii) Calculated values of $v_{s,DM}$ of Cases 3, 4, 5 and 6 are discernibly smaller than corresponding measured values by the down-hole method.

The cause of i) and ii) seems to be attributable to the effect of heterogeneity in the gravel layer, as pointed out previously (Tanaka, 2000). Furthermore, the scattering of shear wave velocity by the cross-hole method, as shown in Figs. 12 and 14, might also be attributable to the effect of heterogeneity in the gravel layer.

Initial Shear Modulus by Cyclic Triaxial Tests

The initial shear modulus obtained by the cyclic triaxial test can be expressed as follows:

$$
\frac{1}{G_{o,ini}} = \frac{1}{E_a} = 3 \cdot \frac{\varepsilon_a}{\sigma_a} = 3 C_{33} \cdot \frac{6 \cos^2 \beta \cdot (\cos^2 \beta - \sin^2 \beta) \cdot (1 - v_h) + 3 \sin^4 \beta}{2(1 + v_h)} \times \frac{1}{G_h} + \frac{3 \cos^2 \beta \cdot \sin^2 \beta}{G_v} \quad (11)
$$

By rewriting Eq. (11), $E_a$ can be expressed as follows:

$$
\frac{1}{E_a} = \frac{\varepsilon_{a1} + \varepsilon_{a2}}{\sigma_a} \quad (12)
$$

where,

$$
\frac{\varepsilon_{a1}}{\sigma_a} = \frac{2 \cos^2 \beta \cdot (\cos^2 \beta - \sin^2 \beta) \cdot (1 - v_h) + \sin^4 \beta}{2(1 + v_h) \cdot G_h} \quad (13a)
$$
Fig. 15. End resistant effect of specimen on results of cyclic triaxial test

\[
\varepsilon_{e2} = \frac{\sin^2 \beta \cdot \cos^2 \beta}{\sigma_a G} 
\]

(13b)

According to Eq. (13b), \( \varepsilon_{e2} \) approaches zero when \( G_c \) is sufficiently large. This means that \( \varepsilon_{e2} \) is induced by the shear deformation along the bedding planes as shown in Fig. 15. However, shear deformation along the bedding planes will not occur in the black parts in Fig. 15 because the upper or lower end of the black parts is in direct contact with the cap or pedestal. This is called the end restraint effect herein. Consequently, it is necessary to consider the end restraint effect when calculating \( \varepsilon_{e2} \) precisely.

Considering the end restraint effect, Eq. (13b) is rewritten as follows:

\[
\varepsilon_{e2} = \left(1 - \frac{D}{H} \cdot \tan \beta\right) \frac{\sin^2 \beta \cdot \cos^2 \beta}{G_c} \cdot \sigma_a
\]

(14)

Substituting Eqs. (13a) and (14) into Eqs. (12) and (11), the following equation is derived.

\[
\frac{1}{G_{b,ini,2}} = 3 \cdot \frac{1}{E_a} = 3 \cdot \frac{\varepsilon_{a1} + \varepsilon_{e2}}{\sigma_a} = 6 \cos^2 \beta \cdot (\cos^2 \beta - \sin^2 \beta) \cdot (1 - \nu_a) + 3 \sin^2 \beta
\]

\[
\times \frac{1}{2(1 + \nu_a) \cdot G_h} + \left(1 - \frac{D}{H} \cdot \tan \beta\right) \frac{3 \sin^2 \beta \cdot \cos^2 \beta}{G_c}
\]

(15)

The shear wave velocity is converted from \( G_{b,ini,2} \) of Eq. (15) by the following equation:

\[
v_{s,ini,2} = \sqrt{G_{b,ini,2} / \rho_s}.
\]

(16)

Values of \( v_{s,ini,2} \), which are calculated using the values of parameters of Cases 3, 4, 5, and 6 in Table 1, are plotted in Fig. 16.

In Fig. 16, measured shear wave velocity by cyclic triaxial tests is also plotted. In the cyclic triaxial tests, all the specimens are consolidated isotropically. The effective confining pressure of specimens of solid circles in Fig. 16 is equal to the effective overburden pressure. Thus, \( K_0 \) is assumed to be 1.0; as mentioned previously, in this paper \( K_0 \) is assumed to be in the region of 0.5 ~ 1.0. Thus, the shear wave velocity of \( K_0=0.5 \) is calculated by modifying the shear wave velocity of \( K_0=1.0 \) in terms of effective confining pressure. According to the results of cyclic triaxial tests as well as shear wave velocity tests (Tanaka et al., 2000), the shear wave velocity of gravelly soil samples is approximately proportional to the 0.3th power of the effective confining pressure. Thus, assuming that shear wave velocity depends on mean effective stress, the shear wave velocity of \( K_0=0.5 \) is obtained by multiplying the shear wave velocity of \( K_0=1.0 \) by \((1+2K_0)/3)^{0.3}\). The shear wave velocity of \( K_0=0.5 \) is also plotted in Fig. 16 as open circles.

Figure 16 indicates that the calculated results \( v_{s,ini,2} \) of Cases 3, 4, 5, and 6 agree with open circles and solid circles on average, whereas measured values have a scattering, which is thought to be attributable to variation in the grain size distributions of triaxial specimens, as described in the literature (Tanaka et al., 2000).

Dynamic Response in Forced Vibration Tests

3-dimensional simulation analyses of the FVT1 were conducted herein by a computer code "ABAQUS" (Hibbit, Karlsson and Sorensen, Inc., 1999) using the orthotropic elastic model described previously.

Figure 17 shows the mesh of 3-dimensional simulation analysis on FVT1. In FVT1, waves generated at the bottom of the model building propagate in the ground in all directions. The solid infinite elements (Hibbit, Karlsson and Sorensen, Inc., 1999), which can model the far field region to reduce the effect of horizontal reflection wave from the lateral boundary, are placed in connection with finite elements at the lateral fringe of the
model ground. The bottom of the model is fixed horizontally and vertically. In the ABAQUS, dynamic response analysis is conducted in time domain.

Figure 18 and Table 2 show the properties of soils for the simulation analysis of FVT1. The sandy layer is modeled by the isotropic elastic body, while the gravelly layer is modeled by the orthotropic elastic body.

Values of $v_s$ and $v_p$ of the gravelly layers, Gravel 2, 3, and 4 are determined based on the results of the inversion analysis conducted at Stage 1. It is assumed that $\beta=35$ (deg) and $E_s/E_r=1$ for Gravel 2, 3 and 4, which are identical to the values used in Case 4 in Table 1.

According to the results of an elastic finite element analysis, the mean effective stresses of Gravel 1 and Gravel 2 at Stage 3 are approximately twice as large as that at Stage 1 (Okamoto et al., 1995a; Kokusho et al., 1997). As mentioned previously, shear wave velocity of gravelly soil samples is proportional to the 0.3th power of the effective confining pressure. Thus, $v_{s,1}$ and $v_{p,1}$ of Gravel 1 at Stage 3 are obtained by multiplying those at Stage 1 by $2^{0.3}$.

As shown in Table 2, the damping ratio which is used for the simulation analyses of FVT1 is 2%, which is

<table>
<thead>
<tr>
<th>Soils</th>
<th>Stress-strain model</th>
<th>Parameters</th>
<th>Free field (Stage 1)</th>
<th>FVT1 (Stage 3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand 1</td>
<td>Isotropic elastic model</td>
<td>$V_s$ (m/s)</td>
<td>133</td>
<td>133</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$h$ (%)</td>
<td>5</td>
<td>2</td>
</tr>
<tr>
<td>Sand 2</td>
<td>Isotropic elastic model</td>
<td>$V_s$ (m/s)</td>
<td>231</td>
<td>231</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$h$ (%)</td>
<td>5</td>
<td>2</td>
</tr>
<tr>
<td>Gravel 1</td>
<td>Orthotropic elastic model</td>
<td>$E_s/E_r$</td>
<td>1</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$V_s$ (m/s)</td>
<td>343</td>
<td>422</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$V_p$ (m/s)</td>
<td>216</td>
<td>266</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$h$ (%)</td>
<td>5</td>
<td>2</td>
</tr>
<tr>
<td>Gravel 2</td>
<td>Orthotropic elastic model</td>
<td>$E_s/E_r$</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$V_s$ (m/s)</td>
<td>343</td>
<td>343</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$V_p$ (m/s)</td>
<td>216</td>
<td>216</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$h$ (%)</td>
<td>5</td>
<td>2</td>
</tr>
<tr>
<td>Gravel 3</td>
<td>Orthotropic elastic model</td>
<td>$E_s/E_r$</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$V_s$ (m/s)</td>
<td>433</td>
<td>433</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$V_p$ (m/s)</td>
<td>299</td>
<td>299</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$h$ (%)</td>
<td>5</td>
<td>2</td>
</tr>
<tr>
<td>Gravel 4</td>
<td>Orthotropic elastic model</td>
<td>$E_s/E_r$</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$V_s$ (m/s)</td>
<td>467</td>
<td>467</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$V_p$ (m/s)</td>
<td>320</td>
<td>320</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$h$ (%)</td>
<td>5</td>
<td>2</td>
</tr>
<tr>
<td>Gravel 5 and Gravel 6</td>
<td>Orthotropic elastic model</td>
<td>$E_s/E_r$</td>
<td>1</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$V_s$ (m/s)</td>
<td>343</td>
<td>182</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$V_p$ (m/s)</td>
<td>216</td>
<td>114</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$h$ (%)</td>
<td>5</td>
<td>2</td>
</tr>
</tbody>
</table>

![Fig. 17. Mesh for numerical simulation analysis of FVT1 at Stage 3](image)

![Fig. 18. Soil model for simulation analysis of FVT1 at Stage 3](image)
based on the results of the cyclic triaxial test (Tanaka et al., 1998a; Tanaka et al., 2000).

$K_0$ of the gravelly layer just beneath the model building at Stage 3 seems smaller than that at Stage 1 because of the effect of large vertical stress induced by the model building. Thus, $E_s/E_r$ of the gravelly layer just beneath the model building at Stage 3 seems smaller than that at Stage 1. As mentioned previously, $E_s/E_r$ of the gravelly layer at Stage 1 seems in the region of 0.4 ~ 1.0. Considering that $E_s/E_r$ at Stage 3 is smaller than that at Stage 1, $E_s/E_r=0.2$ is assumed for Gravel 1, Gravel 5 and Gravel 6.

According to the results of the large penetration tests (LPT) conducted at Stage 4, LPT $N_r$-values of the gravelly layer near the edge of the concrete mat of the model building at Stage 4 are smaller than those at Stage 1 (Okamoto et al., 1995b; Kokusho et al., 1997). This might be attributable to the effect of disturbance on the gravelly soil accompanied by model construction (Okamoto et al., 1995b; Kokusho et al., 1997). Thus, $v_s$, $v_r$, $v_o$ of Gravel 5 and Gravel 6 are reduced considering a reduction in LPT $N_r$-values to 53% of those of Gravel 2 in order that calculated peak frequencies coincide with those by measurement.

The experimental results of FVT1 are compared in Figs. 7(a), 7(b), 7(c) and 7(d) with their simulated results. It can be pointed out that the resonance curves are well simulated. Especially, the whole shape and the double peaks of the simulated curves are very similar to the experimental results.

CONCLUSIONS

The following conclusions are drawn through this study.

1) Modeling anisotropic behavior of gravelly layer as orthotropic elastic body

According to the results of identification of shear wave velocity by inversion analysis, the gravelly layer at the HLSST site has obvious azimuth dependency in shear wave velocity.

The bedding plane in the Milun Formation to which the gravelly layer belongs is reportedly inclined to the horizontal plane by 30 (deg) at most. Moreover, the inclination of the bedding plane was also observed in the photograph of the clod of frozen soil sampled by the in-situ freezing technique. Thus, the stress-strain relationships of the gravelly soil were modeled as an orthotropic elastic body, assuming that the azimuth dependency of the shear wave velocity is attributable to the inclination of the bedding plane to the horizontal plane.

2) Comparison between experimental results and results calculated using the orthotropic elastic model

1) Shear wave velocity by cross-hole velocity logging

The azimuth dependency of shear wave velocity observed both in the results of inversion analyses and velocity loggings by the cross-hole method is expressed by the orthotropic elastic model.

2) Shear wave velocity by the down-hole velocity logging

It can be explained by the orthotropic elastic body model that values of shear wave velocity measured by down-hole velocity logging are divided into two groups.

3) Comparison between experimental results by triaxial tests and calculated results

The shear wave velocity converted from the initial shear modulus using the cyclic triaxial test results was calculated using the orthotropic elastic model. The calculated results agree with the experimental results.

4) Comparison between results of FVT1 and simulated results

The resonance curves of the model building at FVT1 can be successfully simulated by the orthotropic elastic model.

Given the facts (1) and (2) mentioned above, it is revealed that the behavior of the gravelly layer in the HLSST site can be modeled by the orthotropic elastic body. This will enhance the accuracy of the dynamic simulation analysis.

ACKNOWLEDGMENTS

I would like to thank to Prof. T. Kokusho (Chuo University), Dr. K. Nishi (CRIEPI), Dr. T. Ueshima (CRIEPI) and Dr. T. Okamoto (CRIEPI) for their advice as well as their efforts of promoting the HLSST. I am grateful to Mr. Kataoka (CRIEPI), Mr. K. Sakai (Kisojiban Consultants Co., Ltd.) and his colleagues for conducting successful sampling by in-situ freezing technique. I am also grateful to Mr. K. Kudo (CRIEPI) for analyzing the results of the in-situ tests and laboratory tests. I thank Dr. Shin (CRIEPI) for conducting a theoretical calculation of wave velocities of the orthotropic elastic body. I thank Mr. Nakazono (D.C.C. Co., Ltd.) for conducting dynamic response analysis of the model building. I also thank Mr. K. Seo (C.R.S. Co., Ltd.) for conducting laboratory tests.

NOTATION

$E_s$: Undrained Young’s modulus when $\sigma_n$ or $\sigma_r$ is applied solely

$E_r$: Undrained Young’s modulus when $\sigma_r$ is applied solely

$n_s$: Undrained shear modulus when $\tau_{n,n}$ or $\tau_{n,r}$ is applied solely

$n_r$: Undrained shear modulus when $\tau_{n,r}$ is applied solely

$v_r$: Poisson’s ratio defined as $-\varepsilon_r/\varepsilon_t$ when $\sigma_n$ or $\sigma_r$ is applied solely under undrained condition

$v_t$: Poisson’s ratio defined as $-\varepsilon_r/\varepsilon_t$ or $-\varepsilon_r/\varepsilon_t$ when $\sigma_n$ or $\sigma_r$ is applied solely under undrained condition

$G_{s,s}$: Undrained shear modulus when $\tau_{n,n}$ is applied solely

$G_{s,r}$: Undrained shear modulus when $\tau_{n,r}$ is applied solely

$\beta$: Angle between long axes of gravel particles and horizontal steel bar in Fig. 9

$\beta$: Angle between horizontal plane and isotropic plane, i.e., bedding plane in Fig. 11(b)

$\xi$: Angle between direction of inclination of isotropic planes and $x$-axis (see Fig. 11(b))

$G_{max}$: Maximum of $G_{n,n}$ and $G_{n,r}$

$G_{min}$: Minimum of $G_{n,n}$ and $G_{n,r}$
$E_{\theta\phi}$: Drained Young’s modulus when normal stress applied parallel to isotropic plane, i.e., bedding plane

$E_{\phi\phi}$: Drained Young’s modulus when normal stress applied perpendicular to isotropic plane, i.e., bedding plane

$\varphi$, $\theta$: Cosines of angles between direction of propagating plane wave and $x$-axis, $y$-axis and $z$-axis, respectively

$\varrho_{\phi 0}$, $\varrho_{\theta 0}$ under $\varphi = 0$ condition

$\varrho_{\phi 0}$, $\varrho_{\theta 0}$ under $\varphi = \vartheta = 0$ condition

$\sigma$: Axial stress in triaxial test

$\varepsilon$: Axial strain in triaxial test

$\rho$: Wet density

$V_{ph}$, $V_{p2}$: Shear wave velocities of SH waves whose predominant vibration directions are $s$-direction and $t$-direction, respectively

$V_{p,CHI}$: Shear wave velocity by cross-hole velocity logging whose predominant vibration direction of vibration is $z$-direction

$V_{p,DL}$, $V_{p,DL2}$: Shear wave velocities by down-hole velocity logging whose predominant vibration directions are $s$-direction and $t$-direction, respectively

$G_{DNL}$: Initial shear modulus evaluated by results of cyclic triaxial test without considering end restraint effect

$G_{DNL2}$: Initial shear modulus evaluated by results of cyclic triaxial test considering end restraint effect

$V_{s,ML}$: Shear wave velocity converted from $G_{DNL2}$

REFERENCES


