Amplification characteristics at artificial Island considering viscoelastic effects of cohesive soil

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The aim of the present paper was to perform a one-dimensional finite element analysis of layered ground according to a cyclic viscoelastic-viscoplastic theory. In order to estimate the viscous effects of clay over a wide range of strain levels, a viscoelastic-viscoplastic constitutive model for clay was proposed. First, we confirmed the performance of the proposed model by simulating cyclic undrained triaxial tests to determine the cyclic strength and deformation characteristics of natural marine clay. The proposed model was then incorporated into an effective stress-based amplification characteristics analysis to estimate the effects of an intermediate clay layer on the behavior of sand layers. The results show that a cyclic viscoelastic-viscoplastic constitutive model can provide a good description of dynamic behavior including viscoelastic effects, within a small strain range.

Key words: Constitutive model, amplification characteristics, viscoelastic, viscoplastic, clay.

INTRODUCTION

Interest in liquefaction has been on the rise since liquefaction was observed during the Good Friday earthquake of 1964 in Alaska and during the Niigata earthquake in Japan. Earthquake acceleration was first measured for relevant engineering research during the El Centro earthquake in 1940, and the severity of earthquake damage, including liquefaction phenomena, has been widely investigated in earthquakes such as the Michoacan earthquake in 1985 in Mexico, the Loma Prieta and Northridge earthquakes in 1989 and 1994, respectively, in California, USA, and the Hyogoken Nanbu earthquake in 1995 in Japan.

The main elements considered in establishing earthquake-resistant measures to ensure the safety of structures from earthquake damage are as follows: for a large earthquake, the behaviors of sandy soil and clay soil deposits due to a seismic wave are dominated by their plastic properties, and the elasto-plastic model or elasto-viscoplastic model is applied to their subsequent composition models. However, in medium- and small-earthquakes, it is important to apply a model that can accurately explain the viscoelastic damping characteristics of clay soils in multi-layered ground.

The linear viscoelastic approach is valid for behavior in small strain ranges, while a viscoplastic modelling of soils is useful for large strains. In this study, a viscoelastic-viscoplastic (VE-VP) constitutive model based the non-linear kinematic hardening rule is proposed; this model incorporates three parameters from a viscoelastic model based on the Maxwell model and the Voigt model. However, in the proposed model, a three-parameter model is adopted based on the results from other studies (Hori, 1974; Murayama, 1983; Di Benedetto and Tatsuoka, 1997).

approach initially advocated by Armstrong and Frederick (1966) and later developed by Chaboche and Rousselier (1983). Kim (2001) suggested a viscoelastic-viscoplastic (VE-VP) constitutive model for clay based on a non-linear kinematic hardening rule. In the model, the generalized non-associated flow rule and the concept of over consolidated boundary surface within the context of infinitesimal strain was used.

This study performed an amplification characteristics analysis on multiple soil layers using the earthquake that occurred in Kobe, Japan (seismic magnitude of 4-5) as input data and reviewed the feasibility of the cyclic VE-VP constitutive model. For applying the suggested model to multiple soil layers, sandy and clayey soil deposits were represented by an E-VP model and a VE-VP model, respectively.

**A VISCOELASTIC-VISCOPLASTIC CONSTITUTIVE MODEL FOR CLAY**

Chaboche and Rousselier (1983) proposed a constitutive model for metal using concepts of a non-linear kinematic hardening rule. In the present study, a cyclic VE-VP constitutive model for clay is proposed using concepts of non-linear kinematic hardening as follows.

**Viscoelastic model**

The strain-rate component tensor consists of a deviatoric strain-rate component tensor $\dot{\varepsilon}$ and a volumetric strain-rate component tensor $\dot{\varepsilon}_{kk}$. A dot $\cdot$ denotes time differentiation.

$$\dot{\varepsilon}_{ij} = \dot{\varepsilon} + \frac{1}{3} \dot{\varepsilon}_{kk} \delta_{ij} \quad (1)$$

where $\delta_{ij}$ is the Kronecker delta.

Moreover, it can be assumed that the strain-rate tensor $\dot{\varepsilon}$ consists of a viscoelastic strain-rate tensor $\dot{\varepsilon}^{ve}$ and a viscoplastic strain-rate tensor $\dot{\varepsilon}^{vp}$; the deviatoric strain-rate thus decomposes into

$$\dot{\varepsilon}_{ij} = \dot{\varepsilon}_{ij}^{ve} + \dot{\varepsilon}_{ij}^{vp} \quad (2)$$

In the viscoelastic region, a three-parameter model with a Voigt element and an elastic spring was adopted, and the behavior of viscoelastic materials under uni-axial stress closely resembles that of models built from discrete elastic and viscous elements.

The deviatoric elastic strain-rate tensor is

$$\dot{\varepsilon}_{ij}^{e} = \frac{1}{2G_1} \dot{S}_{ij} \quad (3)$$

where $G_1$ is the first shear modulus, $S_{ij}$ is the deviatoric stress tensor $(\sigma_{ij} - \sigma_{0}\delta_{ij})$, $\sigma_{0}$ is the effective stress tensor and $\sigma_{0}$ is the mean effective stress.

The deviatoric stress tensor can be written as

$$S_{ij}^{v} = S_{ij}^{e} + S_{ij}^{e(v)} \quad (4)$$

From the viscous-flow rule, the viscous deviatoric stress tensor is

$$\dot{\varepsilon}_{ij}^{v} = \mu \cdot \dot{\varepsilon}_{ij}^{v} \quad (5)$$

where $\mu$ is the viscosity coefficient.

The elastic deviatoric stress tensor in the Voigt model is the same as that for a free spring which is the first elastic deviatoric stress tensor, so that

$$S_{ij}^{e(v)} = 2G_2 \dot{\varepsilon}_{ij}^{e(v)} \quad (6)$$

where $G_2$ is the second shear modulus. The deviatoric Voigt viscoelastic strain-rate tensor can be obtained as

$$\dot{S}_{ij}^{ve} = \frac{1}{\mu} (S_{ij}^{e} - 2G_2 \dot{\varepsilon}_{ij}^{e(v)}) \quad (7)$$

Therefore, the viscoelastic deviatoric strain-rate tensor $\dot{\varepsilon}_{ij}^{ve}$ consists of elastic and viscoelastic Voigt components as

$$\dot{\varepsilon}_{ij}^{ve} = \dot{\varepsilon}_{ij}^{e} + \dot{\varepsilon}_{ij}^{ve} = \frac{1}{2G_1} \dot{S}_{ij}^{e} + \frac{1}{\mu} (S_{ij}^{e} - 2G_2 \dot{\varepsilon}_{ij}^{e(v)}) \quad (8)$$

where $\dot{\varepsilon}_{ij}^{e}$ and $\dot{\varepsilon}_{ij}^{ve}$ denote the elastic and Voigt viscoelastic deviatoric strain-rate tensor. The total deviatoric strain-rate tensor is expressed as $\dot{\varepsilon}_{ij} = \dot{\varepsilon}_{ij}^{e} + \dot{\varepsilon}_{ij}^{vp}$. The stress rate tensor is given by

$$\dot{\varepsilon}_{ij}^{e} = \dot{\varepsilon}_{ij}^{e} + \dot{\varepsilon}_{ij}^{ve} \quad (9)$$

Here the Voigt viscoelastic volumetric strain-rate is considered to be zero ($\dot{\varepsilon}_{ij}^{ve} = 0$) due to the fact that it is difficult to distinguish between the viscoelastic behavior of the soil skeleton and pseudo-viscoelastic behavior due to interactions between water and soil. Therefore, the viscoelastic strain-rate tensor $\dot{\varepsilon}_{ij}^{ve}$ is given by
Viscoplastic model

The viscoplastic strain-rate tensor is expressed as follows (Oka, 1992; Oka et al., 1999):

$$\dot{\varepsilon}_{ij}^{vp} = C_0 \Phi(f) \frac{\dot{\sigma}_i^p - \dot{\tau}_i^p}{\sigma_i^p} + C_2 \Phi(f) \frac{\dot{\tau}_i^p}{\tau_i^p}$$

$$\eta^* = \sqrt{\eta_0^*}$$

$$\eta^* = \sqrt{(\eta_0^* - \eta_{ijkl}^p)(\eta_0^* - \eta_{ijkl}^p)}$$

$$\eta_0^* = \frac{S_i}{\sigma_m}$$

where $C_0$ and $C_02$ are viscoplastic parameters and the Macauley bracket $< >$ indicates that $<x> = x$ if $x > 0$, and $<x> = 0$ if $x \leq 0$. The static yield function $f_y$ and the viscoplastic parameters $m'$ and $\Phi_1$ are assumed as follows (Adachi and Oka, 1982):

$$\Phi_1(f_y) = \exp\{m' f_y\}$$

$$f_y = \eta^* = 0$$

Referring to Chaboche and Rousselier (1983), the nonlinear kinematical hardening parameter, $\chi_i^*$ is defined by the following evolutional equation:

$$d\chi_i^* = \dot{\gamma}^p (A^* d\dot{\gamma}^p - \chi_i^* d\gamma^p)$$

$$d\gamma^p = \sqrt{d\dot{\gamma}^p \cdot d\gamma^p}$$

where $A^*$ is related to the stress ratio at failure, namely $A^* = M^*$, and $B^*$ is related to the viscoplastic shear modulus $G^p$, namely $B^* = G^p / M^*$. $B^*$ varies with to the viscoplastic shear strain $\gamma^p$, from an initial value $B_0$ to the lower-limit value $B_s$ as follows:

$$B^* = B_s + (B_0 - B_s) \exp(-B_t \gamma^p)$$

where $B_t$ is the rate of decrease of $B^*$.

From Equations (3), (7) and (11), a general description of the cyclic VE-VP model for clay is given by

$$\dot{\varepsilon}_{ij}^{vp} = C_0 \Phi(f) \frac{\dot{\sigma}_i^p - \dot{\tau}_i^p}{\sigma_i^p} + C_2 \Phi(f) \frac{\dot{\tau}_i^p}{\tau_i^p}$$

$$\eta^* = \sqrt{\eta_0^*}$$

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$$\eta^* = \sqrt{\eta_0^*}$$

$$\eta^* = \sqrt{(\eta_0^* - \eta_{ijkl}^p)(\eta_0^* - \eta_{ijkl}^p)}$$

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$$B^* = B_s + (B_0 - B_s) \exp(-B_t \gamma^p)$$

Validation for clay using the proposed model

To evaluate the performance of the proposed VE-VP constitutive model, element simulations of triaxial tests were carried out. First, the viscoelastic behavior of clay under low levels of strain was confirmed by simulating cyclic loading triaxial tests for ideal clay. The cyclic undrained triaxial tests were then conducted to determine the cyclic strength and deformation characteristics of natural marine clay sampled at Tokushima.

Figure 1 shows cyclic stress-strain relationships for low levels of strain with various viscous coefficients from the E-VP and VE-VP models. Figure 1a was produced by the E-VP model, while Figure 1b was obtained using the VE-VP model. A clear difference at low levels of strain can be
seen in the stress-strain relationships obtained by these two models. In the case of the E-VP approach shown in Figure 1a, for example, elastic behavior predominates within 0.01% of the strain level, after which viscoplastic behavior emerges with increasing strain levels. Furthermore, hysteresis curves are flatter for low levels of strain, and the strain-level dependencies of the shear modulus within strain levels of 0.01% seem to be reduced, as shown in Figure 1a. On the other hand, for the VE-VP approach, the viscoelastic behaviors produce more circular hysteresis loops. Hence, the model explains damping due to apparent strain-level dependencies (Kim et al., 2000).

In order to determine the strength and deformation characteristics of natural clay under various cyclic loading conditions, two kinds of triaxial tests were carried out on clay specimens sampled at Komatsujima Port, Japan. One was a conventional undrained cyclic triaxial test. The other was a cyclic deformation test, which was conducted to determine the dynamic strain-level dependencies of deformation properties, that is, the so-called $G$-$\gamma$ and $h$-$\gamma$ relationship, where $G$, $\gamma$ and $h$ are the shear modulus, shear strain and hysteretic damping ratio, respectively. In the present study, the results of the cyclic deformation test are illustrated as both the equivalent Young’s modulus and the hysteretic damping ratio vs. axial strain relationship.

In a cyclic deformation test, a sinusoidal load of 0.05 Hz was used at each strain-level stage. Thirty stages of cyclic loading were performed, covering a strain range of less than 0.0005% to greater than 0.3%. A total of eleven cyclic loadings were applied in each stage by controlling axial load. The deformation properties, that is the equivalent Young’s modulus and the hysteretic damping ratio, were determined using the tenth hysteresis loop of the stress-strain relation because the hysteresis loop gradually settled during cyclic loading. Before the start of each stage, a drainage valve in the test apparatus was opened to dissipate excess pore-water pressure in the specimen.

Figure 2 shows the relationship between the single amplitude axial strain and the equivalent Young’s modulus, as well as the hysteretic damping ratio, as obtained by the cyclic deformation tests and their simulations by the two models. The relationships shown in Figure 2 are often used as input information for soil in earthquake response analyses. The gradual decrease of the equivalent Young’s modulus with increasing strain for low levels of strain is in accordance with the simulated results of the VE-VP model with proper viscoelastic parameters. The hysteretic damping ratio observed in the tests was also reproduced by the E-VP model and was much lower than that predicted by the VE-VP model for both small and large strains. The proposed model seems to be suitable for reproducing the non-linear deformation characteristics of clay over a wide range of strain. Usually a cyclic deformation test is carried out to investigate the $G$-$\gamma$ relationship of clay. However, the present simulations were conducted in the time domain by considering viscoelastic and viscoplastic effects. Thus, the simulation results in Figure 2 depend on the strain-rate and the results were obtained for a prescribed strain-rate in the tests. It can be assumed that the influence of strain-rate is small enough to evaluate the strain dependency of the deformation characteristics of clay.

In undrained cyclic triaxial tests, a symmetrical cyclic loading was applied to the isotropically consolidated clay specimen by a sinusoidal load with constant frequency of 0.1 Hz until the double-axial strain amplitude reached
Table 1. Physical properties of clay specimens.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>T-1</th>
<th>T-2</th>
<th>T-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td>32.0-32.8</td>
<td>35.0-35.8</td>
<td>26.0-26.8</td>
</tr>
<tr>
<td>Soil type</td>
<td>Clayey silt</td>
<td>Silty clay</td>
<td>Clayey silt</td>
</tr>
<tr>
<td>Specific gravity, G_s (g/cm³)</td>
<td>2.713</td>
<td>2.725</td>
<td>2.734</td>
</tr>
<tr>
<td>Water content, w (%)</td>
<td>46.9</td>
<td>40.2</td>
<td>45.6</td>
</tr>
<tr>
<td>Liquid limit, W_L (%)</td>
<td>48.5</td>
<td>39.4</td>
<td>45.1</td>
</tr>
<tr>
<td>Plastic index, I_p</td>
<td>22.0</td>
<td>16.7</td>
<td>18.9</td>
</tr>
<tr>
<td>Compression index, C_c</td>
<td>0.520</td>
<td>0.350</td>
<td>0.517</td>
</tr>
<tr>
<td>Consolidation yield stress, P_c (kPa)</td>
<td>314</td>
<td>392</td>
<td>402</td>
</tr>
<tr>
<td>Effective confining pressure, (kPa)</td>
<td>196</td>
<td>196</td>
<td>196</td>
</tr>
<tr>
<td>Single ampl. Cyclic deviator stress, σ_d</td>
<td>-</td>
<td>98.5</td>
<td>90.7</td>
</tr>
<tr>
<td>Applied shear stress ratio, σ_d/2σ_c'</td>
<td>-</td>
<td>0.251</td>
<td>0.234</td>
</tr>
<tr>
<td>Water content after shear, w(%)</td>
<td>46.76</td>
<td>34.7</td>
<td>43.1</td>
</tr>
</tbody>
</table>

Table 2. Parameters for the element simulations of cyclic stress-strain relationships.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>T-1</th>
<th>T-2</th>
<th>T-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus, E(MPa)</td>
<td>180</td>
<td>157</td>
<td>157</td>
</tr>
<tr>
<td>First shear elastic modulus, G_1(MPa)</td>
<td>60</td>
<td>54.1</td>
<td>54.1</td>
</tr>
<tr>
<td>Second shear elastic modulus, G_2(MPa)</td>
<td>20</td>
<td>10.8</td>
<td>10.8</td>
</tr>
<tr>
<td>Viscoelastic parameters, M^H (kPa·s)</td>
<td>2.9E+04</td>
<td>6.7E+04</td>
<td>6.7E+04</td>
</tr>
<tr>
<td>Stress ratio at failure state, M_f</td>
<td>1.3</td>
<td>1.4</td>
<td>1.4</td>
</tr>
<tr>
<td>Stress ratio at critical state, M_m</td>
<td>1.2</td>
<td>1.1</td>
<td>1.1</td>
</tr>
<tr>
<td>Viscoplastic parameters, C_1(1/s)</td>
<td>9.0E-08</td>
<td>9.0E-05</td>
<td>2.0E-05</td>
</tr>
<tr>
<td>Viscoplastic parameters, C_2(1/s)</td>
<td>1.0E-08</td>
<td>2.0E-05</td>
<td>7.0E-06</td>
</tr>
<tr>
<td>Viscoplastic parameters, m</td>
<td>15</td>
<td>12.5</td>
<td>12.5</td>
</tr>
<tr>
<td>Viscoplastic modulus parameters, B_0</td>
<td>70</td>
<td>40</td>
<td>22</td>
</tr>
<tr>
<td>Viscoplastic modulus parameters, B_1</td>
<td>69</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Viscoplastic modulus parameters, B_2</td>
<td>0.001</td>
<td>0.9</td>
<td>0.95</td>
</tr>
<tr>
<td>Compression index, λ</td>
<td>0.4</td>
<td>1.196</td>
<td>0.2223</td>
</tr>
<tr>
<td>Swelling index, κ</td>
<td>0.04</td>
<td>0.161</td>
<td>0.0376</td>
</tr>
</tbody>
</table>

10%. Table 1 shows the physical properties of the clay specimen used in the test. It should be noted that the clay specimens of T-1 through T-3 were sampled at different depths in different points at Komatsujima Port. Hence, different sets of parameters were used in the element simulations, and are listed in Table 2.

Figures 3 - 4 show the stress-strain relationships and the effective stress paths obtained by the undrained cyclic triaxial tests using samples T-2 and T-3. The results of element simulations using the proposed VE-VP model are also shown for comparison. Numerous cyclic undrained triaxial tests were conducted and most of results were similar to those of T-2 in Figure 3, which showed a remarkable increase in extensional axial strain during cyclic loading. Some cases reached failure state very quickly from a small number of cyclic loadings, such as T-3 in Figure 4.

From Figures 3 - 4, it can be seen that the stress-strain relationships during cyclic loading are well described by the proposed VE-VP model. In numerical simulations, the viscoplastic parameters \( B_0, B_1, \) and \( B_2 \) in Equation (16) dominate the results and control the plastic shear characteristics. In the case of T-2, the values of \( B_0, B_1, \) and \( B_2 \) were 40 and 0.9, while they were 22 and 0.95 for T-3, respectively. The value for T-3 indicates more viscoplastic behavior than T-2.

The above behavior observed in undrained cyclic triaxial tests is mainly related to the non-linearity of soil properties under high levels of strain, and it is clear that the cyclic VE-VP constitutive model gives a good
description of the non-linear characteristics of deformation for cohesive soil over a wide range of strain.

AMPLIFICATION CHARACTERISTICS AT ROKKO ISLAND

Ground condition

Rokko Island is an artificial island due south of the Kobe central business district in Japan. Figure 5 shows the layout of the array system of a borehole station as well as its velocity structure. The seismometers are located at a depth of ground level (GL.) 0.0, -35, -98, and -154.5 m at Rokko Island station. Reclamation work was finished in 1990, and in addition to a decomposed granite soil known as ‘Masado’, the sedimentary rock debris of the Kobe Group, which includes sandstone, mudstone and tuff, was used to create Rokko Island. As shown in Figure 5, the shear-wave velocity of the alluvial clay at Rokko Island is 115 m/sec.

Figure 6 indicates the soil profile and finite-element meshes of the ground model of Rokko Island used in the analysis. The total depth of the FEM mesh at Rokko
the bottom nodes of soils column were fixed in both horizontal and vertical directions; other nodes were fixed in the vertical direction but were free in the horizontal direction, and equi-horizontal displacement was assumed. Drainage was only allowed at the surface, and a plane-strain condition was assumed in the present study.

### Amplification ratio distribution

Table 3 shows a list of amplification ratios at Rokko Island. ‘Record data’ in the table denotes the year, month, day, hour and minute when the earthquake occurred. For example, the record data ‘46281309’ is an earthquake that occurred at 13:09 on June 28th, 1994. In the record data, ‘a’ and ‘b’ respectively denote ‘October’ and ‘November’. Group A includes records obtained before the 1995 Hyogoken Nanbu earthquake that occurred on January 17th, 1995.

Sugito et al. (2000) studied time-dependent characteristics at Port Island and Rokko Island using the same earthquake as the present study, although they used the record at GL.-155 m rather than at GL.-98 m for Rokko Island. They reported that the effects of soil liquefaction, or the reduction of soil rigidity on the amplification of peak ground motion, were predominant near the ground surface. Nine days or more after the 1995 Hyogoken Nanbu earthquake, the amplification ratios were nearly the same as they were before the earthquake both at Port Island and Rokko Island.

This study applied the VE-VP constitutive model for clay into a two-dimensional analysis code (plane-strain condition) known as LIQCA (Coupled Analysis of LIQuefaction), developed by Shibata et al. (1991) and Oka et al. (1994). It also attempted to conduct an amplification characteristics analysis of multi-layered ground as a newly expanded and established LIQCA-2D (VE-VP) code and determine the role of seismic waves in the behavior of clay.

The LIQCA-2D (VE-VP) code used in this study has constitutive equations of an E-VP model for sandy soil and of a VE-VP model for clayey soil, along with coupled soil-water dynamical governing equations based on Biot's (1962) two-phase mixture theory. The E-VP constitutive model for sandy soil expresses non-linear hysteresis by a non-linear kinematic hardening law and dilatancy at the time of dynamic load using an over-consolidated boundary surface. This model can express the behavior of the undrained shear strength of sandy soil under various initial stress conditions while anisotropic consolidations and initial shear stresses are applied. It applies a VE-VP constitutive model for clayey soil.

This study performed an amplification characteristics analysis focusing on the tremors following the Hyogoken Nanbu earthquake (magnitude of 7.2 in the seismic epicenter) of 1995. The acceleration measured during the Hyogoken Nanbu Earthquake amounted to 526.7 (gal) in

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**Figure 5.** Velocity structure and layout of seismometers.

**Figure 6.** Soil profile and finite-element meshes of the ground model.
Table 3. Amplification ratios at Rokko Island.

<table>
<thead>
<tr>
<th>Group</th>
<th>Record data</th>
<th>Amplification ratio (0/98 m)</th>
<th>Amplification ratio (35/98 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0.957</td>
<td>0.757</td>
</tr>
<tr>
<td>A</td>
<td>46281309</td>
<td>1.123</td>
<td>0.973</td>
</tr>
<tr>
<td></td>
<td>47281002</td>
<td>0.883</td>
<td>1.108</td>
</tr>
<tr>
<td></td>
<td>4a241151</td>
<td>1.095</td>
<td>1.068</td>
</tr>
<tr>
<td></td>
<td>4b092027</td>
<td>1.243</td>
<td>1.107</td>
</tr>
<tr>
<td></td>
<td>4b100038</td>
<td>1.519</td>
<td>0.926</td>
</tr>
<tr>
<td>B</td>
<td>51201538</td>
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<td>1.163</td>
</tr>
<tr>
<td></td>
<td>51252316</td>
<td>0.915</td>
<td>1.047</td>
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<tr>
<td></td>
<td>51260101</td>
<td>1.364</td>
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<td>51262308</td>
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<td>51291603</td>
<td>1.058</td>
<td>1.064</td>
</tr>
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<td>C</td>
<td>52030436</td>
<td>0.767</td>
<td>0.878</td>
</tr>
<tr>
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<td>52032037</td>
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<td>0.934</td>
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<td>1.466</td>
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<td>52240804</td>
<td>1.227</td>
<td>1.000</td>
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</tbody>
</table>

Table 4. Summary of aftershocks.

<table>
<thead>
<tr>
<th>Case</th>
<th>A</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Occ. time</td>
<td>95.01.23 21:44</td>
<td>95.02.02 16:19</td>
</tr>
<tr>
<td>Magnitude</td>
<td>4.3</td>
<td>4.2</td>
</tr>
<tr>
<td>Depth(km)</td>
<td>16</td>
<td>18</td>
</tr>
<tr>
<td>Epicenter</td>
<td>Hyogo, SE</td>
<td>Hyogo, E</td>
</tr>
<tr>
<td>Latitude(°)</td>
<td>34.37.6</td>
<td>34.41.4</td>
</tr>
<tr>
<td>Longitude(°)</td>
<td>135.19.1</td>
<td>135.09.0</td>
</tr>
<tr>
<td>Peak acceleration (cm/s²)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NS</td>
<td>23.06</td>
<td>21.741</td>
</tr>
<tr>
<td>EW</td>
<td>25.893</td>
<td>20.692</td>
</tr>
<tr>
<td>UD</td>
<td>63.029</td>
<td>46.29</td>
</tr>
</tbody>
</table>

Port Island GL.-83 m; it was considered a large-scale earthquake and was accompanied by a liquefaction phenomenon that caused countless losses to life and property.

Meanwhile, for small- and middle-scale earthquakes, the application of a constitutive model sensitive to the damping characteristics of clayey soil is an important element to be considered in cases of multiple soil layers. Accordingly, this study attempted to suggest an appropriate constitutive model for small- and middle-scale earthquake response analysis and to perform amplification characteristics for the site where tremors followed the earthquake with seismic magnitude of approximately 4. It then attempted to review the feasibility of the suggested model through an analysis of the results. Table 4 is a list of the aftershocks used in this study.

This study analyzed and compared recorded seismic waves to the results of amplification characteristics analysis of two models: an E-VP model and a VE-VP model that divided the earthquake response behavior in multiple soil layers, including clayey soil layers.

Figures 7 to 8 shows the relationships between amplification ratio and depth in Group B and C. The damping characteristics in the clay layer (GL.-44 to -34 m) were captured by the E-VP model and the VE-VP constitutive model. The damping in clay and amplification in reclaimed ground predicted by the E-VP model were larger than those calculated by the VE-VP model. The E-VP model gave wide variations in the amplification ratio,
while the VE-VP model predicted almost no amplification in the sand layer below GL.-35 m. The VE-VP model more closely matched the obtained records than the E-VP model.

From the examination of amplification characteristics analysis for the aftershock from the E-VP model and the VE-VP model, this study demonstrated that it is possible to reproduce the behaviors of clayey soil over a wide strain range, including small strain levels, using the VE-VP model; however, it is difficult to explain the damping characteristics at small strain using the E-VP model. The reason for this difference was that the VE-VP model contained a Voigt element; thus, the VE-VP model could more accurately explain the damping characteristics of
clayey soil. Therefore, it is appropriate to apply a cyclic VE-VP model that can analyze the damping characteristics of clayey soil during small- and middle-scale earthquakes in multi-layered ground.

**CONCLUSIONS**

This study performed an amplification characteristics analysis on multiple soil layers using a two-dimensional analysis code for plane-strain conditions, LIQCA-2D(VE-VP). The present study also applied a cyclic VE-VP constitutive model of clayey soil and investigated the role of seismic waves in the behavior of clayey soil. The main results of this study are as follows.

1. The results of element simulation indicated that the proposed model can reproduce the viscoelastic behaviors of clay for low levels of strain as well as the viscoplastic behaviors of clay at higher levels of strain.
2. After analyzing the tremor following the earthquake in the artificial reclaimed land of Rokko Island in Kobe, Japan, using the VE-VP model and the E-VP model, the VE-VP model better reproduced the seismic record.
3. It is more appropriate to apply a cyclic VE-VP model that can analyze the damping characteristics of clayey soil during earthquake for multiple soil layers.

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