EFFECTIVE STRESS ANALYSES OF PORT STRUCTURES

SUSUMU IAI, KOJI ICHI, HANLONG LIU, and TOSHIKAZU MORITA

ABSTRACT

Effective stress analyses are performed on the seismic response of port structures during the 1995 Hyogoken-Nambu earthquake. The structures analyzed include: caisson type quay walls which moved up to 5 m maximum toward the sea; pneumatic caisson type bridge foundations which moved toward the sea, reducing the distance between the bridge foundations across a sea channel by a total of 0.8 m; and composite breakwaters which settled 2 m. Although these structures were all a rigid concrete type and shaken by the same earthquake, the mode and extent of deformation/failure were different depending on the backfill and embedment conditions of the structures.

The constitutive model of soil used for the seismic analyses is a multiple mechanism model defined in strain space. The model can take the effect of principal stress axis rotation into account, which is known to play an important role in the cyclic behavior of sand under anisotropic stress conditions. The model parameters are calibrated based on in-situ and laboratory investigations, including in-situ freeze sampling. The results of the effective stress analyses are consistent with those measured, including the mode and extent of deformation/failure, suggesting that the cyclic behavior of soil, idealized through the effective stress model, can explain the variety of seismic response of the port structures in a unified manner.

Key words: constitutive equation of soil, deformation, dynamic, earthquake, effective stress, foundation, harbor, liquefaction, sandy soil, stress path (IGC: E8/E13/E12)

INTRODUCTION

Port structures include such a variety of structures as quay walls, breakwaters, and bridge foundations at the waterfront. These structures may be classified, from a geotechnical perspective, into retaining walls, and deep and shallow foundations. Depending on the type of soil-structure interaction, the action of the soil on a structure component can be different. In a simplified analysis, the action of soil is idealized through earth pressures, subgrade reaction force, and bearing capacity.

Actual seismic response of these structures is a result of dynamic soil-structure interaction, being much more complex than envisioned in simplified analysis. Even before an earthquake, gravity usually puts the soil into a stress state close to the shear failure condition. The active failure state of soils at retaining structures is an example. Earthquake shaking will then affect the soil behavior through the dynamic soil-structure interaction process. The soil will accordingly deform under the action of both the static load due to gravity and the cyclic load due to earthquake shaking. The variety of effects of soils mentioned earlier are determined as a result of this soil deformation. Increasing attention, thus, has been directed toward developing a reasonable constitutive model of soils to capture the essential features of the soil behavior under both static and cyclic loads. In particular, a model based on a multiple shear mechanism defined in strain space has been shown to be promising (Iai et al., 1992a).

During the Hyogoken-Nambu earthquake of 1995, Kobe Port was shaken with a strong motion having peak ground accelerations of 0.54 g and 0.45 g in the horizontal and vertical directions (Inagaki et al., 1996). Many port structures were damaged. Most of the port structures in Kobe Port were of a rigid block type made of concrete caissons. Caisson type quay walls moved up to 5 m, 3 m on average, toward the sea. Pneumatic caissons used for a bridge foundation moved toward the sea, reducing the distance between the bridge foundations across the sea channel by a total of 0.6 to 0.8 m. Composite breakwaters settled about 2 m with small horizontal movement. The different responses of these port structures are presumed to have been determined by the geotechnical conditions such as backfill and embedment conditions.

The objective of this study is to discuss how well the cyclic behavior of soil, idealized through the effective
stress model mentioned earlier, can explain the variety of seismic responses of the port structures in Kobe Port in a unified manner. The structures analyzed are the caisson walls, the pneumatic caisson foundations, and the breakwaters mentioned above.

**CONSTITUTIVE MODEL**

The constitutive model used consists of a multiple shear mechanism defined in the plane strain condition (Iai et al., 1992a). This model can simulate the behavior of sand subjected to the rotation of principal stress axes, which plays an important role in the behavior of initially anisotropically consolidated sand under cyclic simple shear (Iai et al., 1992b). With the effective stress and strain vectors written as:

\[
\{\sigma\}' = \{\sigma'_x, \sigma'_y, \tau_{xy}\} \\
\{\varepsilon\}' = \{\varepsilon_x, \varepsilon_y, \gamma_{xy}\}
\]

(1)

(2)

the basic form of the constitutive relation is given by

\[
\{d\sigma\}' = [D]((de) - \{d\varepsilon\})
\]

(3)

where

\[
[D] = K \{n^{(0)}\}' \{n^{(0)}\}' + \sum_{i=1}^{I} R^{(i)}_{L,U} \{n^{(i)}\}' \{n^{(i)}\}'
\]

(4)

In this relation, the term \(\{d\varepsilon\}\) in Eq. (3) represents the effect of dilatancy and is given from the volumetric strain increment due to the dilatancy as

\[
\{d\varepsilon\}_v = \begin{pmatrix} d\varepsilon_x \\ d\varepsilon_y \\ 0 \end{pmatrix}
\]

(5)

The first term in Eq. (4) represents the volumetric mechanism specified with a rebound modulus \(K\). The direction vector is given by

\[
\{n^{(0)}\}' = \begin{pmatrix} 1 \\ 1 \\ 0 \end{pmatrix}
\]

(6)

The second term in Eq. (4) represents the multiple shear mechanism. Each mechanism \(i = 1, \cdots, I\) represents a virtual simple shear mechanism, with each simple shear plane oriented at an angle \(\theta_i/2 + \pi/4\) relative to the x axis. The tangential shear modulus \(R^{(i)}_{L,U}\) represents the hyperbolic stress strain relationship under the drained condition. The hysteresis characteristics are specified consistent with the laboratory results. The direction vectors for the shear mechanism in Eq. (4) are given by

\[
\{n^{(i)}\}' = \{\cos \theta_i, -\cos \theta_i, \sin \theta_i\} \quad \text{for } i = 1, \cdots, I
\]

(7)

where

\[
\theta_i = (i - 1)\Delta \theta \\
\Delta \theta = \pi/I
\]

(8)

(9)

The loading and unloading for the shear mechanism are separately defined for each mechanism by the sign of \(\{n^{(i)}\}'(de)\). The sign convention for the stress and strain throughout this paper is defined as extension positive. The model has ten parameters; two specify the elastic properties of soil, two others specify plastic shear behavior, and the rest control dilatancy as shown in Table 1.

As mentioned earlier, the soil in the vicinity of a structure undergoes both static and cyclic loads due to gravity and earthquake shaking. The soil below a caisson type quay wall, for example, may be put into the stress state of a compression shear due to the dead weight from the caisson before an earthquake. Horizontal earthquake shaking will then cause cyclic simple shear in this soil element. The effective stress analyses in this study have to be able to capture the essential features of soil behavior under these stress conditions. In order to review the capability of the effective stress model used in this study, the undrained cyclic simple shearing of initially anisotropically consolidated sand was analyzed as a boundary value problem.

In this analysis, the soil element was, first, consolidated with \(K_0 = 0.5\) condition. This is to simulate the effect of gravity before the earthquake. By imposing the undrained condition, the cyclic simple shear was, then, applied to the soil element to simulate the effect of earthquake shaking. The initial axial stress difference was kept unchanged throughout the cyclic shearing to simulate the

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Type of mechanism</th>
<th>Parameter designation</th>
</tr>
</thead>
<tbody>
<tr>
<td>(K_a)</td>
<td>elastic volumetric</td>
<td>rebound modulus</td>
</tr>
<tr>
<td>(G_{ms})</td>
<td>elastic shear</td>
<td>shear modulus</td>
</tr>
<tr>
<td>(\phi_r)</td>
<td>plastic shear</td>
<td>shear resistance angle</td>
</tr>
<tr>
<td>(\phi_p)</td>
<td>plastic dilatancy</td>
<td>phase transformation angle</td>
</tr>
<tr>
<td>(H_m)</td>
<td>plastic shear</td>
<td>upper bound for hysteretic damping factor</td>
</tr>
<tr>
<td>(r_1)</td>
<td>plastic dilatancy</td>
<td>initial phase of cumulative dilatancy</td>
</tr>
<tr>
<td>(r_2)</td>
<td>plastic dilatancy</td>
<td>final phase of cumulative dilatancy</td>
</tr>
<tr>
<td>(w_1)</td>
<td>plastic dilatancy</td>
<td>overall cumulative dilatancy</td>
</tr>
<tr>
<td>(S_1)</td>
<td>plastic dilatancy</td>
<td>ultimate limit of dilatancy</td>
</tr>
<tr>
<td>(c_i)</td>
<td>plastic dilatancy</td>
<td>threshold limit for dilatancy</td>
</tr>
</tbody>
</table>

Fig. 1. Stress conditions for cyclic simple shearing of anisotropically consolidated soil
The results of the model performance obtained by the analysis are as follows. First, the computed stress path is shown in Fig. 2(a). In this figure, the stress path is plotted in a plane defined by the mean effective stress \( \left(-\sigma'_m\right) = -\left(\sigma'_x + \sigma'_y\right)/2 \) and the deviator stress defined by

\[
\tau = \sqrt{\left(\frac{\sigma'_x - \sigma'_y}{2}\right)^2 + \tau'_{xy}^2} \quad (10)
\]

As shown in Fig. 2(a), the effective stress path gradually approaches the failure line as the cyclic loading continues. In accordance with this, the stress path in \( \tau_{xy} - \left(-\sigma'_m\right) \) plane, shown in Fig. 2(b), exhibits a lower bound for \( \left(-\sigma'_m\right) \). This is known as the effect of initial shear. The stress and strain curve for \( \tau_{xy} \) and \( \gamma_{xy} \), shown in Fig. 2(c), indicates gradual growth in the amplitude of \( \gamma_{xy} \), whereas the axial strain difference \( (\varepsilon_x - \varepsilon_y) \), shown in Fig. 2(d), exhibits a cumulative increase. In the constitutive model used in this study, the negative dilatancy \( \left(\varepsilon'_i\right) \) in Eq. (5) is generated by the cyclic simple shear, resulting in a decrease in the mean effective confining pressure as shown in Fig. 2(a). This results in a higher deviator stress ratio (i.e. a higher ratio of the axial stress difference over the current confining pressure), thus generating an axial strain difference. The axial strain difference accumulates as the stress path gradually approaches the shear failure line.

A conceptual image of the deformation of a soil element undergoing the stress and strain conditions discussed here is illustrated in Fig. 3. As shown in this figure, the soil will gradually deform along the directions of the initial principal stresses. This cumulative increase of axial strain difference is presumed to be an important mechanism for governing the movement and settlement of the structures. To evaluate and confirm this notion is the focus of the rest of this paper. The sand behavior under anisotropic stress conditions discussed here have long since been studied and confirmed by laboratory testing (Ishihara and Li, 1972).

### Table 2. Model parameters for analysis of single soil element

<table>
<thead>
<tr>
<th>Soil type</th>
<th>( \rho ; t/m^3 )</th>
<th>( G_{max} ; kPa )</th>
<th>( -\sigma'_m ; kPa )</th>
<th>( \phi'_l ; \text{deg} )</th>
<th>( \phi'_s ; \text{deg} )</th>
<th>Parameters for dilatancy</th>
</tr>
</thead>
<tbody>
<tr>
<td>medium dense sand</td>
<td>1.8</td>
<td>58320</td>
<td>106</td>
<td>37</td>
<td>28</td>
<td>( w_1=5.5, p_1=0.6, p_2=0.6, c_1=2.3, S_1=0.005 )</td>
</tr>
</tbody>
</table>

Fig. 2. Stress and strain relationships of anisotopically consolidated soil during cyclic simple shearing: (a) stress path in \( \tau - \left(-\sigma'_m\right) \) plane, (b) stress path in \( \tau_{xy} - \left(-\sigma'_m\right) \) plane, (c) stress and strain curve in \( \tau_{xy} - \gamma_{xy} \) plane, and (d) stress and strain curve in \( \tau_{xy} - (\varepsilon_x - \varepsilon_y) \) plane.
EFFECTIVE STRESS ANALYSES

For the effective stress analyses, the model parameters, a total of ten, were determined from the in-situ velocity logging, SPT N-values and the results of the cyclic triaxial tests. In particular, in-situ soil freeze sampling was performed at two sites at Kobe Port. Undisturbed samples 60 cm long with a diameter of 30 cm were used for the cyclic triaxial tests (Inagaki et al., 1996). The specific values of the parameters used for the analyses are shown for each structure analyzed.

The constitutive model was incorporated into a finite element computer code and used for the numerical analysis. All the analyses were performed for the plane strain condition. Before the earthquake response analysis, a static analysis was performed with gravity under drained conditions to simulate the stress conditions before the earthquake. The results of the static analyses were used for the initial conditions for the earthquake response analyses. The seismic analyses were performed under undrained conditions (Zienkiewicz and Bettess, 1982) to approximate the behavior of saturated soils under transient and cyclic loads and/or earthquakes. The input earthquake motions were those recorded at the Port Island site successfully operated by the Kobe City Government (Iwasaki and Tai, 1996). The direction component normal to the face line of structures was computed for each structure analyzed by a vector projection of the horizontal earthquake motion. Both the horizontal and vertical components were used as the input excitations for the seismic response analysis. In order to simulate the incoming and outgoing waves through the boundaries of the analysis domain, equivalent viscous dampers were used at the boundaries. The effect of the free field motions was also taken into account by performing one dimensional response analysis at the outside fields and assigning the free field motion through the viscous dampers.

The structures made of concrete caissons were modeled using linear elastic solid elements. The soil-structure interfaces were modeled by the joint elements. The sea water was modeled as incompressible fluid and was formulated as an added mass matrix based on the equilibrium and continuity of the fluid at the solid-fluid interface (Zienkiewicz, 1977). The largest number of nodes and elements used for the analyses in this study were 2660 nodes and 4654 elements. More specific considerations given to the analyses will be explained later for each structure analyzed.

QUAY WALLS ON LIQUEFIABLE SOIL

Most of the quay walls at Kobe Port were caisson type, constructed on loose saturated decomposed granite which was used for backfilling the excavated portion of the alluvial clayey deposit in order to attain the required bearing capacity of the foundation. As mentioned earlier, these walls moved about 5 m maximum, about 3 m on average, toward the sea during the earthquake. The walls also settled about 1 to 2 m and tilted about 4 degrees toward the sea. Figure 4 shows a typical cross section and deformation of the caisson walls. There was little evidence of liquefaction of the backfill in the vicinity of the caisson walls whereas extensive evidence of liquefaction of landfill soil was observed inland about 30 m or further from the walls (Inagaki et al., 1996).

Although the sliding mechanism could explain the large horizontal displacement of the caisson walls, this mechanism does not explain the large settlement and tilting of the caissons. Reduction in the bearing capacity of foundation soils due to excess pore water pressure increase, then, was speculated as a main cause of the damage to the caisson walls at Kobe Port. In order to confirm this speculation, effective stress analysis was performed on the seismic response of the caisson walls. The analysis domain used for the finite element analysis covered a cross sectional area of about 220 m by 40 m in the horizontal and vertical directions, respectively. The model parameters used are shown in Table 3. In order to simplify the analysis, the portion of the subsoil made of the sand drain, shown in Fig. 4, was assumed to have the same parameters as that of the alluvial clay.

Effective stress analysis resulted in the residual deformation shown in Fig. 5 (Iai et al., 1997). As shown in this figure, the computed residual displacements at the top of the caisson wall were 3.5 m and 1.5 m in the horizontal and vertical directions, respectively. The tilting of the caisson wall was 4 degrees toward the sea. The computed deformation was, thus, basically consistent with that observed. The computed mode of deformation of the caisson wall was to tilt into and push out the foundation soil beneath the caisson. This was also consistent with the observed deformation mode of the rubble foundation shown in Fig. 6, which was investigated by divers.

The computed displacements were gradually induced as shown in Fig. 7. The sign convention for displacements and accelerations is defined positive for the right (when looking at the analysis domain) and upward direc-
tions. The computed excess pore water pressures also gradually increased in the foundation soil as shown for Element A in Fig. 8, but never achieved the 100% level, in which the excess pore water pressure ratios were defined by $(1 - \sigma''/\sigma''_0)$, with $\sigma''$ and $\sigma''_0$ denoting the current and initial mean effective stresses. The computed excess pore water pressures just behind the caisson wall showed a rapid increase but then a decrease after about 8 seconds, as shown for Element B in Fig. 8, and never achieved the 100% level, whereas those far inland increased rapidly and achieved a level close to 100%. These results are consistent with the observed phenomena regarding liquefaction.

In order to discuss the details of the mechanism of deformation and the excess pore water pressure increase, the stress and strain relationships of soils below the caisson wall (Element A in Fig. 8) are plotted in Fig. 9. As shown in Fig. 9(a), the computed stress path gradually approaches the failure line with fluctuation around the initial deviator stress. Since the initial deviator stress was maintained close to the original level, further reduction in the mean effective stress (i.e., excess pore water pressure increase) was prevented by the shear failure condition. In accordance with this stress path, the shear strain $\gamma_{xy}$ is gradually induced in the negative (i.e., seaward) direction as shown in Fig. 9(b). This gradual generation of shear strain is presumed to be induced by the earth pressures from the backfill soil. The axial strain difference is also gradually induced as shown in Figs. 9(c) and (d). This soil behavior causes settlements due to the compressive vertical normal strain. The seaward movement can also be induced in accordance with the tensile horizontal normal strain, provided the movement of the soil element toward the land side is restricted. The cumulative increase of axial strain difference in accordance with the initial deviator stress shown here is similar to that of the cyclic behavior of sand under anisotropic stress conditions discussed earlier.

Behind the caisson wall, tensile strain in the horizontal direction was increased in the backfill sand due to the seaward movement of the caisson wall, resulting in dilation of the sand (Iai and Ichii, 1997). Thus, when the wall is

<table>
<thead>
<tr>
<th>Soil type</th>
<th>$\rho$ t/m$^3$</th>
<th>$G_{ns}$ kPa</th>
<th>$-\sigma''$ kPa</th>
<th>$\phi'_n$ deg</th>
<th>$\phi_p$ deg</th>
<th>Parameters for dilatancy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation soil</td>
<td>1.8</td>
<td>58320</td>
<td>106</td>
<td>37</td>
<td>28</td>
<td>$w_i=5.5$, $p_i=0.6$, $p_1=0.6$, $c_i=2.3$, $S_i=0.005$</td>
</tr>
<tr>
<td>Backfill soil</td>
<td>1.8</td>
<td>79380</td>
<td>63</td>
<td>36</td>
<td>28</td>
<td>$w_i=6.0$, $p_i=0.5$, $p_1=0.8$, $c_i=2.43$, $S_i=0.005$</td>
</tr>
<tr>
<td>Clay</td>
<td>1.7</td>
<td>74970</td>
<td>143</td>
<td>30</td>
<td></td>
<td>(assuming $d_{hs}=0$ in Eq. (5))</td>
</tr>
<tr>
<td>Foundation rubble and rubble backfill</td>
<td>2.0</td>
<td>180000</td>
<td>98</td>
<td>40</td>
<td></td>
<td>(assuming $d_{hs}=0$ in Eq. (5))</td>
</tr>
</tbody>
</table>

Friction angle at bottom of concrete caisson $\delta=31$ deg
Friction angle at back of concrete caisson $\delta=15$ deg

Fig. 4. Cross section and deformation of a quay wall at Kobe Port (RC-5, Rokko Island)
Fig. 5. Computed deformation of a quay wall at the end of shaking: (a) Mesh deformation, (b) Displacement vectors (after Iai et al., 1997)

Fig. 6. Deformation of rubble foundation of a quay wall investigated by divers (after Inagaki et al., 1996)
Fig. 7. Computed accelerations and displacements at the upper seaward corner of the caisson for a quay wall (after Iai et al., 1997).

Fig. 8. Computed time histories and distribution of excess pore water pressure ratios for a quay wall.

Fig. 9. Computed stress and strain relationships of soil below the caisson wall: (a) stress path in $\tau - (-\sigma_m)$ plane, (b) stress and strain curve in $\tau_{xy} - \gamma_{xy}$ plane, (c) stress and strain curve in $(\sigma_x - \sigma_y) - (\varepsilon_x - \varepsilon_y)$ plane, and (d) stress and strain curve in $\tau_{xy} - (\varepsilon_x - \varepsilon_y)$ plane.
easier to move, the excess pore water pressures in the backfill soil behind the wall can be kept to a low level, never to achieve complete liquefaction.

In order to evaluate the effects of the excess pore water pressure increase on the displacements of a wall, the following three analyses were performed as a parameter study. The parameter study included a virtual soil model, to be called non-liquefiable soil, which has the same properties as those used in the aforementioned analysis but without the effect of dilatancy.

To distinguish the cases in the parameter study, the case which dealt with the actual quay wall during the earthquake described earlier is designated as Case-1. Case-2, as illustrated in Fig. 10, idealizes the backfill soils both below and behind the caisson wall as non-liquefiable soils. Case-3 idealizes only the soil below the wall as non-liquefiable, while Case-4 idealizes only the backfill soil behind the wall as non-liquefiable.

In order to evaluate the individual effect of the pore water pressure increase below and behind the wall, the major results of the analysis are summarized in Table 4 including those of Case-1. The results of Case-2 show that the caisson wall will move about 1 to 2 m in the horizontal direction and incline about 2 to 3 degrees purely due to the inertia force of the earthquake. The results of

Table 4. Major results of parameter study for quay wall

<table>
<thead>
<tr>
<th>Case</th>
<th>$d_x$ (m)</th>
<th>$d_y$ (m)</th>
<th>$\theta$ (deg)</th>
<th>$a_{\text{max}}$ (m/s²)</th>
<th>$a_{\text{max}}$ (m/s²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case-1</td>
<td>3.5</td>
<td>1.5</td>
<td>4.1</td>
<td>3.3</td>
<td>3.8</td>
</tr>
<tr>
<td>Case-2</td>
<td>1.6</td>
<td>0.6</td>
<td>2.4</td>
<td>4.2</td>
<td>3.7</td>
</tr>
<tr>
<td>Case-3</td>
<td>2.1</td>
<td>0.7</td>
<td>3.1</td>
<td>4.2</td>
<td>3.6</td>
</tr>
<tr>
<td>Case-4</td>
<td>2.5</td>
<td>1.1</td>
<td>2.2</td>
<td>3.3</td>
<td>3.6</td>
</tr>
</tbody>
</table>

$d_x$: horizontal residual displacement at the top of caisson
$d_y$: vertical residual displacement at the top of caisson
$\theta$: inclination of caisson
$a_{\text{max}}$: horizontal maximum acceleration at the top of caisson
$a_{\text{max}}$: vertical maximum acceleration at the top of caisson

Fig. 10. Schematic figure for the parameter study: conditions for Cases-2 through 4

Fig. 11. Cross section of a quay wall founded on loose deposit at Kobe Port (PC-13, Port Island) (after Inagaki et al., 1996)
Case-3 indicate that the excess pore water pressure increase in the backfill soil will increase the deformation of the caisson wall about 30% both in the horizontal displacement and the inclination. The results of Case-4 indicate that the excess pore water pressure increase in the soil below the wall will increase the horizontal displacement about 50% but will reduce the inclination by 10%. Finally, the results of Case-1 indicate that the excess pore water pressure increase both below and behind the wall will increase the horizontal displacement and the inclination by a factor of about two.

The results of Cases-1 and 3 from this parameter study are compared with the performance of the quay walls at Port Island (phase II). As shown in Figs. 11 and 12, one (PC-13) was constructed on a loose deposited foundation, while others (PC-14 and -15) were constructed on a foundation improved by the sand compaction pile (SCP) method. Although these quay walls were constructed...
The displacements of quay walls PC-14 and -15 constructed on SCP foundations were about 2.5 m and 0.3 m in horizontal and vertical directions. The displacements of quay wall PC-13 on a loose foundation soil were about 3.5 m and 1.5 m in the horizontal and vertical directions, respectively. The displacements of PC-13 on a loose foundation soil are 1.0 m larger than those of PC-14 and -15, both in horizontal and vertical directions, suggesting the effects of the excess pore water pressure increase in the foundation soil. The analysis conditions of Case-3, having non-liquefiable foundation, approximate the performance of quay walls PC-14 and 15. The analysis conditions of Case-1, having loose deposited foundation, approximate the performance of quay wall PC-13. The results for the parameter study of Cases-1 and 3 discussed above can thus basically explain the difference between the quay walls constructed on SCP and loose deposited foundations.

PNEUMATIC CAISSON FOUNDATIONS

In contrast to the caisson walls, the pneumatic caisson foundations supporting the bridges in the Kobe Port area were deeply embedded in a firm foundation layer of Pleistocene origin. The Kobe Ohashi Bridge, 319 m total length, with a central span 217 m long, lies at a site between the Fourth Pier and the Port Island in Kobe Port. A cross section of the bridge and its foundations is shown in Fig. 14. As shown in this figure, the pneumatic caissons were embedded to a depth of 33 m, having a 15 m by 30 m horizontal section with the longer face directed toward the sea. The bridge was supported by a fixed shoe at P3 caisson on the Port Island side and movable shoes at the rest, P1, P2 and P4. The pneumatic caissons were used not only to support the bridge but also to serve as a portion of the retaining walls. Soils behind these caissons were a loose backfill of decomposed granite. The excavated areas around the pneumatic caissons were backfilled with rubble stones and loose decomposed granite as shown in Fig. 14. Below the excavated level of K.P. -18 to -23 m lies an alluvial sandy layer having SPT N-values of 20 to 30 to a level of K.P. -26 m, underlain by alternating layers of clay and sandy gravel of Pleistocene origin having SPT N-values ranging from 20 to 50 to a level of K.P. -75 m (Okashita et al., 1996).

During the Hyogoken-Nambu earthquake, the pneumatic caissons moved toward the sea, decreasing the distance between the bridge foundations P2 and P3 by a total of 0.6 to 0.8 m. The pneumatic caissons at P2 and P3 also tilted 0.36 and 0.93 degrees, respectively. Sand boils were seen in the area around P3 and P4. Despite the deep embedment, the pneumatic caissons did move. This

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Fig. 14. Cross section of the Kobe Ohashi Bridge, including the pneumatic caisson foundations

Table 5. Model parameters for pneumatic caisson analysis

<table>
<thead>
<tr>
<th>Soil type</th>
<th>$\rho$ t/m³</th>
<th>$G_{so}$ kPa</th>
<th>$\sigma'_{so}$ kPa</th>
<th>$\phi_f$ deg</th>
<th>$\phi_p$ deg</th>
<th>Parameters for dilatancy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Backfill soil</td>
<td>1.8</td>
<td>79380</td>
<td>62</td>
<td>36</td>
<td>28</td>
<td>$w_f = 10.5, \ p_f = 0.5, \ p_s = 0.6, \ c_1 = 3.4, \ S_i = 0.005$</td>
</tr>
<tr>
<td>Backfill soil</td>
<td>1.8</td>
<td>72520</td>
<td>104</td>
<td>36</td>
<td>28</td>
<td>$w_f = 5.5, \ p_f = 0.6, \ p_s = 0.8, \ c_1 = 2.3, \ S_i = 0.005$</td>
</tr>
<tr>
<td>below rubble backfill</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Alluvial sand</td>
<td>1.96</td>
<td>59780</td>
<td>144</td>
<td>36</td>
<td>28</td>
<td>$w_f = 16.0, \ p_f = 0.5, \ p_s = 0.8, \ c_1 = 2.5, \ S_i = 0.005$</td>
</tr>
<tr>
<td>Pleistocene sand²</td>
<td>1.89</td>
<td>166600</td>
<td>245</td>
<td>40</td>
<td>28</td>
<td>$w_f = 14.5, \ p_f = 0.5, \ p_s = 0.6, \ c_1 = 25.3, \ S_i = 0.005$</td>
</tr>
<tr>
<td>Pleistocene sand²</td>
<td>1.87</td>
<td>205800</td>
<td>245</td>
<td>40</td>
<td>28</td>
<td>$w_f = 16.5, \ p_f = 0.5, \ p_s = 0.8, \ c_1 = 3.43, \ S_i = 0.005$ (assuming $\Delta w_p = 0$ in Eq. (5))</td>
</tr>
<tr>
<td>Rubble backfill</td>
<td>1.92</td>
<td>176400</td>
<td>218</td>
<td>40</td>
<td>—</td>
<td>(assuming $\Delta w_p = 0$ in Eq. (5))</td>
</tr>
<tr>
<td>Alluvial clay</td>
<td>1.62</td>
<td>117600</td>
<td>356</td>
<td>30</td>
<td>—</td>
<td></td>
</tr>
</tbody>
</table>

¹) P2 side
²) P3 side
case history offered an opportunity to test the capability of effective stress analyses to evaluate the effect of embedment for rigid foundations.

The analysis domain used for the finite element analysis covered a cross sectional area of about 450 m by 120 m, including the bridge. The parameters used for the analysis are shown in Table 5. As mentioned earlier, the analysis was performed in two dimensions. Since the actual configuration of the pneumatic caissons is three dimensional, a certain error may arise from the two dimensional analysis. This issue has yet to be thoroughly studied by performing three dimensional analysis. A series of centrifuge model tests on soil-pile and soil-wall systems, however, indicates that the seismic response of the soil-wall system may be a good approximation of the soil-pile system (Funahara et al., 1996; Cubrinovski et al., 1996), suggesting that the two dimensional analysis performed in this study will be a good approximation of the three dimensional analysis.

The results of the effective stress analysis are shown in Figs. 15 through 18 (Liu et al., 1997). Figure 15 shows the deformation modes of the bridge and foundation systems. Deformation toward the sea is shown in this figure, resulting in 0.46 m at P2 and 0.48 m at P3, with a total of 0.94 m toward the sea. This is consistent with post earthquake measurements.

Shown in Fig. 16 are the computed horizontal displacements and accelerations at the top of caissons P2 and P3. The sign convention, as mentioned earlier, is defined positive for the right when facing the cross section of the bridge. A gradual increase in the displacements toward the sea, similar to those of the caisson walls discussed earlier, is noted but the dynamic portion of displacement relative to the residual displacement is much larger than that for the caisson walls. A distinctive difference in the seismic response is noted here between the embedded caisson foundations and the caisson walls.

The excess pore water pressures behind the pneumatic caissons increased up to 100%, as shown in Fig. 17, which shows the pore pressure distribution at the end of the shaking. In contrast to the backfill behind the caisson walls, the soils behind the pneumatic caissons did not freely undergo tensile horizontal normal strain, resulting in liquefaction. This is confirmed by the stress and strain of the backfill soil (at a depth of 5 m and 20 m behind P2) shown in Fig. 18. As shown in this figure, the initial deviator stress rapidly decreases and the stress path is directed toward the origin of the stress plane, resulting in a complete loss of confining pressure. In this context, the behavior of the soil behind the pneumatic caisson resembles
In order to quantify the effect of the excess pore water pressure increase in the soils, a parameter study was performed by assuming all the soils were non-liquefiable. The computed seaward displacements were 0.31 m and 0.18 m at the top of the caissons, resulting in a total of 0.49 m. This is the displacement induced solely by the inertia force. Thus, the result of the excess pore water pressure increase in the soils is an increase in displacement to about twice of that induced purely by the inertia force due to the earthquake.

**BREAKWATERS**

A composite breakwater consists of a concrete caisson put on foundation rubble, with no backfill nor embedment, and thus different from caisson walls and pneumatic caisson foundations. Similar to the caisson walls mentioned earlier, the composite breakwaters in Kobe Port were constructed on a loose decomposed granite, which was backfilled into the area excavated from the original alluvial clay layer. The thickness of the clay layer reaches a maximum of 25 m in Kobe Port at the southern front, where the analyzed breakwater, the No. 7, was located.

A typical cross section and deformation of breakwater No. 7 are shown in Fig. 19. During the earthquake, the breakwater settled about 1.4 to 2.6 m. The horizontal displacements of the breakwater, however, were less than tens of centimeters. The mode of deformation suggests that the caisson was pushed into the rubble foundation and the rubble was also dragged down and pushed into the loose deposit below it.

Although the caissons used for the quay walls and the breakwaters were similar to each other, the modes of deformation were different. The effect of backfill or lack of it is presumed to be the cause of this fundamental difference. Another point of interest is the small horizontal displacement, compared to that of the pneumatic caisson foundations. Since the breakwater does not have an embedment portion, it is possible that a large horizontal...
displacement is induced if the shaking is strong enough. The horizontal displacement due to the earthquake was, however, only tens of centimeters as mentioned earlier, similar to that of the firmly embedded pneumatic caissons.

The behavior of the breakwater with respect to liquefaction was difficult to investigate. Using the acoustic technique sand boils were confirmed on the seabed at the surface of the loose deposit at the edge of the rubble foundation of a breakwater located nearby (Sekiguchi et al., 1996). This indicates that liquefaction occurred in this portion of the loose deposit. It remains to investigate how wide was the area of liquefaction in the loose deposit below the breakwater was or how high the excess pore water pressure increased.

These issues of interest will be discussed below based on the effective stress analysis. The analysis domain covers a cross sectional area about 200 m by 40 m. The parameters used for the analysis are shown in Table 6.

The analysis resulted in the deformation shown in Fig. 20. The deformation mode is consistent with those observed and mentioned earlier. The settlement at the top of the breakwater gradually accumulated and reached 1.3 m as shown in Fig. 21. The measured settlement mentioned earlier ranged from 1.4 m to 2.6 m, about 0.7 m larger than the result of the analysis. The difference amounts to about 3% of the thickness of the loose deposits below the breakwater. If settlement due to dissipation of excess pore water pressures is allowed in the analysis, this difference may be explained.

The computed horizontal displacement at the top of the breakwater is shown in the third row in Fig. 21, having a peak displacement of 0.6 m. The residual displacement, however, is much smaller. About 50% of the horizontal displacement is due to the rocking-like response of the caisson, as understood by the displacement difference between the top and bottom of the caisson shown in Fig. 22. This relative displacement corresponds to the inclination of the caisson if normalized by the height of the caisson.

The computed excess pore water pressure ratios at the end of the shaking are shown in Fig. 23. As shown in this figure, the excess pore water pressure ratios are higher than 0.8 for the area relatively far away from the caisson. However, the major portion of the foundation soils below and in the vicinity of the caisson, which most sensitively affect the behavior of the caisson, show excess pore water pressure ratios less than 0.4.

In order to discuss the details of the soil behavior, the computed stress paths and stress and strain relations for the soil (at K.P. -25 m) below the caisson are shown in Fig. 24. As shown in Fig. 24(a), the computed stress path is very close to the failure line even before the earthquake, and remains the same during the shaking. Thus, the initial deviator stress due to gravity is found to be the main mechanism to suppress the excess pore water pressures. Together with this deviator stress due to gravity, the cyclic loading gradually accumulates the axial strain.

**Fig. 18.** Computed stress and strain relationships of backfill soil behind the pneumatic caisson foundation: (a) stress path in $\tau - (\sigma_z - \sigma_y)$ plane, (b) stress and strain curve in $\tau_y - \gamma_{xy}$ plane, and (c) stress and strain curve in $\delta_y - (\varepsilon_x - \varepsilon_y)$ plane.
difference as shown in Figs. 24(c) and (d), whereas the simple shear strain $\gamma_{xy}$ remains within a level of 2% as shown in Fig. 24(b). Since both sides of the breakwater are open to sea water, there is no static horizontal load acting on the breakwater caisson. This may be the reason why $\gamma_{xy}$ remains within a limited level. This behavior of the soil below the breakwater caisson is similar to the soil behavior discussed earlier under the initial anisotropic stress condition.

**DISCUSSION**

As illustrated by the results of the effective stress analyses on seismic performance of caisson structures, including a caisson wall, a pneumatic caisson foundation and a breakwater, a variety of geotechnical conditions results in a variety of failure modes and varying degrees of deformation for these structures. For a caisson wall constructed on a firm non-liquefiable foundation, an increase in the earth pressure from the backfill plus the effect of an inertia force on the body of the wall cause the seaward movement of the wall. This is the situation most often considered in standard design practice. When the subsoil below the caisson wall is loose and excess pore water pressure increases in the subsoil, however, the movement of the wall is associated with a significant deformation in the foundation soil, resulting in much larger seaward movement involving a tilt and settlement. In this case, the wall movement generates a tensile normal strain in the backfill, reducing the excess pore water pressure to a low level, far from complete liquefaction in the vicinity of the wall. The deformation of the loose subsoil below the caisson wall is generated by the pseudo static deviator stress due to the gravity load from the caisson. The excess pore water pressure increase in the subsoil contributes to a cumulative increase in the axial

**Table 6. Model parameters for breakwater analysis**

<table>
<thead>
<tr>
<th>Soil type</th>
<th>$\rho$ t/m$^3$</th>
<th>$G_{ma}$ kPa</th>
<th>$\sigma_{ma}$ kPa</th>
<th>$\phi_f$ deg</th>
<th>$\phi_p$ deg</th>
<th>Parameters for dilatancy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation soil</td>
<td>1.8</td>
<td>115800</td>
<td>102</td>
<td>36</td>
<td>28</td>
<td>$w_f = 30, p_1 = 0.65, p_2 = 1.03, c_1 = 3.60, S_l = 0.005$ (assuming $d_{eq} = 0$ in Eq. (5))</td>
</tr>
<tr>
<td>Clay</td>
<td>1.7</td>
<td>74970</td>
<td>143</td>
<td>30</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Foundation rubble and rubble backfill</td>
<td>2.0</td>
<td>180000</td>
<td>98</td>
<td>40</td>
<td>—</td>
<td>(assuming $d_{eq} = 0$ in Eq. (5))</td>
</tr>
</tbody>
</table>

Friction angle at bottom of concrete caisson $\delta = 31$ deg  
Friction angle between concrete blocks $\delta = 15$ deg
strain difference of the soil along the direction of the principal stress axes of the pseudo static deviator stress. This soil behavior causes the settlements due to the compressive vertical normal strain. The seaward movement can also be induced in accordance with the tensile horizontal normal strain, provided the movement of the soil element toward the land side is restricted. These statements are supported by the effective stress analyses of the caisson wall on Rokko Island in Kobe Port.

The failure mechanism observed here is governed by two factors. One is the geotechnical condition of the foundation below the caisson, the other is the backfill condition. If a caisson structure is embedded in a firm foundation, the movement of the caisson will be more restricted than that of a caisson without embedment. In this case, the backfill soil will be much more restrictively con-

Fig. 20. Computed deformation of breakwater at the end of shaking: (a) Mesh deformation, (b) Displacement vectors
tained by the caisson structure, and can be brought to a state of complete liquefaction. These statements are supported by the effective stress analyses of the pneumatic caisson foundations for Kobe Bridge.

If a caisson structure is put on a loose saturated subsoil but is not pushed seaward by the earth pressure from the backfill, such as for a breakwater, the residual horizontal displacement of the caisson becomes smaller. Settlement, however, is accumulated with transient and cyclic loads.

Similar to the caisson wall, the gravity load from the caisson to the foundation soil below the caisson is the main driving force to induce the subsoil deformation. An increase in excess pore water pressure contributes to a cumulative increase in the axial strain difference of the soil along the direction of the principal stress axes of the pseudo static deviator stress. This soil behavior causes settlements due to the compressive vertical normal strain. These statements are also supported by the effective stress analyses of the breakwater in Kobe Port.

The common mechanism seen in these different port structures is the effect of gravity and the earthquake shaking. Gravity acts as the main driving force to induce deformation during earthquakes. This causes the initial shear in the soil, and then the cyclic shear effects and accumulate deformation associated with excess pore water pressure increase. In particular, the deformation of soils is gradually induced by accumulating the axial strain difference along the directions of the initial principal stress axes.

Degree of displacement, being one of the essential design parameters to characterize the seismic performance of port structures, is governed by the undrained behavior of sand discussed earlier. For adequate but economical design of waterfront retaining structures, this design parameter should be evaluated with a reasonable level of accuracy based on an appropriate characterization of the undrained behavior of sand and a reasonable seismic response analysis.

An analytical procedure to meet the engineering needs discussed above is emerging and available. The applicability of the analytical procedure to a variety of caisson type port structures, including a caisson wall, a pneumatic caisson foundation, and a breakwater, is encouraging. It is yet at an emerging state of development and there are still unknowns in the reliability of its predictive capability. In design of port structures, it is best to, at first, run the analysis on the seismic case history of a port structure having the design conditions similar to those of the structure to be designed, confirm the applicability of the analytical procedure and the geotechnical parameters, and then go on to vary the parameters of the analysis and the input earthquake motions to fit the specific conditions needed for the design.

**CONCLUSIONS**

The seismic response of a variety of structures in Kobe Port, including retaining walls and embedded and shallow foundations, were analyzed using the effective stress model based on a multiple shear mechanism. The conclusions obtained from the analyses are as follows.

1. The cyclic behavior of soil, idealized through the effective stress model, can reasonably explain the variety of seismic responses of the port structures analyzed, including the modes and extent of the deformation/failure.

2. Gravity acts as the main driving force to induce deformation during earthquakes. Transient and
Fig. 23. Computed distribution of excess pore water pressure ratios for breakwater

Fig. 24. Computed stress and strain relationship of the foundation soil below breakwater: (a) stress path in $\tau - (-\sigma_m')$ plane, (b) stress and strain curve in $\tau_{xy} - \gamma_{xy}$ plane, (c) stress and strain curve in $(\sigma_x - \sigma_y) - (\varepsilon_x - \varepsilon_y)$ plane, and (d) stress and strain curve in $\tau_{xy} - (\varepsilon_x - \varepsilon_y)$ plane.
cyclic shaking together with excess pore water pressure increase help to accumulate the deformation. In particular, deformation of soils are gradually induced by accumulating the axial strain difference along the directions of the initial principal stress axes.

(3) The initial deviator stress induced in the soil due to gravity significantly affects the excess pore water pressure increase. In many cases, the excess pore water pressure ratios in the soils, which most sensitively affect the stability/deformation of structures, never reach the 100% level. This is because the shear deformation/failure precedes the onset of liquefaction.

REFERENCES


