OBSERVED BEHAVIOUR OF A TUNNEL IN SAND SUBJECTED TO SHEAR DEFORMATION IN A CENTRIFUGE

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ABSTRACT

The paper describes observed behaviour of a model tunnel embedded in dry sand subjected to cyclic ground shear deformation in a centrifuge, as well as the behaviour of the model ground during shear deformation. Detailed data on earth pressures acting on the tunnel lining and the sectional forces of the lining are presented during ground shear deformation. The data suggest that the earth pressure at tunnel crown before ground shear deformation is smaller than the full overburden pressure probably due to the formation of arch action and the arch action may deteriorate with the cyclic ground shear deformation, resulting in an increase in the earth pressure at crown and changing the distribution of the sectional forces, which are largely influenced by conditions between tunnel lining and invert.

Key words: arching, centrifuge, cyclic shear, lining, sand, shear deformation, tunnel (IGC: H5).

INTRODUCTION

It is widely accepted from past experiences that tunnel structures during an earthquake are generally stable unless the tunnel is subjected to large ground deformation in a fault or fractured zone (Yashiro et al., 2007). The first reason for this behaviour is that, generally, the apparent weight of a tunnel structure including inner space is regarded as less than that of a corresponding volume of the surrounding ground. Thus, when the seismic forces act on the tunnel structure, the inertia force on the tunnel, which may cause tunnel deformation, is considered to be less than that on the surrounding ground. The second reason is that the surrounding ground in a deep tunnel usually has sufficient rigidity to keep the shape of the tunnel even during a severe earthquake. As a result, the behaviour of the tunnel under earthquake condition is regarded to be subordinate to the surrounding ground, provided that the tunnel lies below a certain depth at which it can be constrained by the surrounding ground, where sufficient confining effect can be obtained. Furthermore, seismic acceleration acting on the soil mass under the ground surface is smaller than the surface acceleration. Generally underground structures, therefore, have less impact from the effect of earthquake in comparison with superstructures (Kawashima, 2000).

Past experiences suggest, however, that underground structures at shallow depths in relatively soft layers suffer extensive seismic damage due to large shear deformation of the surrounding ground. For example, during 1995 Hyogoken-Nanbu Earthquake, Daikai station, one of subway stations in Kobe, experienced collapse of the centre pillars of RC box-type structure which was constructed by the cut-and-cover method. It is considered that the structure was subjected to larger shear deformation during the earthquake, causing the overstress at the connection between upper and lower slabs and some centre pillars, which tend to re-distribution of stress in the structure and to progressive failure of other pillars (Iida et al., 1996). Not only for the case of urban tunnels but also for mountain tunnels, damage was observed due to ground shear deformation especially at portal sections or in the linings near the portal (Asakura and Sato, 1996), where cracks appeared at the shoulders of the tunnels. This type of seismic damages were also observed in Inatori and Imahama railway tunnels located in the Shizuoka prefecture, during the Izu-Oshima-Kinkai Earthquake (Yoshikawa, 1979) and Uonuma tunnel in the Niigata-Chuetsu Earthquake which was the biggest disaster of all Shinkansen tunnels since the introduction of the system of the Shinkansen tunnels (Yashiro et al., 2007). More recently in the 2008 Iwate-Miyagi Nairiku Earthquake, Shin-Tamayama tunnel suffered a continuous transversal crack in the lining shown in Fig. 1.

On the basis of the seismic behaviour of the tunnel, for which the effect of the inertia force can be neglected, most of the current seismic design of the tunnel is conducted by the seismic displacement method (JSCE, 2008). This
method is a kind of pseudo-static approach in which earthquake-induced ground deformation is statically imposed on the tunnel structure through a soil spring system together with peripheral shear force at the interface between the structure and the ground. Although the need for seismic performance of the mountain tunnels at relatively shallow depths in unconsolidated ground is emphasized in JSCE standard specifications (JSCE, 2007a), it has been little known about the seismic behaviour of the mountain tunnel and no detailed design procedures have been established for the mountain tunnels. In particular, the initial stress conditions (i.e., the earth pressures acting on the tunnel structure) are not well-known, and it is expected to occur that the stress conditions of the tunnel have been changed by imposing the earthquake-induced cyclic shear deformation of the ground. Therefore it is required to understand detailed mechanisms of the soil-structure interaction under various ground conditions and geometric constraints. Physical modelling has been adopted to obtain the basic understanding of the complicated seismic behaviour of the tunnels. Kusakabe et al. (2008) presented a direct comparison between centrifuge pseudo-static active type shear box test and centrifuge dynamic shake table test on the same prototype tunnel model. According to direct comparison, the active type shear box test can be a practical substitute for the dynamic shake table test for the study of dynamic behaviour of underground structures such as tunnel.

In this study, a series of centrifuge pseudo-static active type shear box tests on a horseshoe-shaped tunnel in dry sand were conducted. In these tests, cyclic shear deformation was statically imposed on the tunnel structure through the model ground in the centrifuge.

Generally, due to concentration of the bending moment, a joint between the tunnel lining and the invert is structurally weak, thus it has a possibility to become a plastic hinge. If the joint has become the plastic hinge, the bending moment cannot be transferred at the joint between the lining and the invert of the tunnel. In this study, two types of tunnel models with firmly fixed and hinged conditions at the joint between the lining and invert were used to investigate the effect of fixity condition on changes of earth pressure acting on tunnel lining. Actual seismic response of the tunnel might exist between the behaviours of the two models.

**MODEL AND TEST DESCRIPTIONS**

The geotechnical group at the Tokyo Institute of Technology developed the active type shear box apparatus for centrifuge model tests (Takahashi et al., 2001). The original shear box was modified to increase the depth of the shear box to accommodate a larger cover depth, since a tunnel cover ($C/D$) ratio is an important factor for the stress condition around the tunnel (Takemura et al., 2006). The modified shear box with inner dimensions $W_{452} \times B_{202} \times H_{524}$ mm is presented in Fig. 2. Major modifications of the new shear box are the height of the laminar box and number of actuators. Number of the laminae was increased from 13 to 21 and the height of the box from 325 mm to 524 mm. The increase in the height is equivalent to $C/D$ ratio of two for the model tunnel used in this study ($D = 100$ mm). One hydraulic actuator for displacing the laminae was added to the original shear box which had three actuators. In total four actuators are directly connected to four laminae. The displacements of the actuators were imposed on other laminae through plate springs attached to both sides of the
shear box. The specifications of the modified shear box are shown in Table 1.

For modelling a horseshoe-shaped mountain tunnel in sandy ground in prototype, an aluminium model tunnel was manufactured as is shown in Fig. 3. The model tunnel had a diameter of 100 mm, a height of 75 mm and a thickness of 2 mm with smooth surface. These model dimensions approximately correspond to a 5 m diameter tunnel with a RC lining of 300 mm thickness in the prototype scale under 50 g.

Two types of the semi-circular model tunnels were made to examine the effect of the fixity condition at the lining end contacting the invert. Figure 4 shows the two types of arrangements for the lining end conditions; the tunnel lining and the invert are firmly fixed (i.e., welded model) and the end condition modelled as the plastic hinge, in which only the axial force can be transferred (i.e., non-welded model). The welded model had the lining both ends of which were firmly welded to a 2.5 mm thick steel plate as shown in Fig. 4(a), whereas the non-welded model had an end condition where both ends of the lining were cut in a radial direction and contacted onto 90° V shape notches of a 5 mm thick steel plate as is illustrated in Fig. 4(b). The plate was bolted onto a 15 mm thick steel plate placed on the base of the laminar box.

Figure 5 illustrates the position of these sensors attached to the model tunnel with the detailed illustration of the arrangement of the earth pressure cell. Five earth pressure cells of 6.2 mm diameter and 0.75 mm thick were embedded on the outer surface of the model tunnels to measure the earth pressures acting on the lining. Measurement points of the earth pressures were at the tunnel crown, spring lines and the midpoints between the crown and the spring lines. A thin layer of silicon rubber was coated over the surface of the earth pressure cells. 22 strain gauges in total were also attached on both outer and inner sides in pairs for each model tunnel to measure sectional forces of the tunnel lining. A laser displacement transducer and two potentiometers were installed inside the model tunnel to measure vertical and horizontal convergences, respectively.

The model setup is illustrated in Fig. 6. The model
setups and test procedures were identical to all the cases of the welded and the non-welded models. Dry silica sand No. 6 was used for making the model ground. Material properties of the silica sand No. 6 are shown in Table 2. After the model tunnel was placed and fixed to the steel plate on the base of the laminar box, the model ground was made by air pluviation method to achieve a relative density of 80%. The height of the model ground was selected to be 375 mm, creating a cover, C, of 300 mm which corresponds to a cover-diameter ratio, C/D, of 3. The vertical and horizontal displacements were measured at the center of ground surface by a potentiometer and a laser displacement transducer, respectively. The potentiometer was attached to the top laminar ring and the laser displacement transducer was bolted on an outer frame, measuring the displacement relative to that of the ring as illustrated in Fig. 6. Once the whole model preparation was completed, the model was placed on a centrifuge platform and spun up to 50 g using the Mark III centrifuge at the Tokyo Institute of Technology (Takemura et al., 1999). Hereafter, the data of the displacements are presented in the model scale unless otherwise stated.

Figure 7 presents a typical example of the input horizontal displacement distributions of the laminar rings measured by potentiometers at nominal 1%, 2% and 4% shear strains both on the positive and negative side. The measured distributions may be considered a linear distribution with the depth, in particular, at the depth of the tunnel, although there are some kinks between the actuators at the upper part of the model ground compared with intended distributions which are indicated by dotted lines with open circles. The input nominal shear strain time histories are shown by a dotted curve in Fig. 8. The two cycles of sinusoidal nominal shear strain for the amplitudes of 1%, 2% and 4% respectively, were statically imposed on the model ground continuously in the centrifugal acceleration of 50 g. Each cycle of shear strain was applied in 100 sec.

In Fig. 8, the sinusoidal waves shown by a solid curve...
indicate nominal shear strain histories at the ground center. The values of shear strain were calculated from the measured horizontal displacements at the center of ground surface divided by the height of the model ground. The nominal shear strain at the center is consistently about 60% of the input nominal shear strain throughout the shearing histories.

Unlike the earthquake-induced shear deformation, the shear deformation of the model ground in this active type shear box is given only by the displacement of the side boundaries. Tateishi (1995) suggested by comparative results of numerical analysis that a static FE analysis with this type of the loading condition, in which the ground is modelled by solid elements, cannot create appropriate earthquake-induced shear strain in the ground, i.e., the shear strain varies with the distance from the boundaries on which the displacement is given. The previous experimental studies with the active type shear box reported the similar results; Takahashi et al. (2001) used the original shear box and reported 70–80% of the input nominal shear strain under the conditions of 325 mm deep Toyoura sand of relative density of 80% under 25 g. Yamada et al. (2002) also used the original shear box and reported 70% under the conditions of 300 mm deep Toyoura sand of relative density of 90% under 50 g. Although the test conditions and material properties are different in these tests, the general trend is similar to the previous observations. However, in both the tests, even though the magnitude of the horizontal displacement was smaller than that at the side boundaries, the horizontal displacement profile at the center of the box was similar to that at the side boundaries. These suggest that the shear strain around the center of the box can be evaluated by the nominal shear strain at the side boundaries. Thus, it can be said that the prescribed shear strain can be imposed on the tunnel structure and the active type shear test can be regarded as the seismic displacement method.

Figure 9 presents the change in vertical displacement at the ground surface plotted against horizontal displacement during the whole process of centrifuge operation. During the increasing centrifugal acceleration stage (termed increasing g stage, hereafter) the ground surface settled by 0.83 mm (in ratio to the tunnel cover; $S_{\text{max}}/C = 0.27\%$) for the welded model and 1.40 mm ($S_{\text{max}}/C = 0.47\%$) for the non-welded model, which corresponds to the decrease in void ratio by 0.004 and 0.007, respectively. The observed settlements during increasing g stage from 1 to 50 g are the same order of magnitude reported by Ueno et al. (1994) who measured the settlements of dry sand layer during the whole process of centrifuge operation including a several cycles of increasing g and decreasing g stage.

During the shearing cycle stage, the ground surface gradually settled to 3.83 mm for the welded model and 4.98 mm for the non-welded model, when the model ground was sheared at the end of 2% shear strain cycle.
But a slight heaving was noticed when 4% shear strain was applied and the final settlement reached 6.05 mm for the welded model and 7.58 mm for the non-welded model. These accumulated settlement resulted in an increase in relative density by 7.71% and 9.66%, respectively, at the end of the shearing history. The ground responses during the shear deformation are apparently similar to the volume change during triaxial cyclic loading test (Tatsuoka and Ishihara, 1974). It is noted in Fig. 9 that the horizontal displacement cycle is not exactly symmetric and slightly off-centre toward the negative side, probably due to the characteristics of this particular shear box and loading system. The conditions for all the test cases are summarized in Table 3.

RESULTS AND DISCUSSION

Initial Earth Pressure

During the increasing g stage, the model ground will settle due to an increase of self-weight of soil. As described in Fig. 9, the settlement at ground surface was in the order of 1.0 mm, which is equivalent to 0.3% of compressive strain in the ground. Associated with the ground settlement, the model tunnel will deform in the vertical direction due to increasing overburden pressure.

Figure 10(a) displays relationships between the measured earth pressure at crown and the vertical convergence measured at crown, δv, divided by the tunnel diameter, D, for both test cases during increasing g stage. In Fig. 10(a), solid triangles and circles indicate the measured values of the welded and non-welded models, respectively. For both the models, the relationships between the vertical earth pressure and the vertical convergence are approximately linear during this stage, implying that the tunnel exhibits the elastic response. The measured vertical convergence, δv, at the end of increasing g stage, was 0.19 mm (δv/D = 0.19%) for the welded model and 0.39 mm (δv/D = 0.39%) for the non-welded model, respectively. In relation to the tunnel height, h = 75 mm, these vertical convergences are δv/h = 0.25% for the welded model and δv/h = 0.52% for the non-welded model, which can be considered as compressive strains of the model tunnels. As compared with compressive strains in the ground (i.e., Smax − δv/C) of 0.21% for welded model and 0.34% for non-welded model, the compressive strains of the both tunnel models are larger than these in the ground. It is suggested that the ground above the tunnel model was loosened (or the loosening area was created in the ground above the model tunnel). And it is expected that the amount of the loosening for non-welded models was more remarkable than that of the welded model.

Table 4 summarizes the measured vertical convergence.
measured at crown, $\delta_c$, and right and left horizontal convergences at the spring lines, $\delta_h$ and $\delta_l$, and surface settlement of the ground at the center, $S_{\text{max}}$, at the end of the increasing g stage for both cases. It is noticed that the vertical convergence was slightly larger than the horizontal convergence and there was a slight un-asymmetry in values of the horizontal convergence. The reason for this un-asymmetry may be due to the characteristics of the system as was pointed out earlier.

A simple elastic analysis was carried out, as is illustrated in Fig. 11, where the welded and the non-welded models were idealized as an arch shape elastic beam structure with the lining end condition either fixed or hinged, respectively. The vertical convergences at the tunnel crown were calculated by imposing a uniformly distributed vertical pressure over the idealized model tunnels without horizontal confining forces from the surrounding ground. The calculated results are also shown in Fig. 10, in which solid lines with open triangles and circles indicate the welded and the non-welded case, respectively. The gradients of these calculated lines indicate the structural stiffness of the model tunnel in the vertical direction. The elastic analysis well predicts the response of the non-welded model, but underestimates the vertical stiffness of both models, probably due to the fact that the analysis ignored the horizontal confinement of the surrounding ground.

Figure 10(b) shows the data at the end of the increasing g stage, which mean the initial vertical earth pressure acting on the lining at crown before ground shear deformation. In Fig. 10(b), the earth pressures were normalized by the full overburden pressure at the tunnel crown in centrifugal acceleration of 50 g. The normalized earth pressures at the end of increasing g stage are 0.56 for the welded model and 0.26 for the non-welded model. This observation should be considered with the fact that the vertical convergence of the non-welded model was twice larger than that of the welded model.

During construction process of the tunnel, the ground response ahead of the tunnel often idealized by elasto-plastic analysis of a contracting cylindrical cavity (e.g., Mair, 2008), the deformation of the tunnel will decrease the earth pressure above the tunnel crown. Elasto-plastic analysis (Hughes et al., 1977) provides the relationship between the earth pressure and the radial displacement as

$$p - \sigma_v = 2G\varepsilon_c \rho_0 / \rho: \text{Elastic state}$$

$$\ln \left( \frac{p - u_0}{\rho} \right) = s \ln \left( \varepsilon_c + \frac{c_{\text{int}}}{2} \right) + A: \text{Elasto-plastic state}$$

where

$$G = \frac{E}{2(1 + v)}, \quad \varepsilon_c = \frac{K_0}{1 + K_0} \varepsilon_r, \quad \sigma_v = \frac{\rho_0}{\rho}, \quad s = \frac{(1 + \sin \psi) \sin \phi}{1 + \sin \phi},$$

$p$: pressure, $\sigma_v$: initial pressure (i.e., the full overburden pressure at crown; $\gamma_0 C$), $G$: shear modulus of ground, $E$: Young's modulus of ground, $v$: Poisson's ratio of ground, $K_0$: earth pressure at rest, $c$: radial displacement, $\rho_0$: initial radius, $\rho$: current radius, $u_0$: pore water pressure, $\phi$: angle of friction, $A$: constant value, $\sin \psi$ and $c_{\text{int}}$: gradient and intercept of relationship between volumetric and shear strain, respectively, which are obtained by a series of consolidated drained triaxial compression tests.

Pressure at the intersection between elastic and elasto-plastic state was determined by the following equation,

$$p = \sigma_v (1 - \sin \phi)$$

From the series of CD triaxial tests on the silica sand, the following values were obtained as the input data for the Eq. (2): $E = 33 \text{ MPa}$, $\phi = 41^\circ$, $K_0 = 1 - \sin \phi = 0.34$ obtained from Jaky's formula, $\rho_0 = 50 \text{ mm}$, $\sin \psi = 5.22$, $c_{\text{int}} = 0.055$ and $u_0 = 0 \text{ kPa}$. The relationship predicted by the elasto-plastic analysis is also given in Fig. 10(b), as a solid line with open squares, which generally follows the experimental observations. From the comparison between the calculated relationship and the measured data, it can be considered that the initial state of the model ground for the welded model case is in the elastic state, while that for the non-welded case is in the elasto-plastic state.

The concept of loosening earth pressure was originally proposed by Terzaghi (1948), considering arch action above the tunnel. The concept has been adopted in the current tunnel design methods in Japan, including deep shield tunnel and mountain tunnel in sandy ground (JSCE, 2007a, 2007b). The loosening earth pressure can be calculated by Eqs. (4) and (5),

$$\sigma_l = \frac{B(\gamma - c/B)}{K \tan \phi} \left( 1 - e^{-K \tan \phi (c/B)} \right)$$

$$B = \frac{D}{2 \cot \left( \frac{\pi/4 + \phi/2}{2} \right)}$$

where $\sigma_l$: loosening earth pressure, $\gamma$: unit weight of
ground, \( c \): cohesion of ground, \( K \): coefficient of lateral earth pressure, \( \phi \): angle of shearing resistance of ground as shown in Fig. 12. The \( K \)-values were selected to be 1.0 suggested by Terzaghi (1948) and 0.34 obtained from Jaky’s formula. By substituting \( C = 15 \) m \((= 0.300 \text{ m}^2 \times 50 \text{ g})\), \( D = 5 \) m \((= 0.100 \text{ m}^2 \times 50 \text{ g})\), \( K = 1.0 \) or \( 0.34 \), \( c = 0 \) kPa, \( \gamma = \gamma_d = 15.8 \text{ kN/m}^3 \) and \( \phi = 41^\circ \) into Eqs. (3) and (4), the calculated loosening earth pressures are obtained and plotted in Fig. 10, as the long dashed double dotted line \((K = 1.0)\) and the long dashed dotted line \((K = 0.34)\). The initial earth pressure at the crown of the non-welded model is close to the predicted loosening earth pressure assuming \( K \) being 1.0, whereas the earth pressure of the welded model is closer to the prediction with \( K \) being 0.34. It should be noted that the state for the welded model can be expected in the elastic state as mentioned above.

In this particular centrifuge modelling, the deformation of tunnel before shearing occurred simply by increasing overburden pressure, which is obviously different from actual construction sequence of mountain tunnel. In practice, the tunnel lining of the mountain tunnel is constructed after the displacement of the ground has converged. It may be appropriate to compare the present observation with another centrifuge results which did simulate the tunnel excavation process. Kadoyu et al. (1993) conducted centrifuge model tests in which tunnel excavations were simulated by discharging the water in a rubber bag under the centrifugal acceleration of 80 g. The initial tunnel diameter, \( D \), was 50 mm and the tunnel cover-diameter ratios, \( C/D \), were 2 and 4, and the model ground was constructed by Toyoura sand of relative density of about 40%. Their results showed that the earth pressures acting on the tunnel reached minimum values when radial displacement was \( 0.25 \text{ to } 0.50 \text{ mm} \). These values corresponded to \( \delta_r/D = 0.5 \text{ to } 1.0\% \), and the measured minimum earth pressures were almost equal to the Terzaghi’s loosening earth pressure, assuming \( K = 1.0 \). Therefore the initial earth pressure at crown and the vertical convergence in this series of test using the pseudo-static shear box are reasonably consistent with the previous study which modelled the construction process of tunnels in dry sand.

Figure 13 presents the distributions of the initial normalized earth pressures measured by five earth pressure cells (EPC1 to EPC5) for both welded model (shown in a solid line with solid triangles) and non-welded model (shown in a broken line with open circles). All the earth

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**Fig. 12. Loosening earth pressure (JSCE, 2007b)**

![Diagram showing loosening earth pressure](image)

**Calculation of earth pressure distribution**

\[
s_{\text{r}} = \frac{1 + K}{2} \sigma + \frac{1 - K}{2} \cos 2\theta\sigma
\]

\( \gamma_d \): Unit weight of ground

\( \theta \): Angle from horizontal axis

\( K \): Coefficient of lateral earth pressure

\( C \): Tunnel cover

\( z \): Depth from tunnel crown

\( \sigma_{r} \): Earth pressure at tunnel crown

- Full overburden pressure: \( \gamma_d C \)

- Loosening earth pressure: \( \sigma_{r} \)

**Fig. 13. Distributions of initial earth pressures**

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pressures around the tunnel lining of the non-welded model were smaller than those of the welded model.

In Fig. 13, there are two thin broken lines and two dashed lines with shades which are obtained from Eq. (6), assuming the earth pressure acting on the tunnel crown to be the full overburden pressure and the loosening earth pressure with the coefficient of lateral earth pressure \( K \) to be 1.0 and 0.34, respectively.

\[
\sigma_r = \frac{\sigma_v + \sigma_h}{2} - \frac{\sigma_v - \sigma_h}{2} \cos 2\theta = \left( \frac{1 + K}{2} - \frac{1 - K}{2} \cos 2\theta \right) \sigma_v \tag{6}
\]

where \( \sigma_r \): earth pressure in the radial direction, \( \sigma_v \): vertical earth pressure \( \sigma_v = \sigma_c + \gamma_d z \), \( \sigma_h \): earth pressure acting on the tunnel crown (i.e., full overburden pressure or loosening earth pressure), \( \gamma_d \): unit weight of the ground, \( z \): depth from tunnel crown, \( \sigma_c \): horizontal earth pressure \( \sigma_c = K \sigma_v \) and \( \theta \): angle from the horizontal axis in the anti-clockwise direction, as shown in Fig. 13.

Comparing the measured values with the calculated values, it is known that in the case of the non-welded model, the measured distribution is almost the same as the calculated distribution assuming the loosening earth pressure with \( K = 1.0 \), while in the case of the welded model the measured values are in the range between the two broken lines calculated by full overburden pressures, except that the measured value at the tunnel crown (i.e., EPC2) is almost the same as the calculated loosening earth pressure with \( K = 0.34 \).

**Change in Earth Pressure during Shear Deformation**

Figure 14 shows the time histories of the earth pressure at the tunnel crown measured by the earth pressure No. 3 (EPC3) during the shearing processes of three different amplitudes \( \gamma_{\text{amp}} \) separately for both cases of the welded and the non-welded model. Here, the measured earth pressures are also normalized by the overburden pressure at the tunnel crown, \( \gamma_d C \). The input nominal shear strain as the dashed wavy lines and the calculated earth pressures at the tunnel crown are also drawn in the figure for reference.

As was already pointed out, the initial earth pressures at crown before the first shearing \( \gamma_{\text{amp}} = 1\% \) were smaller than the overburden pressure at crown. This is probably due to the formation of the arch action above the tunnel crown during the period of increasing \( g \) stage. For the case of the welded model shown by a solid line in Fig. 14, the earth pressure at crown significantly increased by the positive 1\% shear strain application and then slightly decreased, associated with the negative 1\% shear strain application during the first cycle. In the second cycle, the earth pressure further increased up to the full overburden pressure and then decreased, as the negative shearing process proceeded. For the case of the non-welded model shown in a dotted line, the earth pressure progressively increased regardless of the sign convention of the shear strain until the end of the second cycle of the 1\% shear strain application. During the 2\% shear strain application, the variations of the earth pressures became small within a range of 20\% of the normalized earth pressure. The earth pressures were very close to the full overburden pressure and vary within an even smaller range during the 4\% shear strain application.

The results suggest that the deterioration of the arch action above the tunnel crown was taking place by the cyclic application of shear deformation. It is more marked for the case of the welded model than the non-welded model.
Figure 15 shows the time histories of the normalized earth pressure at EPC2 and EPC1, in which the time histories of the input nominal shear strain are also represented by thin dashed lines. It is observed from the figure that the earth pressures not only at crown but also at spring line and at shoulder increased with the application of shear deformation, although the data at spring line exhibit considerable fluctuation.

Table 5 summarizes the earth pressure increase at three different locations (EPC1, 2 and 3) during the two cycles of shear deformation for three different strain levels. The $\gamma_{amp}=1\%$ shearing event is the largest influence on the increase in earth pressures for all locations of the tunnel lining, regardless of the lining end fixity condition.

Figures 16 and 17 compare the distributions of the normalized earth pressure ($\sigma/\gamma_{C}$) measured before and after two cycles of shear deformation for the cases of 1% and 4% shear strain, respectively, both for the welded and non-welded model. The term ‘before’ in these figures corresponds to the test elapsed time of 0 sec in each shearing event as is indicated in Fig. 15. The calculated earth pressure distributions of the full overburden pressures for $K=1.0$ and 0.34 were also plotted in these figures. It is seen in Figs. 16(a) and (b) that the normalized earth pressures around the tunnel, especially the upper part of the tunnel lining, increased after the $\gamma_{amp}=1\%$ shearing for both models. This observation must be associated with the deterioration of the arch action.

<table>
<thead>
<tr>
<th>Model</th>
<th>EPC No.</th>
<th>Position</th>
<th>1% shearing</th>
<th>2% shearing</th>
<th>4% shearing</th>
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<tr>
<td>EPC1</td>
<td>Welded</td>
<td>S.L.</td>
<td>14.01</td>
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<td>11.13</td>
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<tr>
<td></td>
<td></td>
<td>Mid</td>
<td>133.62</td>
<td>61.77</td>
<td>-5.84</td>
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<tr>
<td></td>
<td></td>
<td>Crown</td>
<td>109.58</td>
<td>11.43</td>
<td>4.24</td>
</tr>
<tr>
<td>EPC2</td>
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<td>S.L.</td>
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<td>-24.65</td>
<td>9.36</td>
</tr>
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<td></td>
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<td>91.71</td>
<td>57.04</td>
</tr>
<tr>
<td></td>
<td></td>
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<td>136.26</td>
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<tr>
<td>EPC3</td>
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<td>Crown</td>
<td>136.26</td>
<td>7.77</td>
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</table>
Fig. 18. Distributions of initial sectional forces

A comparison of the values and the distributions of earth pressure before and after $\gamma_{amp} = 4\%$ shearing presented in Figs. 17, however, show no appreciable change. This experimental fact implies that the arch action was almost fully deteriorated by several shearing events up to 2% shear deformation and the earth pressure distribution reached a certain distribution in which the earth pressures at the tunnel shoulders were the largest. This is more marked for the case of the welded model.

Changes in Sectional Forces

Figures 18 show the initial distributions of bending moment and axial force in the lining for both welded and non-welded model. Figure 18(a) indicates that the bending moment distributions were almost the same for the welded and non-welded model, except the points nearer to the contact between the lining and the invert plate. The bending moments at both ends were very close to zero for the non-welded model and showed negative for the welded model. The axial force distributions are slightly different between the welded and non-welded model. The axial force distribution of the welded model had the peak values at shoulders and the axial forces at the ends were almost zero, while the axial forces of the non-welded model showed maximum values at both ends of the lining. This observation may be related to the magnitude of the vertical convergence.

Figures 19 and 20 show the time histories of the bending moment and axial force at the crown (SG6), right spring line (SG1) and the midpoint between them (SG4). In these figures, the thin dashed lines indicate the time histories of the input nominal shear strain. Changes in bending moment and axial force at crown (SG6) were not significantly different between the two models, but those at SG4 and SG1 indicate clear differences between the two models. At SG4 and SG1 the changes in axial force of the non-welded model were more than those of the welded model, whereas the opposite trend is noticed in the bending moment. Peak values of both the bending moments and the axial forces appeared when maximum negative shear strain of the second cycle of the $\gamma_{amp} = 4\%$ shearing event was imposed on the models. In other words, the bending moment and the axial force at one side of the shoulder and spring line became peak values when the shear forces are detected on the same side.

Figure 21(a) shows the distributions of absolute earth pressures measured at the input nominal shear strain $\gamma = -4\%$ in the second cycle of the $\gamma_{amp} = 4\%$ event, when the sectional forces show the peak values. Figure 21(b) shows the distributions of incremental values measured from the end of the 2% shear cycles to the same point of Fig. 21(a). There was little difference in the incremental values compared with the difference in the absolute pressures; the difference of the distributions of the absolute earth pressure was caused by the initial values before shearing. Figure 22 shows the distributions of the bending moments and the axial forces measured when the sectional forces acting on the lining shoulders show the peak values. The previous experiments (Yamada et al., 2002; Izawa et al., 2006) indicated that sectional forces at the corners are remarkable when pseudo-static shear deformation of the ground was imposed on the model tunnel. The present data shown in Fig. 22 also support the previous experimental observations. As shown in Fig. 22(a), the bending moment distributions of the both the welded and non-welded models were almost the same. However, the values of the bending moment at the right end of the lining were quite different between the non-welded and the welded models. The bending moment at the left end of the welded model was almost equal to the peak value of the bending moment at the right shoulder. It is also noticed that the bending moment at both side ends for the non-welded model exhibited some value, suggesting that the end conditions of the non-welded model might not be the same as an ideal hinge condition. As is seen in Fig. 22(b), the peak axial force of the non-welded model measured at the right shoulder was larger than that of the welded model. These results indicate that if the tunnel linings are rigidly fixed to the invert, the bending moments are significant at the ends of the lining when the earthquake-induced shear deformation of the ground was imposed. And if the lining ends have rotational freedom, larger axial forces appear at the lining shoulders when the shear deformation of the ground was imposed.
Fig. 19. Time histories of bending moment

Fig. 20. Time histories of axial force

Fig. 21. Distributions of earth pressure when sectional forces show peak values
CONCLUSIONS

This paper presented the observed behaviour of a tunnel embedded in dry sand subjected to a cyclic ground shear deformation in a centrifuge. The following findings were obtained:

1) The initial earth pressure acting on the tunnel crown before the first shearing is smaller than the full overburden pressure probably due to the formation of arch action. The formation of arch action is strongly affected by deformation of the ground and the structural rigidity of the tunnel.

2) The deterioration of arch action of the ground above the tunnel occurs by the ground shear deformation. Full overburden pressures are recommended to use for tunnels in relatively shallow sandy ground, when the seismic condition is required to be considered.

3) If the tunnel lining is rigidly fixed to the invert, the seismic response of the bending moment at the lining ends is significant as well as the tunnel shoulders. And if tunnel lining has the rotational free end condition, the seismic responses of the axial force at the shoulders are larger than that of the tunnel with the fixed end condition.

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