TRANSVERSE HORIZONTAL LOAD ON BURIED PIPES DUE TO LIQUEFACTION-INDUCED PERMANENT GROUND DISPLACEMENT

KAZUNORI SHIMAMURA, YUSUKE FUJITA, SEIJI KOJIMA, YOUCI TAJI and MASANORI HAMADA

ABSTRACT

This paper investigated the transverse horizontal load on buried pipes due to liquefaction-induced permanent ground displacement in the horizontal direction. The focus of study was on the maximum load. For this purpose, liquefaction-induced permanent ground displacement in areas behind quay walls and in inland sloping ground areas was reproduced in a model ground and the load exerted on pipes buried in this ground area was measured. The investigation employed dynamic centrifuge model tests. All pipes were buried in non-liquefaction layers situated above liquefaction layers. All of the obtained load due to liquefaction-induced permanent ground displacement in the horizontal direction were much smaller than the load obtained by moving pipes forcibly in a static ground. The ratio of the maximum load due to liquefaction-induced permanent ground displacement to that obtained by moving pipes forcibly was about 0.3-0.4. There is a possibility that a maximum load corresponding to the residual strength of the ground was exerted on the pipes. The ground displacement patterns suggest that large shear strain was generated on the rear side area of the pipe. The cracks on the front side area of the pipe may have decreased the confining pressure of the ground and resulted in the decrease of the load. The tensile strain generated in the ground may have decreased the confining pressure and resulted in a decrease in the load as well.

Key words: centrifuge model test, earth pressure, lateral flow, liquefaction, (pipeline), underground pipe/conduit (IGC: E12/E14/H8)

INTRODUCTION

Transverse horizontal load refers to the horizontal load perpendicular to the pipe axis. For making detailed design of pipelines crossing areas having lateral spreading potential due to possible future earthquakes, it is crucial to evaluate rationally the transverse horizontal load exerted on pipes due to liquefaction-induced permanent ground displacement in the horizontal direction, as transverse horizontal load has a large effect on pipeline deformation. In the case of large deformation in which plastic hinges occur in pipelines, the deflection angle of infinite straight steel pipelines due to transverse horizontal ground displacement is proportional to the square root of the maximum transverse horizontal load (Takada et al., 2000). The deflection angle of a pipe bend which has infinite straight pipelines at both ends decreases to 60% in the case of closing modes, and to 70% in the case of opening modes when the maximum transverse horizontal load decreases to 30% (Takada et al., 2001).

A number of previous researchers have investigated transverse horizontal restraint force on either horizontal pipes or vertical anchor plates buried in a model ground without liquefaction layers. Audibert et al. (1975 and 1977) and Trautmann et al. (1985b) gave excellent reviews of previous research. With respect to present seismic design guidelines, the horizontal restraint force factors proposed by Trautmann et al. (1985b) and Hansen (1961) are illustrated in "Guidelines for the Seismic Design of Oil and Gas Pipeline Systems" (ASCE, 1984). Those proposed by Trautmann et al. (1985b) are adopted in the seismic design of high-pressure gas pipelines for wave propagation in Japan (Japan Gas Association, 2000).

It should be noted that the depth of the crown of pipelines is generally 1.5 m–1.8 m. At this depth, pipelines may be buried in liquefaction layers or in non-liquefaction layers situated above liquefaction layers. Therefore, the load on pipes buried in non-liquefaction layers should be adopted in design, since a larger load is exerted on pipes in this case. The assumption in this paper that pipelines are buried in non-liquefaction layers is consistent with previous research concerning transverse horizontal

1) Tokyo Gas Co., Ltd., Pipeline Department, 5-20, Kaigan 1-chome, Minato-ku, Tokyo 105-8527.
2) Osaka Gas Co., Ltd., Engineering Department, 1-2, Hiranomachi 4-chome, Chuo-ku, Osaka 541-0046.
3) Toho Gas Co., Ltd., Distribution Planning and Administration Department, 19-18, Sakurada-cho, Atsuta-ku, Nagoya 456-8511.
4) Shimizu Corporation, Institute of Technology, 4-17, Etcubuna 3-chome, Koto-ku, Tokyo 135-8530.
5) Professor, Waseda University, 4-1, Ohkubo 3-chome, Shinjuku-ku, Tokyo 169-8555.
Manuscript was received for review on October 24, 2001.
Written discussions on this paper should be submitted before September 1, 2003 to the Japanese Geotechnical Society, Sagayama Bldg. 4F, Kanda Awaji-cho 2-27, Chiyoda-ku, Tokyo 101-0063, Japan. Upon request the closing date may be extended one month.
load. However, in previous research, the load has been evaluated mainly by the tests in which model pipes have been moved forcibly in the transverse horizontal direction in a static model ground.

In design, pipelines are buried continuously from a static ground area, through a permanent ground deformation area and to a static ground area, again. In designing for permanent ground displacement, it is necessary to know the restraint force characteristics in both areas. The results of the pipe movement tests should be applied to the pipeline portion buried in a static ground area where pipelines are forcibly moved by the movement of the continuous pipeline portion buried in a permanent ground deformation area and where the resistance force from the static ground is exerted on pipes. If the same restraint force is applied in a loading portion buried in a ground deformation area, estimation of the pipeline deformation may be inaccurately conservative. As Honegger (1992) stated, the decrease in soil restraint that may result from soil disturbance or the effects of liquefaction is not accounted for in this case.

The effects of liquefaction-induced ground displacement have been studied in the area of piles. Horikoshi et al. (1998) investigated the performance of a single pile subjected to liquefaction-induced ground displacement in areas behind quay walls by carrying out dynamic centrifuge model tests. They reported that the reduction in soil modulus in the non-liquefaction layer near the quay wall might be an important aspect in estimating the pile response. Azuma et al. (1998) reproduced liquefaction-induced ground displacement in areas behind quay walls by shaking-table tests and investigated the effects on bridge foundations. They obtained the experimental results showing that the equivalent linear soil spring coefficient in non-liquefaction layers decreased to about 1/50 at a horizontal displacement of 1 cm, and to 1/200 at a horizontal displacement of 10 cm during lateral spreading. Although these results are not applicable to pipelines directly, they suggest the possibility of decrease in soil restraint force on pipelines buried in a ground deformation area.

This paper investigated the transverse horizontal load on buried pipes due to liquefaction-induced permanent ground displacement in the horizontal direction. The emphasis of investigation was the maximum load. For this purpose, liquefaction-induced permanent ground displacement was reproduced in a model ground and the load exerted on pipes buried in this ground area was measured.

In past earthquakes, liquefaction-induced permanent ground displacement was observed in ground areas behind quay walls and inland sloping ground areas. Thus, model grounds were made reflecting these two types of ground areas. The investigation employed dynamic centrifuge model tests. Specifications of the centrifuge have been described by Sato (1994) and Sato et al. (1995). As mentioned before, all pipes were buried in non-liquefaction layers situated above liquefaction layers.

CENTRIFUGE MODEL TESTS

Pipe Movement Tests

In order to confirm the applicability of centrifuge model tests to the soil-pipe interaction problem, the pipe was forcibly moved in the transverse horizontal direction in a static ground in a centrifuge acceleration field. Figure 1 shows a vertical view and a plan view of the test equipment.

A model ground was made in a space partitioned by an aluminum wall plate with a thickness of 20 mm and rigid boxes in a rigid container. The wall plate and the rigid boxes were fixed rigidly in the container. A rubber membrane was placed on the model ground side of the rigid boxes with silicon oil in order to decrease wall friction. Table 1 shows the test conditions of the pipe movement tests. Table 2 shows the scale ratios satisfying the similarity law at centrifuge model tests. Hereafter, length, force, time and their combinations in the centrifuge model tests are described at prototype scale if not specified. The test conditions for CP1 and CP2 were the same. The relative density of the model ground for CP3 was different from that for CP1 and CP2.

Silica sand No. 8 was used for ground material. Figure 2 shows the grading curve and the physical properties of silica sand No. 8. At CP1 and CP2, the model ground was made by tamping the silica sand with the optimum water content in the container. The sand with the
Table 1. Test conditions of the pipe movement tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Pipe diameter (cm)</th>
<th>Relative density of ground (%)</th>
<th>Centrifuge acceleration (g***)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CP1</td>
<td>60 (2.0)</td>
<td>100 (100)</td>
<td>30</td>
</tr>
<tr>
<td>CP2</td>
<td>60 (2.0)</td>
<td>100 (100)</td>
<td>30</td>
</tr>
<tr>
<td>CP3</td>
<td>65 (1.3)</td>
<td>70 (70)</td>
<td>50</td>
</tr>
</tbody>
</table>

* Depth of the crown of pipes: 1.8 m (6.0 cm at CP1 and CP2; 3.6 cm at CP3).
** Here and hereafter, 'g' denotes gravity level.

Table 2. Scale ratios in centrifuge model tests

<table>
<thead>
<tr>
<th>Item</th>
<th>Scale ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geometrical scale</td>
<td>1/λ</td>
</tr>
<tr>
<td>Centrifuge acceleration</td>
<td>λ</td>
</tr>
<tr>
<td>Displacement</td>
<td>1/λ</td>
</tr>
<tr>
<td>Velocity</td>
<td>1</td>
</tr>
<tr>
<td>Acceleration</td>
<td>λ</td>
</tr>
<tr>
<td>Density</td>
<td>1</td>
</tr>
<tr>
<td>Force</td>
<td>1/λ^2</td>
</tr>
<tr>
<td>Strain</td>
<td>1</td>
</tr>
<tr>
<td>Stress</td>
<td>1</td>
</tr>
<tr>
<td>Time</td>
<td>1/λ</td>
</tr>
<tr>
<td>Frequency</td>
<td>λ</td>
</tr>
</tbody>
</table>

The diameter was 60 cm at CP1 and CP2, and 65 cm at CP3. The rigidity was much larger than pipes, however, the load was not large enough to cause a notable change in diameter of pipes. Therefore, the load on a rod could be regarded to be equal to the load on pipes of the same diameter. At the middle of the model pipe, a steel rod of the same diameter as the model pipe was welded such that the rod and the model pipe formed a capital letter 'T'. After the ground was filled to the surface level and consolidated in a centrifuge field, a load cell and the rod of the model pipe were connected by a flexible joint. This joint allowed for the rotation of the rod.

The model pipe was moved forcibly in the transverse horizontal direction at a rate of 1 mm/min. A rate of 1 mm/min was determined by considering the specification of the jack system.

Figure 3 shows the relationship between load per unit area and displacement obtained in three tests. The load per unit area was evaluated from \( \sigma = F/(DL) \), where \( \sigma \): the load per unit area, \( F \): the measured force, \( D \): the pipe diameter, and \( L \): the pipe length. The measured force included the friction force exerted on the steel rod welded to the model pipe. The estimated magnitude of the friction force was less than 5% of the total measured force. Therefore, the correction to exclude the friction force from the measured force was not done. CP1 and CP2, conducted under the same test conditions, produced almost the same results.

To focus on the maximum load, the horizontal force factor \( N_h \) obtained at CP1, CP2 and CP3 is plotted in Fig. 4, using a technique which was proposed by Trautmann et al. (1985b). \( N_h \) expresses the normalized maximum load per unit area and is defined as \( N_h = \sigma_{\text{max}} / (\gamma H) \), where \( \sigma_{\text{max}} \): the maximum load per unit area, \( \gamma \): the unit weight of the model ground, and \( H \): the depth of the pipe center. Based on the drained triaxial compression test, the internal friction angle of the model ground was determined to be 45 degrees at CP1 and CP2, and 40 degrees at CP3. Trautmann et al. moved forcibly the steel pipe of mainly 102 mm in outside diameter, 6.4 mm in wall thickness and 1.2 m in length in a model ground made of dry sand in a 1 g field. Load was transmitted to both ends of the pipe by steel rods. The deflection of the pipe was negligibly small (Trautmann et al., 1985a and 1985b). Although the test conditions and the boundary

Fig. 2. Grading curve and physical properties of silica and No. 8 optimum water content was used to achieve 100% relative density. The optimum water content was 16%. At CP3, the model ground was made by pouring dry silica sand in the container from above. To compare the test results, the same manner as that at later-mentioned quay wall model tests and sloping ground model tests was adopted to make the model ground at CP3. When the space was filled by the silica sand to the depth of installation of the pipe, the model steel pipe was placed at the level on the ground. Actually, the model pipe was a rigid steel rod.
conditions of the pipe were different, the maximum load per unit area obtained by authors and expressed in the form of the horizontal force factor are consistent with the research results by Trautmann et al. The model ground was made of wet sand with the optimum water content at CP1 and CP2, of dry sand at CP3 and the tests by Trautmann et al. The effects of the difference of the water content on the maximum load are not obvious in Fig. 4.

Figure 5 shows a cross section of the ground for CP1 after the test. From the discontinuities in the thin coloured horizontal sand layers, a well-defined passive soil wedge was observed in front of the pipe. This passive soil wedge is usually observed in tests involving forcibly moved pipes (Audibert et al., 1975 and 1977).

From these tests, it was confirmed that the centrifuge model tests gave a maximum transverse horizontal load on pipes which was consistent with previous research results.

---

### Table 3. Test conditions of the quay wall model tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Ground water level</th>
<th>Input acceleration</th>
</tr>
</thead>
<tbody>
<tr>
<td>CQ1</td>
<td>GL. ~ 4.6 m (GL. ~ 9.2 cm)</td>
<td>250 gal (12.8 g)</td>
</tr>
<tr>
<td>CQ2</td>
<td>GL. ~ 4.0 m (GL. ~ 8.0 cm)</td>
<td>300 gal (15.3 g)</td>
</tr>
<tr>
<td>CQ3</td>
<td>GL. ~ 3.0 m (GL. ~ 6.0 cm)</td>
<td>150 gal (7.65 g)</td>
</tr>
<tr>
<td>CQ4</td>
<td>GL. ~ 4.2 m (GL. ~ 8.4 cm)</td>
<td>140 gal (7.14 g)</td>
</tr>
</tbody>
</table>

In common:

* Centrifuge acceleration: 50 g.
** Pipe diameter and wall thickness: 65 cm (1.3 cm) and 2.5 cm (0.05 cm).
*** Depth of the crown of pipes: 1.8 m (3.6 cm).
**** Distance from box caissons to pipes: 7.5 m (15 cm).
***** Input waves: 2 Hz (100 Hz), thirty sine waves.
****** Relative density in the upper sand layer: 70% (70%).

---

### Quay Wall Model Tests

Test conditions

Four tests shown in Table 3 were carried out to investigate the transverse horizontal load exerted on pipes buried parallel to quay walls. Centrifuge acceleration was 50 g. The test manner to reproduce liquefaction-induced permanent ground displacement in areas behind quay walls was the same as that by Sato (1997). Figure 6 shows a vertical and a plan view of the model ground for CQ1. Box caissons were used to model quay walls. They were made of steel. The average density was 2.1 t/m³. It was the same as the standard density of caissons (Coastal Development Institute of Technology, 1993). The vertical and plan views for the other tests were, with the exception of ground water level, the same as the views provided in Fig. 6.

A model ground was made in a laminar container by pouring dry silica sand No.8 in from above. Silica sand No.8 was used because its coefficient of permeability was 2.93–3.14 × 10⁻³ cm/s. The coefficient of permeability in a centrifuge field is the same as that in a 1 g field and the coefficient of permeability of silica sand No. 8 is close to that of actual ground with high liquefaction potential.

After the ground was filled to the depth of the bottom of box caissons, the laminar container was placed in a vacuum tank and silicon oil was made to permeate into the ground from the bottom of the container. Next, the box caissons, divided into three sections in advance, were placed on a foundation made of fine gravel. Again, the ground was filled to surface level, and silicon oil was made to permeate into the ground in the vacuum tank. The ground water level in areas behind the quay walls was almost the same as the water level in front of the caissons at the test. The viscosity of silicon oil was 50 times that of water for satisfying the similitude law in a 50 g centrifuge field.

The ground was comprised of two layers. The lower was a loose sand layer whose relative density was about 40%. The upper was a medium-dense sand layer whose relative density was about 70%. The relative density was controlled by altering the pouring height of silica sand.
The boundary of two layers was at GL. -2.8 m–GL. -3.0 m.

The model steel pipe was buried parallel to the box caissons at a depth of 1.8 m from the crown of the pipe in the upper medium-dense sand layer. Four pairs of strain gages were put on the pipe’s inside wall facing each other. Both of the pipe ends were tied to the rigid piles using thin and long stainless steel plates. The boundary conditions of the pipe were the same as those of a simply supported beam.

As shown in Table 3, thirty sine waves of 140–300 gal and 2 Hz were input to cause liquefaction. Shaking was given in the direction of the long axis of the laminar container to reduce the effects of the rocking of the laminar container. Preliminary quay wall model tests had shown that cyclic components inevitably added to the model pipe response; however, preliminary results had confirmed that the magnitude of cyclic components had been much smaller than one-way components due to the liquefaction-induced permanent ground displacement.
This result was reconfirmed by the tests, as described later relating to Fig. 12 and Fig. 13.

Test results

Figures 7 and 8 show time histories of acceleration responses and excess pore water pressure at CQ1. The acceleration responses at A-G3, A-G4, A-G5 and A-G6 decreased after 3–7 sec. The excess pore water pressure at P-2 reached the effective overburden pressure at that depth at around 9–10 sec. These time histories indicate that liquefaction occurred in the ground area behind the caissons. It should be noted that excess pore water pressure at P-5 buried by the side of the pipe did not increase with time.

Figure 9 shows time histories of displacement of the box caisson and the ground surface just above the pipe at CQ1. The shaking ended after around 20 sec. The caissons and the ground surface moved after the end of shaking although the magnitude of the movement was small.

Figure 10 shows the ground displacement patterns in a cross section for CQ1. These displacement patterns were measured by tracing the movement of bead targets buried in the ground. As shown in Fig. 10, the horizontal displacement of the ground on the rear side of the pipe was restrained by the presence of the pipe.

Figure 11 shows a cross sectional sketch for CQ1. The ground just behind the caisson collapsed. The pipe was placed outside the collapsed area and was not directly
affected by the collapse.

The ground on the caisson side of the pipe and under the pipe moved forward with the movement of the lower loose sand layer. On the contrary, the movement of the ground on the rear side of the pipe was restrained. This discontinuous ground movement produced a crack on the caisson side of the pipe and the void was filled with the collapsed ground, as shown in Fig. 11. It should be noted that there were no slip surfaces on the rear side of the pipe and that passive soil wedges were not observed there. Slip surfaces going toward the lower loose sand layer were not found, either.

Figure 12 shows time histories of bending strain generated on the pipe at CQ1. Figure 13 shows time histories of bending strain at the point of 2-6 at CQ2, CQ3 and CQ4. Bending strain was estimated from the pairs of facing strain gages placed at four points along the pipe axis. As shaking was given in the direction of the long axis of the laminar container, elastic cyclic components were added to the response of the model pipe which was buried perpendicularly to the shaking direction. The magnitude of cyclic components, however, was much smaller than that of one-way components due to liquefaction-induced permanent ground displacement.

At CQ1 and CQ2, it was not obvious whether the bending strain achieved a peak value or not, although it seemed to approach toward a peak value after around 17 second in both tests. At CQ3 and CQ4, the bending strain could be regarded to achieve a peak value. The input acceleration at CQ1 and CQ2 was much larger than that at CQ3 and CQ4 as shown in Table 3. The difference in the magnitude of input acceleration may have caused the difference in the bending strain responses.

Cyclic components of the bending strain after the occurrence of liquefaction were excluded from the analysis of the load on pipes, since this paper focused on the effects of liquefaction-induced permanent ground displacement on pipes. By comparing Fig. 12 with Fig. 9, it is found that bending strain increased with an increase in ground displacement.

Solid marks in Fig. 14 show the bending moment at four points along the pipe axis at CQ1. The bending moment was estimated from measured bending strain. Solid lines in the figure represent a theoretical bending moment working on a simply supported beam subjected to a uni-
formly distributed load. The magnitude of the uniformly distributed load was determined by the method of least squares. From this figure, it was concluded that a uniformly distributed load was exerted along the pipe axis.

From the time histories of bending strains, time histories of the uniformly distributed load could be estimated. By combining these with the time histories of ground surface displacement just above the pipe, the relationship between the load per unit area and the ground surface displacement just above the pipe could be obtained, as shown in Fig. 15. This figure includes the determined relationship in all tests. There was no notable difference attributable to ground water level. The maximum load per unit area was 0.08–0.11 MPa in these tests. Even if the maximum load per unit area at CQ1 and CQ2, at which it was not obvious whether the bending strain achieved a peak value, is excluded, the upper and the lower bound of the maximum load per unit area will not change. The maximum load per unit area was 0.32 MPa at CP3 where the pipe was forcibly moved in the static ground with a relative density of 70%. The ratio of the maximum load per unit area in the quay wall model tests to that in the pipe movement test was 0.25–0.34.

The crack adjacent to the pipe on the caisson side of the pipe as shown in Fig. 11 was observed in all of the tests. There may have been some relation between the formation of such a crack and the test results that the transverse horizontal load achieved a peak value at a certain amount of ground displacement.

**Sloping Ground Model Tests**

**Test conditions**

Five tests shown in Table 4 were carried out to investigate the transverse horizontal load exerted on pipes buried in an inland sloping ground. Centrifuge accelerat-
tion was 30 g. The test manner to reproduce liquefaction-induced permanent ground displacement in sloping ground areas was the same as that by Dobry et al. (1995). Figure 16 shows a vertical and a plan view of the model ground for CS1. Pipes were buried perpendicularly to the direction of slope. The vertical and plan view for the other tests were, with the exception of certain dimensions, the same as the views provided in Fig. 16. In four tests out of five, the relative density in the upper sand layer was about 70%. That in the lower sand layer was about 40%. The boundary was set at the ground water level shown in Table 4. In one test, the relative density in both layers was about 40%. The viscosity of silicon oil was 30 times as much as that of water for satisfying the simili-
tude law in a 30 g centrifuge field. Except for these points, the model ground was made in the same manner as that used in the quay wall model tests.

The laminar container was set at an angle of five degrees or three degrees on the shaking table. The model ground was shaken in an inclined state. The ground water level was horizontal and not parallel to the ground surface. The effect of the horizontal ground water level on the transverse horizontal load on pipes may have been negligible because the pipe was buried above the ground water level in the middle of the model ground.

As shown in Table 4, thirty sine waves of 150 gal and 2 Hz were input to cause the liquefaction. Shaking was given in the direction of the long axis of the container. Test results

Figures 17 and 18 show time histories of acceleration responses and excess pore water pressure at CS1. The acceleration response at A-G5 decreased after around 10 sec; however, others did not. The excess pore water pressure at P-2 and P-1 was about 0.8 times as much as the effective overburden pressure at that depth at the maximum. These facts did not necessarily mean that liquefaction did not occur. The effective stress could not decrease to zero due to the initial shear stress in the sloping ground. The excess pore water pressure at P-2 and P-1 increased rapidly during the initial 3–4 seconds and then remained constant after that. The ground was considered to be a liquefied state. It should be noted that excess pore water pressure at P-4 buried by the side of the pipe did not increase.

Figure 19 shows time histories of horizontal and vertical ground surface displacement at CS1. D-GH2 and D-GV, respectively) and horizontal ground surface displacement about 6 m away from the pipe (D-GH1) at CS1. D-GH1 and D-GH2 exhibited almost
the same time histories. It seems that uniform ground surface displacement occurred in the horizontal direction in the upper sand layer.

Figure 20 shows ground displacement patterns in a cross section for CS1. These patterns were measured by tracing the movement of bead targets buried in the ground. At GL. -7 - 9 m, a notable shear deformation was observed. The majority of ground displacement was produced at this area. On the downstream side of the pipe, the ground displacement was almost uniform from the ground surface to around GL. -7 m. On the rear side of the pipe, the ground displacement was restrained around the pipe due to the presence of the pipe.

Figure 21 shows a cross sectional sketch for CS1. A crack was produced on the downstream side of the pipe and the void was filled with the collapsed ground. It should be noted again that there were no slip surfaces on the rear side of the pipe and that passive soil wedges were not observed there. Slip surfaces going toward the lower loose sand layer were not found, either. The ground displacement patterns around the pipe were essentially the same as those displayed in the quay wall model tests.

Figure 22 shows time histories of bending strain generated on the pipe at CS1. Bending strain could be regarded to achieve peak values at CS1. It could be regarded to achieve peak ones at the other tests, too. Cyclic components were excluded in the analysis of the load on pipes.

Figure 23 shows the relationship between the load per unit area on pipes and the ground surface displacement on the downstream side of the pipe (D-GH1) in the tests of ground water level of GL. -3 m. The load on pipes was estimated in the same manner as in the quay wall model tests. When the relative density in the upper sand layer was the same, the load-displacement relationship was almost the same, even if the sloping angle was different. It should be noted that the maximum load at CS3, where a relative density in the upper sand layer was 40%, was nearly equal to the maximum load at other tests where a relative density in the upper sand layer was 70%. This point will be discussed in a later chapter. The ground
displacement at CS3 in Fig. 23 may not represent the ground displacement as a whole due to local differences of the displacement.

Figure 24 shows the relationship between the load per unit area on pipes and the ground surface displacement on the downstream side of the pipe (D-GH1) in the tests of the different ground water levels. The maximum load measured at CS5 was 20% less than that at CS4.

The maximum load per unit area in these tests was 0.10–0.14 MPa. The ratio of the maximum load per unit area in the sloping ground tests to that in the pipe movement test of the same relative density was 0.31–0.44.

The crack adjacent to the pipe on the downstream side of the pipe as shown in Fig. 21 was observed in all of the tests. In the sloping ground model tests, as well as the quay wall model tests, there may have been some relation between the formation of such a crack and the test results that the transverse horizontal load achieved a peak value at a certain amount of ground displacement.

MAXIMUM LOAD DUE TO LIQUEFACTION-INDUCED GROUND DISPLACEMENT

The vertical axis in Fig. 25 shows the ratio of the maximum load exerted on pipes due to liquefaction-induced permanent ground displacement to that due to forcible pipe movement in static ground. The maximum load obtained in the quay wall model tests and the sloping ground model tests was divided by the maximum load in the CP3 pipe movement test in which the relative density of the ground was 70%. The horizontal axis in Fig. 25 represents the distance from pipe centers to the top of liquefaction layers, normalized by pipe diameter. The top of liquefaction layers was estimated by examining ground displacement patterns. This was defined to be the depth at which the curvature of the ground displacement took a maximum value. An example of the maximum curvature point of ground displacement is shown in Fig. 20.

The maximum load obtained in the quay wall model tests and the sloping ground model tests can be compared directly with that obtained in the pipe movement tests because all three tests are regarded approximately as a kind of element tests in two dimensions. The pipe movement tests were a kind of element tests in two dimensions. In the quay wall model tests and the sloping ground model tests, the pipe was simply supported at both ends. Strictly speaking, the tests were not performed in two dimensions. However, the load exerted on the pipe was uniformly distributed along the pipe axis, and the maximum deflection of the pipe was 3 cm–4 cm and negligible when it was compared with the pipe length. The maximum deflection of the pipe was much smaller than the ground displacement at which the maximum load was taken. Accordingly, a kind of element test was performed approximately in two dimensions in the quay wall model tests and the sloping ground model tests.

Figure 25 indicates that the maximum load exerted on pipes due to liquefaction-induced permanent ground dis-
The shaking ended after around 20 sec. Figures 9 and 19 show that a certain amount of ground displacement occurred after the end of shaking. Figures 12 and 22 show that the bending strain on pipes was continuous before and after the end of shaking. This means that the load on pipes did not increase notably due to the ground displacement after the end of shaking. Based on this observation, shaking effects were not the cause of the decrease in the maximum load.

A Rate of Ground Displacement

In the pipe movement tests, a model pipe was moved forcibly at a rate of 1 mm/min. In the quay wall model tests and the sloping ground model tests, a rate of ground displacement was much larger than 1 mm/min.

Zhang et al. (1998) reported that the seismic earth pressure exerted on retaining walls could be determined by equations using soil strength parameters obtained by static soil element tests. They did not include factors to take into account the effect of the rate of ground displacement. It can be considered that the transverse horizontal load on pipes buried in sand has similar characteristics to the seismic earth pressure on retaining walls regarding the effect of a rate of ground displacement. Accordingly, the effect of the rate of ground displacement on the transverse horizontal load on pipes buried in sand may be negligible.

Residual Strength of the Ground Material

In the case of the pipe movement tests, the maximum load at CP1 and CP2 in which an initial relative density of the model ground was 100%, was larger than the maximum load at CP3 in which an initial relative density was 70%, as shown in Fig. 3. In the pipe movement tests, the strength characteristics of the ground material at an initial condition affected the maximum load.

In the case of the sloping ground model tests, the maximum load at CS3, in which an initial relative density in the upper sand layer was 40%, was nearly equal to the maximum load at other tests in which an initial relative density in the upper sand layer was 70%, as shown in Fig. 23. This means that the strength characteristics of the ground material around the pipe at CS3 were almost the same as those at other tests when the maximum load was achieved. Since the residual strength of the ground material does not differ so much even if the initial relative density does (Zhang et al., 1997), there is a possibility that a maximum load corresponding to the residual strength of the ground material was exerted on the pipes.

Residual strength is exhibited in a large shear strain area. The ground displacement on the rear side area of the pipe was restrained because of the presence of the pipe, as shown in Fig. 10, and more clearly in Fig. 20. Due to the difference of ground displacement between the ground surface area and the rear side area of the pipe, and between the rear side area of the pipe and the liquefaction layer surface area, large shear strain might have been generated on the rear side area of the pipe.
**Fig. 27.** Horizontal displacement at ground surface around the pipe at CQ1 in the quay wall model tests and at CS1 in the sloping ground model tests.

**Confining Pressure**

On the caisson side of the pipe in the quay wall model tests and on the downstream side of the pipe in the sloping ground tests, cracks were formed due to the presence of the pipe, as shown in Figs. 11 and 21. These cracks may have decreased the confining pressure of the ground around the pipe. The decrease in confining pressure may have caused the decrease in the maximum load exerted on the pipe.

Figure 27 shows the horizontal displacement at ground surface around the pipe at CQ1 in the quay wall model tests and at CS1 in the sloping ground model tests. At CQ1 in the quay wall model tests, the ground displacement increased approaching the caissons. Therefore, the pipe was buried in a tensile ground strain area. On the other hand, the ground displacement at CS1 in the sloping ground model tests was almost constant and there was no notable tensile strain around the pipe. Due to the presence of the tensile strain which decreased the confining pressure around the pipe, the maximum load obtained in the quay wall model tests may have been smaller than the maximum load obtained in the sloping ground model tests, as shown in Fig. 25.

**Rigidity of Pipes**

The maximum change in the pipe diameter was estimated to be less than 0.5 cm or less than 1% of the pipe diameter in both of the quay wall model tests and the sloping ground model tests. This magnitude of the change in diameter may not have affected the maximum load on the pipe.

One of the indexes to express the rigidity of pipes is \( \Delta D/D \), where \( \Delta D \) is the change in pipe diameter and \( D \) is the pipe diameter. \( \Delta D/D \) is proportional to \( (D/t)^3 \), where \( t \) is the wall thickness of the pipe, when pipes are subjected to an uniformly distributed load (Japan Society of Civil Engineers, 1986). In the case of gas transmission pipelines, the smaller the pipe diameter is, the smaller \( D/t \) is. Accordingly, the smaller the pipe diameter is, the smaller \( \Delta D/D \) is. Therefore, it can be said that the test results are applicable to gas transmission pipelines whose diameter is 65 cm or less, as the rigidity of pipes is concerned. 65 cm was the largest diameter of the pipes used in the tests.

**CONCLUSIONS**

The transverse horizontal load exerted on pipes due to liquefaction-induced permanent ground displacement in the horizontal direction was investigated using dynamic centrifuge model. Pipes were buried in non-liquefaction layers above liquefaction layers. The emphasis was put on the maximum load. The main conclusions are as follows.

1. The maximum transverse horizontal load obtained by moving pipes forcibly in the horizontal direction in a static ground in a centrifuge acceleration field showed good agreement with previous research results in a 1 g field. It was confirmed that the centrifuge model tests gave a maximum transverse horizontal load consistent with previous research results.

2. All of the load due to liquefaction-induced permanent ground displacement obtained in the dynamic centrifuge quay wall model tests, and the dynamic centrifuge sloping ground model tests were much smaller than the load obtained by moving pipes forcibly in static ground in a centrifuge field. The ratio of the maximum load due to liquefaction-induced permanent ground displacement to that obtained by moving pipes forcibly was 0.3–0.4.

3. Excess pore water pressure did not increase at pipe depth. Accordingly, soil rigidity did not decrease due to the decrease in effective stress. Slip surfaces going toward liquefaction layers and forming a passive wedge of the ground between the pipe and the liquefaction layer were not observed. These factors did not cause the decrease in the maximum load. The load-ground displacement curves were continuous before and after the end of shaking in the tests. Therefore, shaking effects were not the cause of the decrease in the maximum load, either. The effect of the rate of ground displacement may be negligible.

4. The maximum load in the test in which an initial relative density in the upper sand layer was 40%, was almost equal to the maximum load in other tests in which an initial relative density in the upper sand layer was 70%. There is a possibility that a maximum load corresponding to the residual strength of the ground material was exerted on the pipe. Based on observations of the ground displacement patterns in a cross section of the model ground after the test, it was suggested that large shear strain was generated on the rear side area of the pipe.

5. On the caisson side of the pipe in the quay wall...
model tests and on the downstream side of the pipe in the sloping ground model tests, cracks were observed after the test. These cracks may have decreased the confining pressure of the ground around the pipe and resulted in a decrease in the load on the pipe. The tensile strain generated in the ground may have decreased the confining pressure and resulted in the decrease in the load on the pipe.

(6) The change in pipe diameter may not have affected the maximum load on the pipe. It can be said that the test results are applicable to gas transmission pipelines whose diameter is 65 cm or less, as far as the rigidity of pipes is concerned.

ACKNOWLEDGEMENTS

After the 1995 Hyogoken-nambu (Kobe) Earthquake, Agency of Natural Resources and Energy, Ministry of Economy, Trade and Industry (Ministry of International Trade and Industry at that time) commissioned the Japan Gas Association to investigate gas pipeline behavior in the event of liquefaction. This investigation started in 1996 and took five years to complete. It was supervised by the committee for the investigation of the effects of liquefaction on gas pipelines, chaired by Dr. Tsuneo Katayama, Director-General, National Research Institute for Earth Science and Disaster Prevention. The authors wish to express their gratitude to all those at METI for their permission to publish this paper. They also wish to express their gratitude to committee members for their invaluable suggestions.

REFERENCES

1) American Society of Civil Engineers (1984): Guidelines for the seismic design of oil and gas transmission systems, 163–169.