Three-dimensional dynamic behaviour of embankments on liquefiable ground

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There has been serious damage to embankments on liquefied ground due to large earthquakes. To understand such damage process, two-dimensional shaking table model tests have been usually performed, in both gravitational and centrifugal fields, to investigate the dynamic behaviour and residual displacement of embankments on liquefiable ground. In recent years, three-dimensional (3D) numerical analysis has been used in practical design to consider dynamic behaviours of 3D embankments on liquefiable ground. However, there are only a limited number of cases in which the applicability of 3D analysis has been validated based on comparisons with data measured at model tests or actual disasters. Therefore, in this study, a series of shaking table tests were conducted to investigate the seismic behaviour of a 3D embankment on liquefiable ground. In addition, the effect of the shaking direction on the seismic behaviour of the embankment was evaluated. The experiment revealed that the residual deformation and its dominant direction were significantly affected by the 3D shape and total weight of the embankment, not by the shaking direction. This result indicates that the influence of the 3D shape of the embankment on the deformation behaviour cannot be ignored, and that the influence should be properly evaluated in seismic design.

KEYWORDS: embankments; liquefaction; model tests

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INTRODUCTION

Liquefaction of sandy soil during earthquakes causes settlements in embankments and river dikes, which are constructed on loose sandy deposits – for example, the 1983 Nihonkai-Chubu earthquake caused deformations in road embankments, and several cracks in the pavement were observed in the longitudinal direction of the embankment on the liquefied ground (PWRI, 1985; Towhata, 2008). A similar failure of river dikes caused by liquefaction was also observed during the 1993 Kushiro-oki earthquake, the 1995 Kobe earthquake (Towhata, 2008) and the 2011 off the Pacific Coast of Tohoku earthquake (Koseki *et al.*, 2012; Sasaki *et al.*, 2012; Yamaguchi *et al.*, 2012). Based on such actual cases of damage, several two-dimensional (2D) shaking table model tests have been performed in both gravitational and centrifugal fields to investigate the dynamic behaviour of embankments and river dikes on liquefiable ground (Koga & Matsuo, 1990; Sasaki *et al.*, 1992; Adalier *et al.*, 1998; Okamura & Tamura, 2004; Sharp & Adalier, 2006; Maharjan & Takahashi, 2014).

Embankments for thermal power plants, which are the subject of this study, are usually constructed near the sea in Japan. Important facilities such as gas pipes for transporting power-generating fuel in thermal power plants are often located across three-dimensional (3D) embankments (Fig. 1(a)). In the seismic design of such important infrastructures, it is necessary to predict how the embankment behaves three-dimensionally during an earthquake,

particularly when the embankment is constructed on liquefiable ground. In such a case, the initial shear stress in the supporting ground is larger than that under the 2D embankments such as river dikes, so the liquefaction-induced deformation is expected to become larger. In addition, the direction of seismic motion may affect major direction of embankment deformation. However, none of the above effects have been fully clarified.

In recent years, 3D numerical analysis has been used in practical design because it is difficult to reproduce the 3D deformation of liquefied ground by using a conventional 2D analysis. However, there are only a limited number of cases in which the applicability of 3D analysis has been validated based on comparisons with true values obtained from model tests. This is because most of the model tests in the literature studies were conducted under plane strain conditions, focusing on the behaviour of 2D embankments on liquefied ground.

Therefore, in this study, a series of shaking table tests were conducted to investigate the seismic behaviour of a 3D embankment on liquefiable ground. As shown in Fig. 1(b), the embankment model was constructed as a 2D embankment (case 3) with a uniform cross-section in the longitudinal direction and as an L-shaped 3D embankment (case 4). To investigate the influence of the shaking direction, a shaking table test was conducted by changing the shaking direction by 90° to the aforementioned 2D embankment (case 5).

TESTING PROCEDURE

Model of embankment and liquefiable ground

The data used in this study were obtained from experiments conducted at the University of Tokyo by using a shaking table. A rigid soil container (2.2 m long, 1.97 m wide and 0.80 m high) was placed on the table with a side glass plate for visual observation. A 2D and 3D embankment model was constructed in a soil container (Fig. 2).

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Fig. 1. Typical example of pipeline crossing 3D embankment and modelling in shaking table tests: (a) typical example of pipeline crossing 3D embankment, (b) simple modelling in shaking table tests

Fig. 2. Overview of the liquefiable ground and embankment model

The model consisted of liquefiable ground (height: 200 mm) and embankment (height: 200 mm), and these were made of silica sand number 7 (Table 1). Due to the large size of the soil container, the air-pluviation method using a sand hopper, which is commonly used to make homogeneous loose-ground models such as liquefiable ground, could not be applied. For this reason, the ground model was prepared by manually pouring dry silica sand as evenly as possible while controlling the thickness of each layer to 50 mm and then compacting them very softly using a wooden trowel. Layer thickness was controlled at 49 locations on the ground surface (2000 × 1750 mm) using a laser horizontal marker, to an accuracy of less than ± 0.5 mm. To simulate liquefiable ground with loose conditions as much as possible, a temporary scaffold was set-up during ground preparation

Table 1. Physical properties of silica sand number 7 (JIS A1204:2020)

Properties	Silica sand number 7
Specific gravity	2.640
Minimum void ratio	0.671
Maximum void ratio	1.175
Maximum particle diameter, $D_{\max}$: mm	0.425
Mean particle diameter, D_{50} : mm	0.202
Effective particle diameter, D_{10} : mm	0.120
Coefficient of uniformity, U_c	1.940
Coefficient of curvature, U_c'	0.980

so that the experimenter did not have to ride directly on the model ground during preparation. A relative density of 52% was achieved using this method. The embankment model was created by employing a tamping method using wet silica sand (3% water content).

After the completion of the model ground and embankment, carbon dioxide and degassed water were supplied sequentially from several water pipes placed under the soil container. A water head difference of 20 cm between the water height in the ground and the water supply tank was maintained during saturation. After the water level reached the ground surface, the supply of the degassed water from the bottom of the soil container was continued while the water was drained from the ground surface to the outside of the soil container. This process was continued for approximately 24 h to increase the saturation level of the liquefiable ground as much as possible.

To measure the dynamic response and excess pore water pressure (EPWP in kPa) of the model, several accelerometers and pore water pressure gauges were installed in the ground. The base acceleration was measured by using an accelerometer placed directly on the shaking table. The displacement of the embankment at the toe and shoulder was measured by using laser displacement transducers in both the horizontal and vertical directions. The locations of the representative pore water pressure gauges and laser displacement transducers are shown in Fig. 3. In addition, black spherical targets (10 mm wide) were prepared on the surface of the ground and embankment, to visually measure the displacement of the model after shaking. A total of 49 targets were installed at a distance of 250 mm.

Fig. 3. Location of representative sensors and black targets on ground surface (unit: mm)

Fig. 4. Typical time history of base acceleration (Case 3)

Seismic load application

The seismic load applied to the shaking table was a four-stage sinusoidal gradient wave (5 Hz, Fig. 4). The initial base acceleration was set at 100 Gal (Gal is a unit of acceleration equivalent to 0.01 m/s^2) (1 s, five waves) and increased to 200 Gal (1 s, five waves), 400 Gal (2 s, ten waves) and 800 Gal (2 s, ten waves).

TEST RESULTS AND DISCUSSION

Residual deformation of the embankment model

Figure 5 shows the side-view images of case 4 taken before and after the shaking. The ground under the embankment lost its bearing capacity, and then the embankment settled, accompanied by lateral spread at the bottom of the embankment.

Figure 6 shows the deformation of the embankment observed from above at an angle and movement of the

Fig. 5. Front view of model: (a) before shaking and (b) after shaking (case 4)

target-point positions before and after the shaking. The 2D displacements of representative targets, such as B4, D2 and D4 (Fig. 6), are listed in Table 2. The deformation of the 2D embankments, namely cases 3 and 5, was dominant in the direction of the embankment slope, regardless of the shaking direction. For example, the horizontal displacement of case 3 at the bottom toe of the embankment (D4) was 69 mm, which is at the same level as that of case 5 (D4, 65 mm). However, the horizontal displacement at the same position (D4) for the 3D embankment (case 4) was observed in both

Fig. 6. Deformation of embankment model after shaking

Table 2. Movement of black targets on the shoulder and bottom toe of embankment model (unit: mm)

Target name	Case 3		Case 4			Case 5	
	B4 (shoulder)	D4 (bottom toe)	B4 (shoulder)	D2 (shoulder)	D4 (bottom toe)	D2 (shoulder)	D4 (bottom toe)
X-direction (shaking direction)	82	69	51	28	55	7	6
Y-direction (perpendicular to shaking direction)	2	-3	25	60	82	52	65
Total displacement	82	69	57	67	99	53	65

the shaking direction (55 mm) and the perpendicular direction (82 mm), where the total displacement (99 mm) was much larger than that of the 2D embankments (cases 3 and 5). This indicates that the dominant horizontal direction and level of deformation at the bottom toe of the embankment changed depending on the shape of the embankment, but the influence of the shaking direction was not significant.

In contrast, the horizontal displacement of the embankment shoulder of case 3 (B4, 82 mm) was larger than that of case 5 (D2, 53 mm). This is because the horizontal displacement of the shoulder was affected by the deformation of both the liquefied ground and the embankment body, which increased the deformation in the shaking direction. In the case of the 3D embankment, the total displacement at the embankment shoulders of B4 and D2 were 57 and 67 mm, respectively, which were intermediate values between case 3 (B4) and case 5 (D2).

Displacement of embankment during shaking

Figure 7 shows the time history of the base acceleration, EPWP ratio and displacement of the embankment measured at several points. Vertical displacements, such as the settlement at the embankment shoulder and uplift at the bottom toe of the embankment, were observed in all cases, and these displacements were increasing, clearly from the

second shaking step (200 Gal), when the EPWP increased considerably. The EPWP attained the initial effective stress at the final shaking step (400 Gal), indicating perfect liquefaction. The settlement at the embankment shoulder occurred rapidly after the second shaking step for case 4, whereas it occurred slowly in a similar manner for cases 3 and 5 (Fig. 7(a)). A similar trend was observed for the uplift behaviour of the bottom toe of the embankment (Fig. 7(b)). Rapid and large deformation of case 4 may be caused by the large initial static shear stress acting on both the shaking direction and shaking orthogonal direction in the supporting ground, which induces larger cyclic strain development for loose sand (Vaid & Chern, 1983). In other words, the deformation of liquefied ground will be at the same level regardless of the shaking direction if the initial shear stress is the same (Cases 3 and 5).

Dynamic response of embankment during shaking

The dynamic fluctuation of displacement during shaking was clearly different in the shaking direction. The horizontal displacement at the bottom toe of the embankment in the shaking direction of cases 3 and 4 (Fig. 7(c)) exhibited large dynamic fluctuations with increasing and decreasing acceleration. However, almost no dynamic fluctuation was observed in the horizontal displacement perpendicular to the shaking direction for case 5, and it occurred at an almost

Fig. 7. Time history of displacement of embankment, EPWP ratio and acceleration: (a) settlement at embankment shoulder (LS01), (b) uplift at bottom toe of embankment (LS04), (c) horizontal displacement at bottom toe of embankment (LS03), (d) EPWP ratio (PWP3, measured at 175 mm depth), (e) base acceleration

constant velocity during each shaking step. A similar trend was also observed in the settlement (Fig. 7(a)).

It should be noted that the displacement velocity shown in Figs 7(a) and 7(c) clearly increased with the increase in acceleration, such as the third and fourth shaking steps of case 5. This indicates that the shear resistance of the liquefied ground changes with both the intensity of shaking and liquefaction level. A similar trend was observed in another study (Watanabe *et al.*, 2016) in the uplift behaviour of underground structures in liquefied ground. This behaviour is related to the viscous resistance of liquefied soil during earthquakes, and further studies are needed to understand this behaviour.

CONCLUSIONS

In this study, shaking table tests on 2D and 3D embankment models on liquefiable ground were conducted to verify the 3D behaviour of the embankment and the effect of the shaking direction. The following conclusions were drawn.

- The dominant direction of deformation was significantly affected by the shape of the embankment, and the influence of the shaking direction was limited.
- The vertical displacement, which is a very important index for evaluating the seismic performance of embankments, was influenced by the large initial static

shear stress in both the shaking direction and shaking orthogonal direction in the supporting ground and was less influenced by the shaking direction.

- The vertical displacement occurred at an almost constant velocity during each shaking step, but the velocity clearly increased in the next step with a stronger intensity of shaking. A similar trend was observed in past experiments performed by the author, which focused on the uplift behaviour of underground structures in liquefied ground. These behaviours are related to the viscous resistance of liquefied soil during earthquakes, and further studies are needed to understand this behaviour.

The experimental results indicate that the 3D displacement of a 3D shape embankment, which cannot be evaluated by a general 2D numerical analysis, is too large to be ignored in practical design. This is because the horizontal displacement in the cross-section of the 2D analysis can be very large. This important finding in this study should be properly evaluated in seismic design.

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