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# Seismic behaviour of reinforced embankments in dynamic centrifuge model tests

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**Abstract**

A series of dynamic centrifuge model tests was conducted to investigate the effects of reinforcement on the seismic behaviour of hillside embankments consisting of sandy soils and resting on stiff base slopes. In total, three types of seismic reinforcements, namely, large-scale gabions, drainage-reinforcing piles, and ground anchors with pressure plates, were employed in the tests. The test results showed that: (1) the seismic performance of both lower and higher embankments was remarkably improved by installing large-scale gabions at the toe as they restrained the completion of the formation of sliding planes; (2) the installation of drainage-reinforcing piles at the embankment toe was rather effective in reducing the overall earthquake-induced deformation due to their high permeability and restraint effect against sliding displacement at the reinforced region; and (3) the embankments improved by ground anchors with pressure plates were not vulnerable to earthquake-induced damage due to their constraint effects even under high water table conditions. The improvement effects by the above-mentioned three types of reinforcements were presented by evaluating the global safety factors based on the results of a series of triaxial compression tests.



*Keywords:* Dynamic centrifuge model test; Embankment; Seismic reinforcement; Laboratory triaxial test; Global safety factor (**IGC:** D6\D7)

## 1. Introduction

Road embankments constructed on mountainsides and hillsides have frequently experienced catastrophic failures due to past strong earthquakes. In some cases, a long period of time was required to restore the damaged hillside embankments, particularly in the case of the higher ones. For example, according to Hashimoto et al. (2008), it took 44 days to restore embankments for the national highway damaged by the 1994 Hokkaido-Touhouoki earthquake. Tokida (2012) mentioned that the Noto Toll Road, damaged by the 2007 Noto-Hanto earthquake, was opened to the public about one month after the main shock and that the permanent restoration of all the damaged embankments was completed about five months after the temporary recovery.

To reduce earthquake-induced damage to embankments, some studies using dynamic centrifuge apparatuses have been conducted (e.g., Kutter and James, 1989; Dobry et al., 1997; Pilgrim, 1998; Okamura et al., 2001; Matsuo et al., 2002; Egawa et al., 2004; Okamura and Tamamura, 2011; Higo et al., 2015). However, these studies focused on the seismic behaviour of embankments with no countermeasures, and the number of investigations on the effectiveness of reinforcement has been limited. As pointed out by Tokida (2012), one of the reasons for this may be that damage to road embankments induced by

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earthquakes is still considered easy to repair, in spite of the above-mentioned examples of having to spend a long period to restore them.

Among a limited number of recent studies, the following experimental investigations were conducted on seismic countermeasures for road embankments using dynamic centrifuge apparatuses:

(1) The seismic behaviour of embankments with countermeasures for liquefiable subsoil was investigated by Adalier et al. (1998).
(2) Hashimoto et al. (2008) examined the effects of reinforcing bars with and without small pressure plates on the seismic performance of embankments.
(3) The seismic performance of embankments crest-retrofitted with geotextile was confirmed by Ueno et al. (2009).
(4) Tokida (2012) reported the effectiveness of crest reinforcement with geosynthetics, the placement of toe rigid blocks, and the installation of rigid barrier walls at the embankment shoulder.

However, these experimental studies used model embankments with a height of 10 m or less in prototype scale due to the limited capacities of the shaking table and the soil container, in spite of the susceptibility of embankments more than 15 m in height to earthquake-induced damage, which was reported by Okimura et al. (1999) and PWRI (2008). The excitation capacities of the shaking tables were also limited (i.e., a peak input acceleration smaller than 0.5 g). In addition, the seismic behaviour of embankments with other types of countermeasures, excluding the above-mentioned retrofit techniques, has not been examined.

In view of the above, a series of dynamic centrifuge model tests on the seismic performance of reinforced hillside road embankments, 15 m or 30 m in height, was conducted in the present study by improving the excitation capacity of the shaking table and using a large-scale soil container. This paper focuses on reinforcements by large-scale gabions, drainage-reinforcing piles, and ground anchors with pressure plates, where they can often be employed in geotechnical practice. Although the concept of employing large-scale gabions itself was the same as that in the tests conducted by Tokida (2012), the gabions in the present study were stacked in three layers without fixing them in order to reproduce more realistic conditions. Furthermore, to quantify the effectiveness of the above-mentioned three seismic countermeasures, the global safety factors of the reinforced embankments were also evaluated based on the results of a series of triaxial compression tests on fill materials.

## 2. Tested materials

Fig. 1 shows the grading curves and the laboratory compaction test data for Edosaki sands from four different batches used for model and laboratory element tests. Edosaki A and B sands were used for centrifuge test Cases 1, 8, and 9 conducted by Enomoto and Sasaki (2015), where their model embankments had no seismic countermeasures. Edosaki C sand was used in the present study for Cases 13 through 20 with the installation of the above-mentioned reinforcements. Details of the centrifuge model tests are presented later in this paper. Edosaki B, C, and D sands were also used in the present study for laboratory stress-strain tests. The values of the specific gravity ($G_s$), maximum diameter ($D_{max}$), mean diameter ($D_{50}$), uniformity coefficient ($U_c$), fines content ($F_c$), optimum water content ($w_{opt}$), maximum dry density ($\rho_{dmax}$), plasticity index ($I_p$), maximum and minimum void ratios ($e_{max}$ and $e_{min}$), and permeability coefficient ($k$) of these materials are summarized in Table 1. The values of $w_{opt}$ and $\rho_{dmax}$ were determined by the A-c method in Japanese Industrial Standards (JIS) A 1210, where the materials were compacted in a cylindrical mould (inner diameter = 100 mm) in three layers with 25 free drops of a 2.5-kg rammer from a height of 30 cm for each layer.

The soil properties of Edosaki A through D sands from different batches were quite similar to each other. The initial degree of compaction, $D_{c0}$, defined by Eq. (1), was used in the present study as the density index.

$$D_{c0} = \rho_{d0} / \rho_{d\,\max} \tag{1}$$

where $\rho_{d0}$ is the initial dry density of the soil.

The $k$ value of Edosaki A sand, shown in Table 1, was evaluated by the constant head permeability test (JIS A 1218), where the specimen was prepared at the same water content and density as in the model and laboratory stress-strain tests (i.e., $w_{opt} = 16.7\%$ and $D_{c0} = 82\%$, respectively).

## 3. Laboratory element tests on employed soils

### 3.1. Test procedures

The undrained shear behaviour of saturated Edosaki sand under triaxial conditions was reported by Enomoto and Sasaki (2015). In the present study, a series of drained triaxial compression tests was conducted on this saturated sand, ET1 through ET6, summarized in Table 2. The parameters obtained in these tests are used later in this paper to evaluate the global safety factors of the model embankments.

Small-scale cylindrical specimens, 5 cm in diameter and 10 cm in height, were produced by compacting wet materials inside a split mould in five layers using a hammer to $D_{c0} = 82\%$ under the respective $w_{opt}$ values. The specimen was set into the triaxial apparatus and was saturated under an isotropic state with an effective mean principal stress of $p' = (\sigma'_v + 2\sigma'_h)/3 = 20$ kPa, where $\sigma'_v$ and $\sigma'_h$ are the effective vertical and horizontal principal stresses, respectively. After isotropic compression was performed toward the target initial effective confining stress, $\sigma'_0$, shown in Table 2, strain-controlled vertical monotonic load was applied at a

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