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Shaking table tests on a new type bridge abutment with geogrid-reinforced cement treated backfill

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ABSTRACT: To develop new aseismic types of bridge abutment, a series of model shaking table tests were conducted using sinusoidal and irregular input motions on five models of three conventional and two new types of bridge abutment. The test results revealed that the seismic stability of the new type abutments with backfill of geogrid-reinforced cement-treated gravel with the reinforcement layers connected to a full-height rigid facing supporting directly a bridge girder is much higher than the conventional type ones. The connection between the reinforcement and the facing structure was found essential to dynamically stabilize the facing structure that was less stable than the backfill.

1 INTRODUCTION

A great number of conventional type railway bridge abutments were seriously damaged with a large relative settlement between the bridge abutment and the backfill, as typically shown in Picture 1, during the 1995 Hyogoken-Nambu Earthquake. Such relative settlement as above could endanger safe train operation, even when it is small, say several centimeters. In view of the above, a long-term research project started 1997 jointly at Railway Technical Research Institute and University of Tokyo aiming at developing new aseismic types of bridge abutment. The following two new structural types have been proposed and studied as feasible ones:

1) the backfill consists of a zone of geogrid-reinforced cement-mixed gravel immediately behind a full-height rigid facing structure supporting a bridge girder; and
2) the backfill is geogrid-reinforced gravel supporting a bridge girder that is preloaded and prestressed by using tie rods. The top ends of the tie rods are fixed to the top reaction block placed on the crest of backfill by using a special connection device (Nakarai et al. 2002).

With both types, the ends of reinforcement layers are connected to the back of the facing that is constructed after the full-height backfill is completed.

The objective of the present study is therefore to evaluate the seismic stability of the former type of bridge abutment by performing a series of model shaking table tests, in particular to ensure whether the new type bridge abutment can behave satisfactorily even during very high-intensity seismic load (so-called Level 2 earthquake). The results from similar tests for the latter type are reported in Nakarai et al. (2002).

2 TESTING PROCEDURES

2.1 *Model of retaining wall and backfill*

The abutment models investigated are shown in Figure 1 (conventional type; models 1, 2 & 3) and Figure 2 (new type; models 4 & 5), while all the tests are listed in Table 1. The facing structure was made of aluminum to have a height of 620 mm with a footing base having a width of 390mm (models 1 – 3), 290 mm (model 4) or 200 mm (model 5). The facing structure supported a model bridge girder with a mass of 200 kg through a hinged support (so the lateral seismic load acting to the girder was transmitted to the facing structure). For Model 1, simulating the most conventional type of bridge abutment, the unreinforced backfill was made by pluviating through-air air-dried fine sand (Toyoura sand) from a sand hopper at a constant falling height to have a relative density D_r of 75 %. Models 2 and 3 simulate other conventional but relatively new types used to decrease seismic load-induced settlement of the backfill immediately behind the abutment.

Picture 1. The damage of bridge abutment at Hyogoken-Nanbu Earthquake (1995.1.17 M=7.2 Kobe)

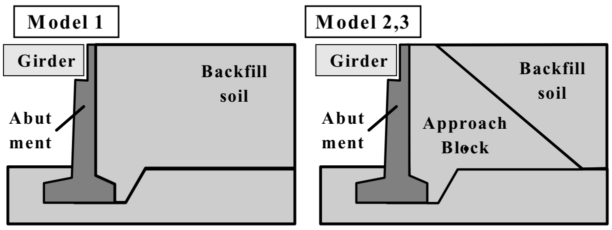

Figure 1. Typical conventional bridge abutment for railway

Table 1. The Type of abutment models

	Approach Block	Rein-forcment	Input motion	Width of footing
Model 1 (Conventional)	sand (Dr=75%)	No	Sinusoidal & Kobe wave	390mm
Model 2 (Conventional)	dry gravel (ρ_d=1.9g/cm^3)	No	Sinusoidal Wave	390mm
Model 3 (Conventional)	cement-mixed soil	No	Sinusoidal Wave	390mm
Model 4 (Proposed type)	cement-mixed soil	Yes	Kobe wave & Sinusoidal	290mm
Model 5 (Proposed type)	cement-mixed soil	Yes	Kobe wave & Sinusoidal	200mm

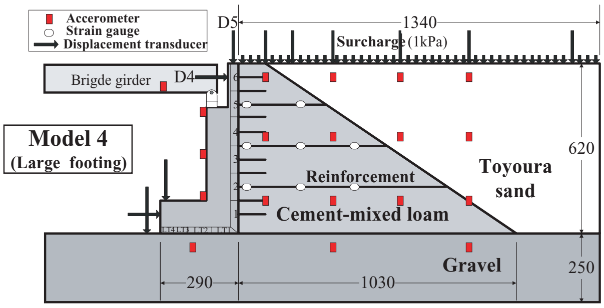

Figure 2. Cross-sections of reinforced abutment with cement treated backfill (Model 4)
(Only the width of base footing was different between Model 4 & 5)

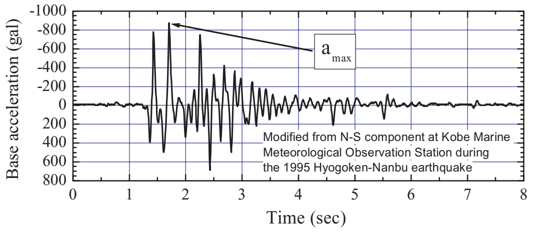

Figure 3. Typical time history of base acceleration for irregular shaking tests

Prototype bridge abutments of these types have a triangle-shaped backfill zone made of compacted well-graded gravel or cement-mixed gravel immediately behind the facing structure (called the approach block; Figure 1). The approach block of model 2 was made by compacting air-dried gravel (U_c=10.7; D_{50}= 1.1 mm, D_{max}=5.0 mm and a fines content = 5.2 %) to a dry density ρ_d= 1.9 g/cm^3. The model approach block of models 3, 4 & 5 was made of a cement-mixed loam having an unconfined compressive strength of 0.2N/mm^2, which was determined by considering the model similitude. Models 4 and 5 simulate the new structural types having a triangle-shaped approach block made of cement-mixed gravel that are reinforced with geogrid reinforcement layers connected to the back face of the facing directly supporting a bridge girder. It was originally considered necessary to connect the reinforcement layers and the facing to restrain the settlement of the approach block relatively to the facing structure. It is shown in this paper however that this measure is also essential to maintain a high integrity of the abutment structure. The footing of the facing was either relatively wide with model 4 or relatively narrow with model 5. The model reinforcement was a grid of 0.2 mm-thick and 3 mm-wide phosphor-bronze strips that were soldered to each other with an aperture of 50 mm and 100 mm in the transversal and axial directions. The tensile force in the model reinforcement was measured by using electric-resistant strain gauges attached to the central strip at three levels. Obviously, the stiffness of this model reinforcement, in particular at the connection with the back of facing, was too large when considering the model similitude. Another series of model tests to evaluate the stiffness of model reinforcement are now underway.

For all the models, the subsoil ground was made by compacting another type of air-dried gravel (U_c= 12.1; D_{50}= 10.0 mm, D_{max}= 32.0 mm and a fines content = 2.0 %) to ρ_d= 1.9 g/cm^3. The dynamic response of the abutment and backfill was measured with a number of displacement transducers and accelerometers (Figure 2). The dynamic earth pressure acting on the back face of the facing and the bottom of the base footing of the facing were evaluated with two-component load cells measuring normal and shear forces.

2.2 *Shaking table tests*

The table supporting the models was shaken horizontally by using uniform 50 sinusoidal waves lasting 10 seconds at a frequency of 5 Hz (models 1, 2 and 3) and irregular waves at a predominant frequency of 5 Hz (models 1, 4 and 5). The amplitude of acceleration a_{max} of the sinusoidal waves was increased step by step from the initial value of 50 gals with an increment of 50 gals until the displacements of the abutment became considerably large. As the irregular waves, a time history of acceleration with an adjusted predominant frequency of 5 Hz made from the time history of horizontal acceleration recorded on the ground at the Kobe Marine Meteorological Observation Station during the 1995 Hyogo-ken Nambu Earthquake was used (Figure 3). The maximum amplitude a_{max} was increased step by step with an increment of 100 gals from 100 gals to 1,400 gals. In the tests on models 4 and 5 using the irregular waves, as the abutment did not reach ultimate failure even when a_{max} became 1,400 gals, subsequently sinusoidal waves were applied to the models stepwise increasing a_{max} from 100 gals with an increment of 100 gals until the models exhibited ultimate failure. On the other hand, in the test on model 1, the sinusoidal shaking test and irregular shaking test were carried out independently.

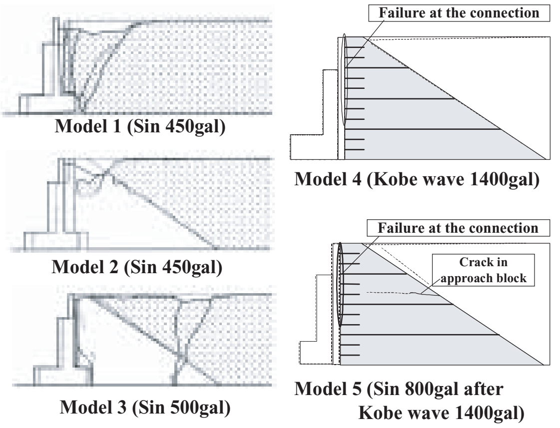

Figure 4. The deformation of the models after shaking

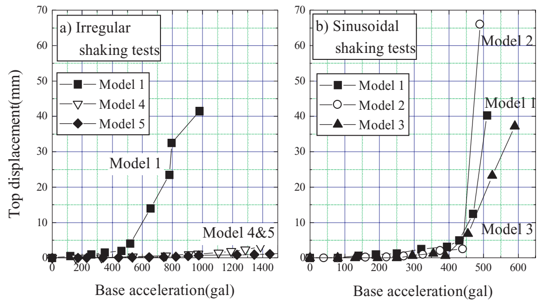

Figure 5. The residual displacement of all abutments

3 TEST RESULTS AND DISCUSSION

3.1 *Behavior of models 1, 2 and 3 (conventional types)*

Figure 4 shows the deformed models after the respective test which was observed through the transparent side wall of the shaking table, while Figures 5a and b show the relationships between the residual displacement at the top of the facing and the a_{max} value for all the models, subjected to a) irregular and b) sinusoidal input motions. The following trends of behaviour can be seen:

1. The deformation of model 1 became very large, showing ultimate failure with a well developed single failure plane fully extending in the backfill, when a_{max} was 450 gals.
2. Model 2 exhibited brittle failure when a_{max} was 450 gals, where the deformation of the approach block of gravel became very large, in particular at the upper part.
3. The deformation of model 3, in particular the settlement at the crest of the approach block, was noticeably smaller than that of model 2. Despite the above, when a_{max} became 500 gals, the facing started separating from the approach block as a result of

a high dynamic response, because of no connection between them. Cracks developed at several places in the approach block, resulting into the loss of structural integrity.

These results shown above indicate that the seismic stability of these conventional types of abutment could be insufficient when subjected to high seismic load, while the seismic stability of abutment can be increased by the following three measures:

1) Constructing an approach block using a stiffer and stronger material such as cement-mixed soil can substantially reduce the settlement of backfill immediately behind the facing structure supporting a bridge girder.
2) A high integrity of the approach block can be ensured by arranging horizontal reinforcement layers preventing the development of cracks in the zones where the tensile stress may exceed the tensile strength of cement-mixed soil, despite that an increase in the shear strength by using reinforcement layers of cement-mixed soil before the appearance of cracks cannot be expected.
3) The ends of reinforcement layers should be connected to the back of the facing structure directly supporting a bridge girder to restrain a relative settlement between them and to ensure a high integrity of the whole abutment structure.

Based on the above, eleven layers of horizontal reinforcements were placed inside the approach block of cement-mixed soil of models 4 and 5 with the ends connected by soldering to the back face of the facing structure (Figure 2).

3.2 *Behavior of models 4 and 5 (proposed new types)*

Figure 6 shows the relationships between the maximum and residual displacement at the top of the facing and the a_{max} value for models 4 and 5, subjected to a) irregular and b) subsequently sinusoidal input motions, together with the relationship between the residual displacement of the facing and the a_{max} value for model 1 subjected to sinusoidal waves as reference. The following trends of behaviour may be seen:

1) Models 4 and 5 were much more dynamically stable than model 1 (i.e., the most conventional type abutment).
2) With models 4 and 5, the increasing rate of the facing displacement with a_{max} started becoming larger at a certain level of shaking. It was found that this change in the behaviour was due to the start of progressive failure from the top of the connection between the facing and the reinforcement as noted from a sudden change in the reading of tensile strain at the respective reinforcement.
3) The tensile rupture of the connection started when a_{max} of irregular waves became 1,400 gals with model 4 and when a_{max} of sinusoidal waves became 800 gals (after having applied a series of irregular waves) with model 5. The fact that the connection failure started earlier with model 4 (having a wider footing base of facing) than with model 5 (having a narrow footing base of facing) was due probably to larger relative vertical displacements between the approach block and the facing with model 4, compared with model 5 as a result of a larger footing size of model 4 (Figure 7).
4) Despite a smaller footing of the facing, model 5 was more dynamically stable than model 4. As discussed in the next section, this apparently contradicting behaviour was due to the characteristic features of the resistance mechanism with this type of abutment.

The fact described above indicates a significant importance of the connection strength for a high seismic stability of this type of abutment structure. It was considered however that the connections between the reinforcement and the facing of full-scale prototype structures were not properly modeled in this study: that is:

1) The model reinforcement was made of phosphor bronze to reliably measure the tensile force of reinforcement, which resulted in a too high stiffness of reinforcement compared to that of geogrids that are usually used with prototype structures.

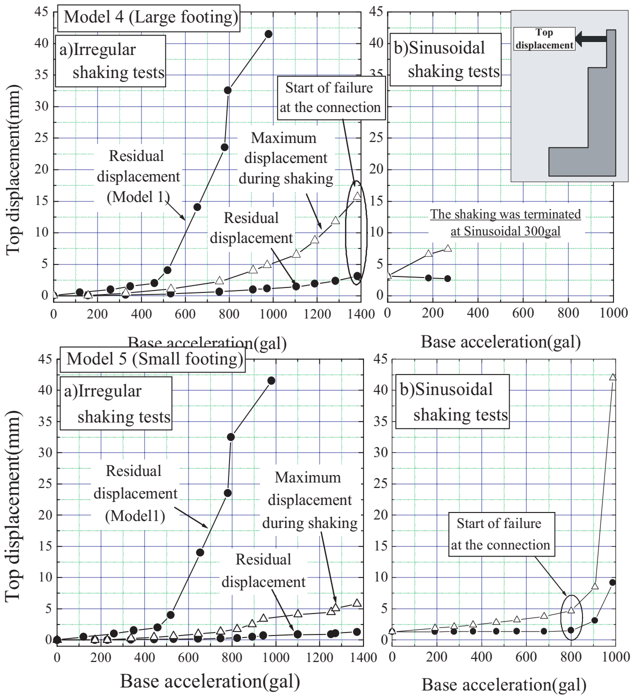

Figure 6. The displacement of proposed type abutment

2) The connection between the reinforcement and the facing

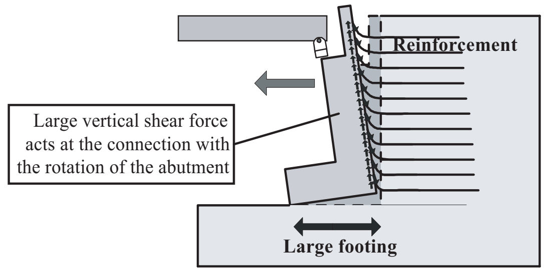

Figure 7. The shear force acting at the connection of reinforcement

made by soldering in this study was too stiff while too weak (i.e., brittle), compared with the one that would be actually used with prototype structures.

A study on these issues is now underway.

3.3 *Dynamic disturbing force acting on reinforced abutment*

Before the start of this study, the authors considered as follows:

1) The dynamic disturbing force that destabilizes the abutment during earthquakes consists of; 1a) dynamic earth pressure; 1b) inertia force of the facing structure; and 1c) dynamic load of the girder applied at the top of the facing structure.
2) The resisting force consists of; 2a) the tensile force in reinforcement; and 2b) the dynamic reaction force acting on the base of the footing of the facing. It is shown below however that factor 1a is not disturbing force, but it is a part of resisting force.

Figure 8 shows the time histories of dynamic components of 1a, 2a and 2b together with the displacement at the top of the facing and the input acceleration at one shaking stage using irregular waves with a_{max}= 539 gals on model 4. The following trends of behaviour may be seen from this figure:

1) Under dynamically active condition where the dynamic component of the displacement of the facing was directing outwards, the resisting components (i.e., the reaction force near the toe of footing and the reinforcement tensile force) increased. At this moment (as denoted A in Figure 8), the contact force near the heel of the footing decreased, indicating overturning displacements of the facing structure.

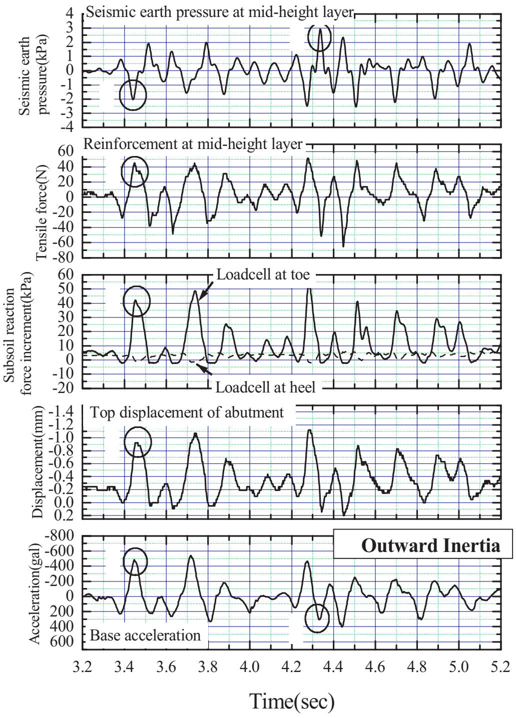

Figure 8. Typical time history of external forces for reinforced abutment Model 4 (Irregular shaking, a_{max}=539gal)

2) Importantly, under this active condition, the dynamic component of earth pressure decreased, showing that the facing structure was less stable than the backfill including the approach block. On the other hand, the maximum earth pressure in each cycle of dynamic loading was attained under dynamically passive condition (Point B in Figure 8).

These trends of earth pressure are opposite to those assumed in the design of conventional retaining walls, in which the dynamic active earth pressure is considered to destabilize the retaining structure under dynamically active condition. Another important implication of this fact is that a high connection strength between the facing and the reinforcement is essential for a high seismic stability of this type of bridge abutment.

Similar tread of earth pressure was found in other shaking table tests using retaining wall model where the phase relationship between the earth pressure and the inertia force changed with the intensity of shaking (Watanabe et al. 1999). Further investigation is required on the relationship between the seismic earth pressure and the dynamic response of soil and structure.

3.4 *Effects of the width of base footing*

Figure 9 shows the relationship between the maximum values of the resistant moment acting at the footing, defined about the heel of the footing, and the rotation angle of the facing during each shaking stage using irregular waves for models 4 and 5. Figure 10 shows the corresponding relationships between the reinforcement tensile force measured at the point closest to the back of facing (see Figure 2). It may be seen from these figures that the resistance moment acting at the footing was larger with model 4 (having a wider footing) than with model 5 (having a narrower footing), while the opposite is true with the tensile force mobilized in the reinforcement. That is, the major resisting force for model 5 was the tensile force at the connection between the facing and the reinforcement, and for this reason, model 5 was more stable than model 4. It is to be noted that with model 5, larger connection force resulted in larger tensile force in the reinforcement with larger load transmitted to the cement-mixed soil approach block, which resulted in the development of a horizontal crack in the approach block (Figure 4). This type of crack did not develop in model 4.

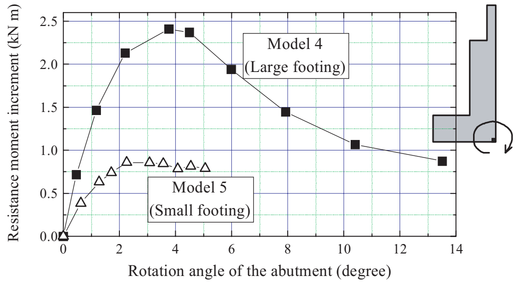

Figure 9. Relation between the resistance moment from subsoil and rotation angle

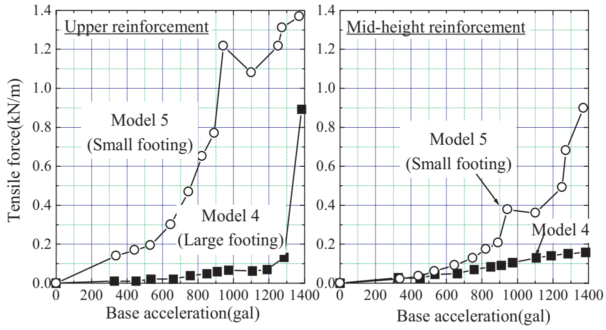

Figure 10. Relation between the tensile force in reinforcement layer and the base acceleration

This result suggests that when the connection strength and the tensile strength of the approach block can be designed to be sufficiently high, the size of the footing of the facing can be made rather small as model 5, which makes this type of abutment more cost-effective. In addition, to develop the limit state-based design procedure evaluating the deformation of structure and the ultimate failure state for this new type of bridge abutment, further study will be necessary on a number of topics, including the strength and deformation characteristics of cement-mixed gravel that will be used to construct prototype approach blocks.

4 CONCLUSION

The following conclusions can be derived from test results presented in this paper:

1. The seismic stability of several conventional types of railway bridge abutment is not sufficiently high when subjected to a high intensity of seismic load.
2. To increase the seismic stability of bridge abutment, it is efficient and cost-effective to construct an approach block of stiff and strong material, such as cement-mixed soil, that is reinforced with geogrid layers with the ends connected to a full-height rigid facing structure supporting a bridge girder.
3. A high strength of the connection between the reinforcement layers and the facing structure is essential not only to restrain the settlement of the backfill relative to the facing structure but also to dynamically stabilize the facing structure, which is less stable than the approach block.

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