DAMAGE TO RESIDENTIAL RETAINING WALLS AT THE GENKAI-JIMA ISLAND
INDUCED BY THE 2005 FUKUOKA-KEN SEIHO-OKI EARTHQUAKE

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ABSTRACT

The 2005 Fukuoka-ken Seiho-oki earthquake caused heavy damage to residential areas at the Genkai-jima Island. A field survey was carried out to investigate the damage of retaining walls in the residential areas. From the survey of 218 retaining walls, it appeared that 83% of the retaining walls were damaged and 61% of the whole lost their functions (collapsed or distorted/cracked). Especially in masonry retaining walls using natural rocks and boulders, the damage was serious. The damage features of dry masonry retaining walls (collapsed: 62%) and wet masonry retaining walls (collapsed: 25%) were different, and the difference in the earthquake-resistant capacity between the two retaining walls became obvious. Moreover, it appeared that most of the residential houses are built on transitions of cut slopes and fill embankments. It seems that this condition is one of a great factor which causes the heavy damage. Only 17% of the retaining walls escaped from any damage. Furthermore, to evaluate the collapse mechanism of masonry retaining walls during earthquake, a theoretical model based on the earth pressure theory was proposed. Through the theoretical considerations, it became possible to quantitatively evaluate the stability of the masonry retaining walls.

Key words: damage, earth pressure, earthquake, masonry retaining wall, retaining wall, slip line method (IGC: B4/E5/E6)

INTRODUCTION

Residential areas of the Genkai-jima Island locating about 10 km northwest of Fukuoka City were seriously harmed by the Fukuoka-ken Seiho-oki earthquake which occurred at 10:53 AM on March 20, 2006. The island is located about 8 km southeast of the epicenter as shown in Fig. 1. Since seismometers were not set up, actual seismic intensity at the Genkai-jima Island is unclear, however, a research group from the Earthquake Research Institute, University of Tokyo estimates that the seismic intensity of the main shock was about 6 (Applied Seismology Laboratory & Strong Motion Observation Office, Univ. of Tokyo, 2005). According to a report dated Jan. 31, 2006 by the Fire Defense and Disaster Prevention Division of Fukuoka Prefectural Government, 107 houses were completely collapsed, 46 were half and 61 were partially collapsed. The number of completely collapsed houses at the Genkai-jima Island accounts for 80% of total number of completely collapsed houses during the earthquake. Due to the serious damage, almost all the residents were compelled to take refuge out of the island. As shown in Photo 1, the houses are crowded on steep slopes, consequently, damage of residential areas was enormous, especially on retaining walls. Field surveys with respect to 218 retaining walls of the Genkai-jima Island were carried out by the investigation committee of the Japanese Geotechnical Society. In this paper, outlines of the investigation results are presented, and then the collapse mechanism of masonry retaining walls during earthquake are discussed based on the earth pressure theory.

GENKAI-JIMA ISLAND

The Genkai-jima Island is an elliptical island with 1.5 km in diameter of longer axis and 4 km in perimeter, and forms mountainous features with its top of 218.3 m in height above sea level. The hillsides slope down gradually from 120 to 150 m above sea level, and steep slopes form...
near the seashore. As shown in Fig. 2, the residential area has concentrated in the south of the island so that the residents can avoid a seasonal northwest wind. The houses are built on mainly two areas; one is on the steep slopes of the coastal terrace and second is on the gentle slopes along the circuit road. Most of the houses are built on masonry retaining walls using natural rocks and boulders.

As shown in Fig. 3, the basic rock of the island consists of the Shikano-shima granodiorite, and the surface is covered with weathered granite soils. Basaltic lavas form a cap rock as far as 100 m above sea level from the mountain top. At the end of the lavas, volcanic breccias can be seen locally.

Collapsed soils, which consist of weathered granite soils and basaltic lavas, are distributed onto the residential areas including ground of the houses and the backside slopes. New runoniarites were not observed around the residential areas. Most of the houses are built on embankments prepared by cuttings of hillside, that is, the houses are on the transitions of cut slope and fill embankment. Furthermore, more than 70% of the ground preparations and constructions of masonry retaining walls were achieved by manual work, therefore, it can be said that the strength of the grounds has became weaker than that of original ground.

**DAMAGE TO RETAINING WALLS**

*Site Investigation and Classification of Damage*

A field survey organized by the investigation committee of the Japanese Geotechnical Society was conducted to examine the damage to the retaining walls at residential areas. The investigation was conducted with respect to following items:

1. Types of retaining wall (see Table 1): a) dry masonry, b) wet masonry, c) gravity type, d) concrete block type, e) reinforced soil type, f) slope protection.

2. Measurements of retaining wall: a) height, b) breadth, c) depth, d) inclination.

(4) Damage categorization: a) collapsed, b) distorted/cracked, c) cracked, d) undamaged.

As to the damage categorization, the classification was carried out by following definitions (see Table 2):

a) Collapsed: a state that the structural materials of the retaining wall does not keep the original form and completely lost its functions.

b) Distorted/cracked: a state that the structural materials of the retaining wall are heavily distorted and/or cracked and but barely keep the original form. The functions are seemed to be lost and restoration is required.

c) Cracked: a state that the original form are almost kept but locally cracked. The functions are held.

d) Undamaged: a state that the structural materials of the retaining wall completely remains the original form.

Steep slopes of the residential areas fronts toward south and then most of the retaining walls face toward south to southeast. Because almost all the seismic motions were prominent in northwest to southeast directions by lateral faults, the damage was mainly seen
at retaining walls facing toward south. This survey mainly dealt with the retaining walls facing toward south, then small sidewalls along paths were not mentioned except for great damage. The number of the investigations totally amount to 218 retaining walls.

**Formation and Distribution of Retaining Walls**

Figure 4 shows distribution of retaining walls observed at the residential areas. As shown in this figure, the masonry retaining walls including dry and wet types occupy the most part of the slope, and gravity type concrete retaining walls and/or concrete block type retaining walls are seen at flatlands along the circuit road and public facilities. Reinforced soil type retaining walls are observed only at two places, namely, at the Genkaijima park and southern part of the Genkai elementary school. There are a lot of wet masonry retaining walls which were reinforced later by filling mortar into the openings of dry masonry retaining walls. Figure 5 shows the composition ratio of retaining wall types. It can be seen that the wet and dry masonry retaining walls amount to 74% of the whole. From Figs. 4 and 5, it becomes clear that almost all houses have individual retaining wall, which are mostly masonry retaining wall.

Damage category with regard to the types of retaining walls is listed in Table 3 and composition of the damage groupings with respect to total number of retaining walls is presented in Fig. 6. From the table and figure, it can be seen that 83% of the retaining walls were damaged and 61% of the whole lost their functions (collapsed or distorted/cracked). Percentage of retaining walls escaped from damage was only 17%. From easy calculations based on Table 3, it became clear that the masonry retaining walls accounts for 74% of the whole retaining walls and 39% of them had collapsed.

Figure 7 shows the distribution of the damage situations at southern part of the island. The collapses are remarkably seen in areas between the lowland along the circuit road and the marine terrace of 10–15 m height above sea level.

**Relationships between Retaining Wall Type and Damage**

Pie charts of composition ratio of the retaining wall type with respect to damage category are shown in Fig. 8. As to the collapse (a), the dry masonry retaining walls account to 62% of the whole collapses, and by adding the wet masonry type, 97% of this category is occupied by the masonry retaining walls. This result indicates that the

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**Table 3. Summary of damage**

<table>
<thead>
<tr>
<th>Retaining wall type</th>
<th>Symbol</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Collapsed</td>
<td>Distorted/</td>
<td>Cracked</td>
<td>Undamaged</td>
<td>%</td>
</tr>
<tr>
<td>Dry masonry</td>
<td>①</td>
<td>40</td>
<td>18</td>
<td>6</td>
<td>9</td>
<td>73</td>
</tr>
<tr>
<td>Wet masonry</td>
<td>②</td>
<td>23</td>
<td>25</td>
<td>31</td>
<td>10</td>
<td>89</td>
</tr>
<tr>
<td>Gravity type</td>
<td>③</td>
<td>2</td>
<td>8</td>
<td>8</td>
<td>16</td>
<td>34</td>
</tr>
<tr>
<td>Concrete block type</td>
<td>④</td>
<td>0</td>
<td>3</td>
<td>1</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>Reinforced soil type</td>
<td>⑤</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>0</td>
<td>2</td>
</tr>
<tr>
<td>Slope protection</td>
<td>⑥</td>
<td>0</td>
<td>15</td>
<td>46</td>
<td>36</td>
<td>15</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>65</td>
<td>71</td>
<td>46</td>
<td>36</td>
<td>218</td>
</tr>
<tr>
<td>%</td>
<td></td>
<td>30</td>
<td>33</td>
<td>21</td>
<td>17</td>
<td>100</td>
</tr>
</tbody>
</table>
Table 4. Damage and geological/geotechnical features

<table>
<thead>
<tr>
<th>Geological/geotechnical features</th>
<th>Observed number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Basaltic lavas</td>
</tr>
<tr>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>Embankment</td>
<td>2</td>
</tr>
<tr>
<td>Cut</td>
<td>2</td>
</tr>
<tr>
<td>Transition</td>
<td>5</td>
</tr>
<tr>
<td>Unclear</td>
<td>0</td>
</tr>
<tr>
<td>Total</td>
<td>9</td>
</tr>
</tbody>
</table>

Subtotal: 11  40  169  220

A: collapsed, B: distorted/cracked, C: cracked, D: undamaged

Fig. 6. Composition ratio of damage category

Fig. 7. Distribution of damage to retaining walls

masonry retaining wall, which is made of irregular rocks and boulders, is not resistant to seismic motions. As for the chart of distortion/crack (b), the masonry retaining walls amount for 60% together with the dry and the wet type. All the reinforced soil type retaining walls and the slope protections are categorized into this group. For the category of crack (c), 68% is amounted by the wet masonry retaining walls. The composition ratio of the wet masonry retaining walls makes a contrast with the dry ones shown in the chart of collapse (a). In the chart of undamage (d), approximately half of the ratio is occupied by the gravity type retaining walls. It can be said that this result is due to the high earthquake-resistant capacity and the situation of the gravity type retaining walls. At the Genkai-jima Island, the gravity type retaining walls are generally placed at flatlands along the circuit road.

Pie charts of composition ratio of the damage category with respect to the retaining wall type are shown in Fig. 9. In case of dry masonry retaining walls (1), percentage of the collapse is prominent. Damage of which functions are lost (collapsed and distorted/cracked) accounts to 80%. Therefore, the dry masonry retaining walls were seriously damaged by this earthquake and this composition ratio revealed the vulnerability to the seismic motion. As to the wet masonry retaining walls (2), the percentage of collapse occupies only 26% but the ratio of crack increases. That is, damage of the wet masonry retaining walls is mainly composed of cracks. It is thought that mortar filling restricts popping-out of rocks and consequently improves the damage circumstances. The damage of which functions are lost is 54% and it is 30% improvement over dry ones. This result suggests that mortar filling is effective for the disaster prevention. From Fig. 9
(3), it can be seen that the percentage of collapse and distortion/crack of gravity type retaining walls is low compared to the masonry retaining walls. However, it is impossible to say definitely that the gravity retaining walls are seismic-resistant, because the gravity retaining walls tend to be set up on the stable site such as flatland along the circuit road, and this geographical condition may significantly influence on the damage. As for the concrete block type retaining walls Fig. 9 (4), the damage of distortion/crack is outstanding but no collapse is found. It seems that homogeneity of the structural material of the concrete blocks results in the reduction of serious collapses. In this survey, two reinforced type retaining walls were observed, and both are distorted or cracked Fig. 9 (5). It was observed that reinforced materials were partially torn; nevertheless the whole did not collapse. The slope protections observed at east side of the residential area are made up of shotcrete (upper part) and anchoring method (lower part). It was observed that the slopes were pushed out by the seismic motions and the borders of the slopes generated gaps. Cracks on the mortar protections were often seen but the damage of which functions are completely lost was not observed (Fig. 9 (6)).

Figure 10 shows damage composition with respect to the height of retaining walls. As shown in the figures, height of the masonry retaining walls is concentrating at 1 to 3 m, and the wet type shows slightly higher distribution compared to the dry ones. As to the gravity retaining walls, the height is evenly distributed to 1 to 4 m. Higher retaining walls seem to have a tendency of easily being damaged, however, clear relationships between damage composition and the height of retaining wall are not found from Fig. 10.

Figure 11 shows damage composition with respect to the inclination of retaining walls. Note that the inclinations listed are present states after the earthquake. As shown in the figures, inclinations of masonry retaining walls are concentrating at 1:0:1, but clear relationships between damage composition and the inclination of retaining wall are not confirmed.

Because almost all the residential areas are built on the steep slopes, the grounds of the houses are composed of cut slopes and fill embankments. As mentioned above, the soils consist of whethered granite soils and basaltic lavas. Table 3 shows lists of the damage category and the geological features. Note that there are a lot of places where the soil types could not be examined according to the states of the damage, therefore the examinations of the soil are inevitably limited to the serious damage spots. Consequently, the observed number for slight damage becomes small. 54% of the geological features are unidentified in this survey. Photo 2 shows a typical example of the collapse observed at a transition of cut slope and fill embankment. In this survey, 15 spots of the transition of cut slope and fill embankment were found.

![Fig. 9. Composition ratio of damage with respect to retaining wall type](image)

![Fig. 10. Damage composition with respect to height of retaining wall](image)
and 11 of them are collapsed.

**STABILITY ANALYSIS OF MASONRY RETAINING WALL**

**Modeling of Masonry Retaining Wall**

To evaluate the stability of masonry retaining walls in seismic motions, a plane strain model as shown in Fig. 12 was considered in this study. The rocks and boulders of a masonry retaining wall are assumed as rectangular blocks. Moreover, active earth pressure including effects of inertial force induced by seismic motion is considered to act on the backside of the retaining wall, and the dead load and the inertial force of the retaining wall are simultaneously considered to act on the blocks themselves. Soil parameters and calculation conditions used in this analysis are listed in Table 5. These are typical and representative values measured in this survey. The seismic active earth pressure is obtained by numerical calculations based on the method of stress characteristics (the slip line analysis).

**Stability Analysis for Slippage and Tumble**

Assuming that a section of a retaining wall partitioned from the upper end to a position at a distance of x is a one block, the forces acting on the block can be shown as Fig. 13. As an external force, an active earth pressure (lateral resultant force, \( P_a \) and vertical resultant force, \( P_v \)) acts on the backside of the retaining wall, and a unit weight of the block, \( W \) and an seismic inertial force, \( k_s W \) act as body forces. The total resultant force, \( Q \) acts on the point of an imaginary section A-A' (the lower end of the block) at a distance of \( e \) from the center, and the resultant force is balanced with a reaction force, \( R \). Friction and adhesion are considered to exist on the A-A' section. The difference between the dry masonry retaining walls and wet ones is expressed by presence of adhesion acting between the blocks. The dry masonry retaining walls are assumed by disregarding the adhesion.
Table 5. Soil parameters and calculation conditions

<table>
<thead>
<tr>
<th>Conditions of retaining wall</th>
<th>Height, $H$ (m)</th>
<th>1.0 − 5.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inclination, $n/m$</td>
<td>0.0 − 0.4</td>
<td></td>
</tr>
<tr>
<td>Breadth, $(m)$</td>
<td>0.3 − 0.5</td>
<td></td>
</tr>
<tr>
<td>Unit weight of rocks, $(kN/m^3)$</td>
<td>25.0</td>
<td></td>
</tr>
<tr>
<td>Adhesion between rocks, $a$ $(kN/m^2)$</td>
<td>0 − 600</td>
<td></td>
</tr>
<tr>
<td>Backfill soil parameters</td>
<td>Cohesion, $c$ $(kN/m^2)$</td>
<td>0, 10</td>
</tr>
<tr>
<td>Internal friction angle, $\phi$ (deg)</td>
<td>30, 35</td>
<td></td>
</tr>
<tr>
<td>Unit weight, $y$ $(kN/m^2)$</td>
<td>18, 20</td>
<td></td>
</tr>
<tr>
<td>Friction angle between soil and retaining wall (deg)</td>
<td>$2/3 \phi$</td>
<td></td>
</tr>
<tr>
<td>Adhesion between soil and retaining wall $(kN/m^2)$</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Calculation conditions</td>
<td>Surcharge, $q$ $(kN/m^2)$</td>
<td>3.0</td>
</tr>
<tr>
<td>Lateral seismic coefficient, $k_s$</td>
<td>0.0 − 0.4</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 14. Forces acting on A-A' section

For the modeling of slippage at the A-A’ section, the balance of the shear force, $S_h$, and the resistance force, $R_h$, are examined. Figure 14 shows the mechanism of forces acting on the A-A’ section. The shear force, $S_h$ and the resistance force, $R_h$ are expressed as follows:

$$ S_h = P_h + k_i W $$  \hspace{1cm} (1)

$$ R_h = \mu (P_r + W) + a B $$  \hspace{1cm} (2)

where, $S_h$: shear force on the A-A’ section, $P_h$ and $P_r$: lateral and vertical resultant earth pressure (obtained by numerical calculations), $k_i$: lateral seismic coefficient, $W$: dead load of the block section, $\mu$: friction coefficient, $a$: adhesion by filling of mortar, $B$: breadth of retaining wall. The stability for the slippage will be satisfied when $R_h$ exceeds $S_h$, that is, the safe condition for the slippage can be expressed as:

$$ S_h \leq R_h $$  \hspace{1cm} (3)

As for the modeling of tumble, the balance of the rotational moment, $M_{rot}$ with respect to point A and resistance moment, $M_{res}$ are examined. When the following condition is satisfied, the stability for the tumble will be satisfied:

$$ M_{rot} \leq M_{res} $$  \hspace{1cm} (4)

$M_{rot}$ and $M_{res}$ are expressed as follows:

$$ M_{rot} = P_h h + \frac{1}{2} k_i W x $$  \hspace{1cm} (5)

$$ M_{res} = P_r I_1 + W l_2 + R_h d $$  \hspace{1cm} (6)

where, $h$: vertical distance from A-A’ section to the point of action of the resultant earth pressure, $l_1$: horizontal distance from the point A to the point of action of the resultant earth pressure, $l_2$: vertical distance from the point A to the point of action of the dead load, $R_h$: vertical reaction force on A-A’ section, $d$: horizontal distance from the point A to the point of action of $R_h$. As to $l_1$ and $l_2$, these lengths can be obtained by geometrical relationships of the retaining wall as shown in Fig. 13. As to $h$, the numerical calculations of the stress characteristics provide earth pressure distributions along the retaining wall so that the point of action of the resultant force can be known. When the backfill soil has cohesion and then negative earth pressure (i.e., tensile stress) partially appears at the backside of the retaining wall, the tensile stress is disregarded into the stability evaluations. While $d$ is the only undetermined parameter, it can be expressed as follows from a critical condition between $M_{rot}$ and $M_{res}$:

$$ d = \frac{P_r h + \frac{1}{2} k_i W x - P_r I_1 - W l_2}{P_r + W} $$  \hspace{1cm} (7)

It is generally known that the retaining wall becomes stable if the eccentric distance, $e$ is within the middle-third of the base of the retaining wall. This arises from a condition that a negative reaction force (i.e., tensile force) does not appear at the heel of the base. It seems that this condition can also be applied to the stability evaluation for dry masonry retaining walls which have no adhesions between the rocks. However, it is not always necessary to satisfy the condition in case of wet masonry retaining walls which have tensile strength by mortar fill-
ing. The vertical reaction force can be obtained from the soil reaction spring model and the distribution can be illustrated as Fig. 14. In the case of wet masonry retaining wall, the tensile stress appears at the heel of the base. According to the soil reaction spring model, the vertical reaction stresses represent two triangles as shown in Fig. 14. Because the summation of the vertical reaction stresses must coincide to the vertical resultant force on A-A' section, following relationship can be made:

\[
\frac{q_{v}^{+}}{2} - \frac{q_{v}^{-} (B - s)}{2} = P_{r} + W \tag{8}
\]

where \(s\) is a distance from point A to the inversion point of the positive and negative vertical reaction stresses. \(q_{v}^{+}\) and \(q_{v}^{-}\) are the maximum values of the positive and negative vertical resultant stresses, respectively and are expressed as:

\[
q_{v}^{+} = \frac{P_{r} + W}{B} \left(1 + \frac{6e}{B}\right) \tag{9}
\]

\[
q_{v}^{-} = \frac{P_{r} + W}{B} \left(1 - \frac{6e}{B}\right) \tag{10}
\]

Substituting Eqs. (9) and (10) into Eq. (8), \(s\) can be expressed as:

\[
s = \frac{3}{2} B - 3e \tag{11}
\]

Then, the maximum tensile stress, \(q_{v}^{-}\) can be rewritten as:

\[
q_{v}^{-} = \frac{P_{r} + W}{B} \left(1 + \frac{6e}{B}\right) \left(\frac{3B}{2} - 3e\right) - 2(P_{r} + W) \left(\frac{3e - B}{2}\right) \tag{12}
\]

Besides the stability condition of Eq. (4), as long as the absolute value of \(q_{v}^{-}\) does not exceed the adhesion, \(a\), the masonry retaining wall is thought to be stable for the tumble.

**Calculations for Seismic Active Earth Pressures**

To solve the earth pressure in seismic motions, Mononobe-Okabe equation (Okabe, 1926; Mononobe and Matsuo, 1929) based on the Coulomb’s wedge theory is well known. Nevertheless, it is difficult to take into account the cohesion of the soil and the method does not make it possible to provide stress distributions. In the meantime, method of stress characteristics has advantages of the Coulomb’s wedge theory, then, in this study, the seismic active earth pressures were calculated by the method of stress characteristics. The stress distributions along the retaining wall can be obtained by integrating differential equations called the Köttner’s equation. Under the plane strain condition of \((x, y)\) coordinate system, the Köttner’s equation which takes into account of the lateral seismic force can be expressed as:

Characteristics curve:

\[
\frac{dy}{dx} = \tan (\alpha \pm \eta) \tag{13}
\]

**Basic equations:**

\[
\pm \frac{d\sigma_{n}}{dx} + 2(\sigma_{n} \tan \phi + c) \frac{d\alpha}{dx} = \frac{\gamma \cos (\alpha \pm \eta) - k_{s} \gamma \sin (\alpha \pm \eta)}{\cos \phi \cos (\alpha \pm \eta)} \tag{14}
\]

where, \(\sigma_{n}\): mean principal stress, \(\alpha\): angle between the \(x\) axis and the major principal stress, \(\eta\): \(\pi/4 - \phi/2\), \(\gamma\): unit weight of the soil, \(c\): cohesion of the soil, \(\phi\): internal friction angle of the soil. Above equations are derived from the stress equilibrium and Mohr-Coulomb failure criterion, and the solutions obtained by numerical integrations are, as a matter of course, satisfied both conditions. Refer to other documents (e.g., Ishihara and Kumura, 1977; Kobayashi, 2002) for the details.

**Results and Discussions**

Figure 15 shows examples of calculation results of lateral earth pressure distributions along the retaining wall. From Fig. 15(a), it can be seen that lateral earth pressures increases as \(\phi\) decreases. When we imagine an object on an inclined board, it can be deduced that the object is easy to slip if the friction coefficient between the object and the board is small. Moreover, the inclination of the board when the object starts to slip becomes gentle if the friction coefficient is small. If the inclined board is a slip surface in the soil, we can infer that the failure region and the earth pressure becomes large when \(\phi\) is small, and consequently the soil is prone to collapse by the unit weight. Moreover, it can be said that reduction of the
earth pressure can be achieved by using high frictional soils. Figure 15(b) shows the effect of \( c \) on the earth pressures. It can be seen that the earth pressure is in proportion to the cohesion, and negative regions (tensile stresses) appear at the upper zone of the retaining wall when \( c \) is large. However, the negative regions will not participate in the stability of the retaining wall because the cohesion is not considered to hold the tensile strength between the soil and the retaining wall. In the meantime, considerable earth pressure reduction can be expected by use of soil having large \( c \). Figure 15(c) shows the effect of inclination of the retaining wall on the earth pressure. This figure shows that the earth pressure becomes small as the inclination increases. Figure 15(d) shows the effect of the lateral seismic coefficient on the earth pressure. The earth pressure increases as \( k_0 \) increases, but the increment of the earth pressures is not proportional to the increments of \( k_0 \), indicating that sensitivity of the earth pressures increases when \( k_0 \) is large. From these discussions, it is shown that earth pressure can be reduced by using soil with large strength parameters and appropriate design of retaining wall.

In this paragraph, the stability for slippage is discussed. Figure 16 shows the effect of \( c \) on the critical height, \( z_c \) for slippage of the dry retaining wall. The critical height, \( z_c \) indicates that the stability for slippage is ensured when a retaining wall of which the height is lower than \( z_c \) is applied. In this calculation, parameters listed in the figure were used. These are representative conditions that were observed at the Genkai-jima Island. As for inertia forces in the retaining wall systems, the maximum accelerations actually observed around north area of the Kyushu Island were 205–355 Gal, and these measurements give seismic coefficients as 0.21–0.36. However, it is difficult to assume the actual lateral seismic coefficient, therefore, following calculations were performed with \( k_s = 0.2 \). This value is generally used in practical design for the seismic coefficient method. From this figure, it can be seen that \( z_c \) increases as \( c \) increases, indicating that it becomes possible to build a higher retaining wall by use of the soil having large \( c \). Figure 17 shows the relationships between the shear forces, \( S_s \) and resistant forces, \( R_s \), acting along the backside of retaining wall. Figures 17(a) and (b) show the influences on the slippage with respect to the inclination, \( n/m \) and breadth, \( B \) of the retaining wall, respectively. It can be seen that the critical heights (intersection points of \( S_s \) and \( R_s \)) shift to larger as \( B \) and \( n/m \) increases, that is, the retaining wall becomes stable with increasing \( B \) and \( n/m \). From these figures, it became clear that \( n/m \) affects \( S_s \) and \( B \) affects \( R_s \). Moreover, it can be said that the stability can be greatly improved by increasing the breadth of retaining walls.

Figure 18 shows the influence of \( B \) and \( n/m \) of the retaining wall on the critical height, \( z_c \). Here, in order to assume wet masonry retaining walls, adhesion of \( \alpha = 600 \) kN/m\(^2\) as an allowable tensile strength is applied to the calculations. This figure shows that sensitivity of \( z_c \) increases with increasing \( n/m \) and sensitivity of \( n/m \) increases with increasing \( B \). This diagram will be helpful in designing retaining walls that ensure the stability of slippage.

In this paragraph, the stability for the tumble is discussed. Figure 19 shows the influence of \( c \) on the critical height for the tumble and allowable tensile strength of the dry masonry retaining wall. Here, \( z_{c_t} \) and \( z_{c_a} \) are the critical height for the tumble and the tensile strength, respectively. From this figure, as well as the case of the slippage shown in Fig. 16, it can be seen that both critical heights increase as \( c \) increases. Seeing that \( z_{c_t} \) is always below \( z_{c_a} \), we can assume that there is a case where
the stability of the retaining wall is kept by the resistant to the
tumble even if the stability for the allowable tensile
strength is broken. Therefore, it is thought that \( z_{0} \) gives a
criterion on safe side and \( z_{0} \) gives criterion of which the
actual tumble and crack appear. Figure 20 shows the
point of the reaction force, \( d \) and the maximum tensile
stress \( q_{t}^{1} \) acting on the A-A' section with variations of
\( n/m \) and \( B \). From Fig. 20(a), it can be seen that the stability
for the tumble is improved as \( n/m \) increases. In case of a retaining
wall of which the height is 2.5 m, it is shown that there is no danger of the tumble if \( n/m \) is around 0.25 or more. As shown in Fig. 20(b), when \( n/m \)
is comparatively small, \( q_{t}^{1} \) drastically changes at lower part of the retaining wall. Under the calculation conditions listed in this figure, it can be said that serious
tensile stresses do not appear when \( n/m \) is less than

around 0.3. Figure 20(c) shows that the stability for the
tumble can be improved by increasing \( B \). It is shown that
when \( B \) is around 0.6 m or more, the risk of the tumble
will disappear. At the Genkai-jima Island, rocks and boulders, whose breadths are 0.3–0.5 m, are used to construct the retaining walls, because a lot of retaining
walls seem to lose the stability owing to the thinness of the
breadth of rock walls. From Fig. 20(d), it is shown that the tensile stress is prone to be influenced as \( B \) decreases, and the sensitivity of \( B \) on \( q_{t}^{1} \) becomes low as
\( B \) increases. Therefore, the problem of tensile stress will disappear when the wet masonry retaining wall has a certain breadth and cohesion. Figure 21 shows variations of the critical heights with respect to the tumble and
tensile strength. It can be seen that the critical heights increase as \( B \) and \( n/m \) increases and the sensitivity of \( B \)
on the critical heights increases as \( n/m \) increases. As well as Fig. 18, these figures will also provide useful information
to design a stable retaining wall.

In this section, the stability of masonry retaining walls with respect to the slippage, tumble and tensile strength were discussed through the theoretical considerations. As
mentioned at the section of the damage report, many masonry retaining walls were damaged by the earthquake. Especially in dry masonry retaining walls which have no tensile strength, more than half of the retaining
walls had collapsed. The theoretical considerations prove that the dry masonry retaining walls at the Genkai-jima
Island had great risks for both slippage and tumble. The height of the wet masonry retaining walls observed at the
Island were larger than the dry ones on average, and so the structures might be unstable, but the adhesion by mortars filled in the openings of rocks seems to sig-
ificantly contribute to the stability of the slippages. As
an evidence, the percentage of the collapses of the wet masonry retaining walls decreases by half compared to that of dry ones as shown in Fig. 9. It can be said that the earthquake-resistant will be significantly improved by the
mortar fillings. The inclination most frequently used for
both the dry and wet masonry retaining walls was 0°01, and the breadth of the rocks composing the structures
was 0.6 or so. As can be seen from Figs. 17, 18, 20, 21, it
came clear that the effects of the inclination and
breadth of retaining walls on the stabilities are also...
remarkable. Considering that the size of the rock and boulder may be restricted owing to a difficulty in handling for the construction process, a role of design of the inclination is important for useful restorations. Besides the qualitative discussions above mentioned, quantitative evaluations for the earthquake-resistant design of masonry retaining walls could be shown by the proposed calculation model for the stability of masonry retaining walls. The authors hope that this model and calculation results would be helpful for suggestions of the restorations.

CONCLUSIONS

The residential retaining walls at the Genkai-jima Island were seriously damaged by the 2005 Fukuoka-ken Seiho-oki earthquake. A field survey was carried out to investigate the damage of retaining walls in the residential areas.

As a result of the survey of 218 retaining walls, it appeared that 83% of the retaining walls were damaged and 61% of the retaining walls lost their functions (collapsed or distorted/cracked). Especially in masonry retaining walls using natural rocks and boulders, the damage was serious. The masonry retaining walls including dry and wet masonry accounts for 74% of the retaining walls, and 39% of them were completely collapsed. For the dry masonry retaining walls, 62% of them were collapsed, while the fatal damage of the wet masonry retaining walls amounts to 25%. The difference in the earthquake-resistant capacity between the dry retaining walls and wet ones became obvious. As for the other types of retaining walls including gravity type, concrete block type, reinforced soil type and slope protection, distortions and/or cracks were seen but the collapses were hardly observed. Percentage of the retaining walls escaped from any damage was only 17%.

From the geological and geotechnical investigation, it appeared that collapsed soils, which consist of whethered granite soils and basaltic lavas, are distributed onto the residential areas. Most of the residential houses are built on the transition of cut slope and fill embankment. It seems that this condition is one of important factors which causes the collapses.

Moreover, to evaluate the collapse mechanism of masonry retaining walls during earthquake, a theoretical model based on the earth pressure theory was proposed. Through the theoretical considerations, it became possible to quantitatively evaluate the stability of masonry retaining walls. Authors hope that this model and calculation results would be helpful for the restoration and the design of retaining walls.

REFERENCES