The Ninth International Symposium on Mitigation of Geo-disasters in Asia

Yogyakarta, 19-20 December 2011

Editors
Teuku Faisal Fathani
Masakatsu Miyajima
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Department of Civil and Environmental Engineering, Universitas Gadjah Mada, Indonesia
Laboratory of Earthquake Engineering, Kanazawa University, Japan

Supported by:
MSD NetWork
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Multimodal Sediment Disaster Network
Disaster Management Network
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PREFACE

The first International Symposium on Mitigation of Geo-hazards in Areas around Japan Sea was held in 2003 at Kanazawa University, Japan. Its participants comprised researchers and engineers in China, Korea, Taiwan and Japan who were interested in hazard mitigation in areas around Japan Sea. The second, third and fourth symposiums were also held at Kanazawa, Japan in 2004, 2005 and 2006. The fifth to seventh symposiums were held in Xi’an, Kunming, and Harbin, China from 2007 to 2009, respectively. The eighth symposium was held in Vladivostok, Russia in 2010.

In the last eight years, the disasters occurred in Asian countries have been reported and examined. Through the symposiums, communications were well conducted and the network is gradually formed and enlarged. The International Symposium on Mitigation of Geo-disasters in Asia in 2011 is held in the Department of Civil and Environmental Engineering, Universitas Gadjah Mada, in collaboration with Laboratory of Earthquake Engineering, Kanazawa University, Japan and Department of Geoscience, Shimane University, Japan and supported by Multimodal Sediment Disaster Network (MSD-Network) and Disaster Management Network (DMN-Network).

Around 29 papers from 4 countries (Japan, China, Iran and Indonesia) are presented and discussed in this symposium. The papers are of various disaster related topics, i.e. earthquake related disasters, volcano related geo-hazards, heavy rainfall related geo-hazards, heavy snow related geo-hazards, active tectonics related geo-hazards, ground fissures caused damages, damage analysis, monitoring system, countermeasures, and risk management.

The organizing committee would like to extend its deepest gratitude to all participants who have contributed their papers and all parties involved throughout the symposium without which this conference would not have been a success. The organizing committee wishes all participants a fruitful discussion during the symposium and an enjoyable stay in Yogyakarta.

Yogyakarta, 19 December 2011

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Mechanisms of Earthquake-induced Permanent Deformations in Riverbank Slope and Its Countermeasures

Teuku Faisal Fathani · Hefryan Sukma Kharismalatri
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Abstract. After the occurrence of the $M_w$ 6.3 Mid-Java Earthquake 2006, permanent deformations were found along the riverbank slope of Code River, in the center of Yogyakarta City. About 250 m long and 1-5 cm wide crack causes damage in houses built on a 15-20 m-high steep slope. Further, deformations in the embankment body occurred and the gabion on downslope moved due to seismic induced active lateral earth pressure. This research is aimed at assessing the mechanisms of seismically induced permanent deformations in slopes, and examining appropriate countermeasures. The existing slope stability in term of safety factor was calculated by using finite element model considering phi-c reduction method. The stability analysis considers various load conditions, groundwater level and river elevation. The result shows that the safety factor of the existing slope is less than 1.00 with maximum deformation of 111 mm and 211 mm in case of applied fixed load and seismic load, respectively. Furthermore, in order to stop or reduce the deformation, the construction of river bank protection in the form of cantilever wall and gabion is designed and examined. It is clarified that several factors affecting slope stability are physical properties of soil materials, external loading condition, slope inclination, groundwater condition, and seismic forces. Other factors that need to be taken into consideration are erosion in the outer river bend and reduction of river wet perimeter that causes erosion on the toe of embankment slope.

Keywords. Slope deformation, seismic forces, riverbank protection, stability analysis

1. Introduction

The Yogyakarta earthquake on 27 May 2006 resulted in more than 150 landslides, mainly distributed along the western and northern cliffs of the high land of Yogyakarta Special Province (Fathani and Kamawati, 2009). Most of these earthquake-induced landslides occurred as rock falls and/or rock slides of various sizes. The failed slopes occurred in various rock types, such as tuff – breccia, pumice breccia, tuff sandstone, tuff, andesitic breccia and limestone which were persistently jointed (Rahardjo et al., 1995; Sudarno, 1997).

Other than landslide occurred at western and northern cliffs of the high land, permanent deformations were found along the riverbank slope of Code River, in the center of Yogyakarta City. About 250 m long and 1-5 cm wide crack causes damage in densely houses built on a 15-20 m-high steep slope. Other damages at the embankment include the deformation at embankment body due to seismic induced active lateral earth pressure, land subsidence at several locations, malfunction of embankment drainage, and movement of the gabion on the toe of embankment slope. In addition, erosion in the outer river bend and reduction of river wet perimeter that causes erosion on the toe of embankment slope were also occurred.

Stability analysis should be conducted to assess the existing condition of riverbank slope of Code River. Therefore, mitigation and prevention measures can be conducted to prevent the riverbank slope from failure. In this study, a stress-strain analysis by using finite element method was conducted. Design and examination on the proposed construction of river bank protection in the form of cantilever wall and gabion were also conducted.

2. Damages of Riverbank Slope due to the Yogyakarta Earthquake 2006

Based on the field observation in Code River, Yogyakarta City, some damages were found and elaborated as follows:

a. Deformations of the riverbank slope

The slope at the southern part of the study area experienced deformation of masonry structure in the embankment body and the movement of gabion on the downslope toward the river (Figure 1). These structures were not able to resist the seismic induced active lateral earth pressure, and thus experienced deformations. The deformations were more severe during and after the $M_w$ 6.3 Yogyakarta Earthquake on 27 May 2006.

![Figure 1. Deformation at the southern part of the slope; (a) deformation at the embankment; (b) gabion movement.](image-url)
At the northern part of the slope, particularly at the embankment, damage in the form of 1-5 cm wide crack in the masonry revetment was found. The deformation caused land subsidence which could be seen clearly at the downslope. The deformation was more severe due to the malfunction horizontal drainage at the masonry revetment (Figure 2).

![Figure 2. Crack at the northern part of the slope.](image)

b. Erosion in the foundation of masonry revetment

In the revetment foundation of the outer river bend, erosion occurred due to strong river flow in rainy season and the change of river wet perimeter (Figure 3). The erosion could reduce the stability of the riverbank slope, therefore, necessary measure should be conducted immediately.

c. Reduction of river wet perimeter

River wet perimeter reduction is caused by illegal constructions at the riverbank and fish cages built by the local people (Figure 3). River wet perimeter reduction mostly occurred in the inner river bend would cause the stream in the outer river bend stronger and erode the revetment foundation.

![Figure 3. River wet perimeter reduction](image)

d. Cracks at the housing area

In the houses located at the upper part of the riverbank slope, a horizontal crack was found. The crack was located 4-5 m from the slope crest and was 1-5 cm wide. Other smaller cracks were also found at several locations, particularly in houses with permanent walls which give bigger external load contribution to the slope.

3. Results of Soil Investigation

Data used in the analysis includes Dutch Cone Penetration (DCP), hand bor, and laboratory test. The observation on groundwater level depth, soil surface condition and external loadings on the slope were performed. Other than the above mentioned test, geometrical measurement of the riverbank slope was also conducted.

3.1. Geotechnical condition

The geotechnical condition of the subsurface soil shows the following sand layer stratification. The first layer is silty sand layer, soft to medium which was found up to 2.50 - 3.50 m depth, with average cone resistance ($q_c$) of 20 kg/cm². Below the silty sand layer, dense gravelly sand layer of 1.0 to 3.0 m thick with average $q_c$ of 40 kg/cm² was found. In the next layer, greyed medium to dense sand, with average $q_c$ of 35-40 kg/cm² was found. The result of laboratory test on the dense gravelly sand shows that the soil can be classified as SW or SP group which is comprised of well graded sand to poorly graded sand and gravelly sand which contain small amount or no finer (USCS, 1985). The water content in this depth is 11.76%. The engineering properties of the soil are: saturated unit weight ($\gamma_s$) = 1.683 gr/cm³; dry unit weight ($\gamma_d$) = 1.506 gr/cm³; specific gravity ($G_s$) = 2.604; internal friction angle ($\phi$) = 35.8°; and cohesion ($c$) = 0.55 kg/cm².

Medium to dense sand in the bottom part is classified as SW or SP group which is comprised of well graded sand to poorly graded sand and gravelly sand which contain small amount or no finer. The water content of the soil is 19.48%. The engineering properties of the soil are: saturated unit weight ($\gamma_s$) = 1.856 gr/cm³; dry unit weight ($\gamma_d$) = 1.553 gr/cm³; specific gravity ($G_s$) = 2.779; internal friction angle ($\phi$) = 31°; and cohesion ($c$) = 0.81 kg/cm².

In the stress-strain analysis, modulus of elasticity of the soil was examined using three methods i.e. CPT interpretation, borelog data interpretation, and by stress-strain relationship based on direct shear test result. For dense gravelly sand, $E = 8000$ kN/m² and $E = 2000$ kN/m² for medium to dense sand in the bottom are used.

Interface reduction factor ($R_{int}$) is used to illustrate the interaction behavior between soil (internal friction angle and cohesion) and structural element (adhesion and shear force). $R_{int}$ is equal to one (rigid), meaning that interface does not affect the soil strength around structural element. In this analysis, $R_{int}$ is determined to be 0.60 with consideration that it would be sufficient to give illustration on the interaction among real soil, embankment of masonry, concrete structure and gabion.

3.2. External loading

A dense settlement is located at the upper part of the embankment. Calculation for distributed load was based on the guideline of design load for earthquake resistance house and building in Indonesia (Ministry of Public Works, 1987). Distributed load is 10 kN/m² and 20 kN/m² for one storey and two storey building, respectively.
3.3. Seismic coefficient

Seismic coefficient was calculated based on Indonesian Earthquake Zoning Map in SNI 1726-2002 modified in 2010. From the zoning map (Figure 4), the study area is located in the seismic zone 3. The average value for Standard Penetration Test (SPT) up to 30 m depth was less than 15, meaning that the study area in Cikapundung may be classified as soft soil (Table 1). Based on Table 2, peak ground acceleration of 0.30g was obtained.

Table 1. Soil classification based on SNI-1726-2002

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Average shear wave velocity, $v_s$ (m/s)</th>
<th>Average Standard Penetration Test, N</th>
<th>Average undrained shear strength, $S_u$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hard soil</td>
<td>$v_s \geq 350$</td>
<td>$N \geq 50$</td>
<td>$S_u \geq 100$</td>
</tr>
<tr>
<td>Medium Soil</td>
<td>$175 \leq v_s &lt; 350$</td>
<td>$15 \leq N &lt; 50$</td>
<td>$50 \leq S_u &lt; 100$</td>
</tr>
<tr>
<td>Soft Soil</td>
<td>$v_s &lt; 175$</td>
<td>$N &lt; 15$</td>
<td>$S_u &lt; 50$</td>
</tr>
<tr>
<td>Special Soil</td>
<td>Required special examination on every site</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4. Stability analysis of existing riverbank

The condition of existing slope of Cikapundung was analyzed using finite element method with plane strain idealization. Figure 5 shows the geometry of the existing slope in the northern and southern part of the riverbank segment. The model was analyzed with several conditions, namely groundwater level variations, effect of one storey and two storey building load, and seismic load. In general, the modelling phase is divided into two, i.e. phase with groundwater level variations and phase with loading variations. In each phase, two calculation types were used, namely plastic and phi-c reduction. Plastic is used to calculate the occurring displacement, while phi-c reduction is used to obtain safety factor.

Figure 4. Indonesian Earthquake Zoning Map (SNI 1726-2002, modified in 2010)
The groundwater level modelled in the calculation is stated as several phases. At the same groundwater level, calculation phase with two types of loading, which are one storey building and two storey building is used. The groundwater level was modelled from the depth of -1 m to -7 m from the surface with interval of 1.0 m.

River water level in groundwater level variations is average river water level in rainy season, which is 0.7 m from the river bed. In addition, simulation of seismic load was also performed. Therefore, in each geometrical model 28 simulation phases with different conditions were performed.

The distribution of plastic points at the model is shown in Figure 6. Plastic points are locations which show shear stress occurring in a model which has reached plastic condition. The white dots or tension cut-off points, illustrate tensile condition of soil element due to active lateral earth pressure to the soil behind the slope, whereas the red dots are Mohr-Coulomb points which illustrate shear stress occurring to the model which has reached plastic condition and is in line with Mohr-Coulomb failure criteria. The position of the dots shows that the locations experiencing tensile stress. The denser the strain dots the higher the possibility of a crack to occur. To prevent the crack from occurring, further analysis on riverbank revetment, either with concrete or geotextile should be conducted.

From the result of the analysis, tension cut-off points at the upper part of the slope and masonry structure in almost all simulation phases can be seen. It shows that in every condition the area experiences tensile and has a great potential of cracks. This condition is in line with the field condition, in which a horizontal crack of 5 cm wide in the houses located at the upper part of the river slope and a 1-5 cm crack at northern part of the masonry revetment occurred.

From the simulation result for fixed load condition, the safety factor of the slope varies from 0.90 to 1.30 in the northern part and from 0.84 to 1.11 in the southern part of the slope. Meanwhile, from the simulation with seismic load, smaller safety factor of 0.65 to 1.03 in the northern part of the slope and 0.63 to 0.85 in the southern part of the slope was obtained. Figure 7 and 8 show the correlation between safety factor and groundwater level of all simulation phases in the riverbank existing model.

Other than safety factor, deformation occurring in both embankment models was assessed. Large deformation occurred in the models, far exceeding the safety standard (10 mm) as shown in Figure 7 and 8. In the simulation with fixed load, deformations in the northern part of the slope vary from 40 to 85 mm and in the southern part of slope vary from 61 to 111 mm. Due to seismic load, deformation of the northern part of the slope rises to 115 – 163 mm and of the southern part of the slope rises to 130 – 211 mm.

Based on the safety factor and deformation obtained from the simulation of the existing riverbank, it was found that the existing condition is not safe. When the groundwater level behind masonry revetment rises, the slope is more unstable. The same phenomenon happens for the loading factor on the embankment, when the load on the slope is bigger, then deformation will be bigger and the slope is more unstable. The existence of external loads also worsens the condition as it would reduce the safety factor to 75% from initial condition and it will increase the deformation twice from the initial one.

The unsafe condition of the riverbank will pose a threat to its surrounding. Therefore, a new riverbank protection in term of retaining wall is proposed. The recommended retaining wall structures are cantilever wall and gabion.
5. Proposed Mitigation Works

As the mitigation and prevention measure, the design of retaining wall was performed to reduce the landslide risk and increase the safety factor. The constructions used are gabion or cantilever wall. Retaining wall structure shown in Figure 5 is already safe in term of sliding stability, overturning stability, soil bearing capacity stability and control on weight creep ratio. The retaining wall dimension is already controlled for dynamic load (earthquake) by using Seed-Whitman (1970), Day (2002), and Mononobe-Okabe method.

Seed and Whitman (1970) developed an equation which can be used to apply pseudostatic horizontal force valid for walls as follow:

\[
P_x = \frac{3}{8} \left( \frac{a_{\text{max}}}{g} \right) h^2 \gamma
\]  

(1)

where, \( P_x \) is pseudostatic earthquake force (kN), \( a_{\text{max}} \) is maximum pseudostatic acceleration (m/s²), \( g \) is acceleration of gravity (m/s²), \( h \) is height of the wall (m), and \( \gamma \) is unit weight (kN/m³).
Empirical approach of Day (2002) is the most common method used in the analysis of retaining wall with seismic load. The empirical approach can be written as follows:

\[ P_E = 0.5 \sqrt{K_a} \left( \frac{a_{max}}{g} \right) h^2 \gamma \]  

(2)

where \( K_a \) is coefficient of active lateral earth pressure.

Mononobe-Okabe also developed an equation which can be used to determine pseudostatic horizontal force valid for retaining wall. This method is direct development from Coulomb static theory to pseudostatic condition. In this analysis, pseudostatic acceleration is acting on active and passive wedge of Coulomb:

\[ P_{AS} = 0.5 K_{AS} H^2 \gamma (1 - k_v) \]  

(3)

where, \( P_{AS} \) is seismic force (kN).

Based on the result of retaining wall dimension control to seismic load by using the above methods, it is found that the retaining wall is in a safe condition. The smallest value of safety factor to overturning stability is obtained from Mononobe-Okabe method while the smallest safety factor to sliding stability is obtained from Day method.

In the next phase, a simulation is performed to analyze the stability of retaining wall designed in the previous phase. The analysis was conducted by modeling proposed structure of retaining wall with finite element method to Plaxis version 8.6.

At the retaining wall structure, houses located at 10 m from the slope edge are recommended to be relocated and flattening the unstable slope is also recommended. The simulation is similar to the simulation at the existing riverbank slope, which varies in groundwater depth, working load and seismic load.

Figure 9 shows geometrical cross-section of the proposed retaining walls. The simulation stages performed in the retaining wall model is similar to the stages performed in the existing riverbank model.

In Figure 10, distribution of soil deformation as the result of retaining wall simulation is shown. The occurring deformation is caused by retaining wall construction followed by flattening of the slope and houses relocation to 10 m from the slope edge. In all retaining wall simulation phases, dominant soil deformation area occurred in the retaining wall and the toe part of the upper slope. The area below the slope experienced the biggest compression and settlement due to the external loads.

Figure 11 and 12 show the simulation result on the deformation and change of safety factor due to groundwater fluctuation. The maximum deformation occurring in the model is 74 mm for gabion and 93 mm for cantilever wall. The value of soil deformation increases in line with the increase in groundwater level and external load. Simulation on seismic load increased the deformation up to 200%.

Safety factor of retaining wall structure increases to 140% from the safety factor in the existing condition. In fixed load condition, the proposed retaining wall is still in relatively safe condition with safety factor between 1.00 - 1.58. However, when earthquake simulation was performed, the safety factor decreased to 0.91 - 1.41. In Figure 11 and 12, it can be seen that even if the existing masonry structure was replaced with the proposed retaining wall, the slope is flattened and the housing is relocated, the safety factor and deformation still do not meet the standard requirement on some phases, especially when the groundwater level is high. When the groundwater level is low, the occurring safety factor and deformation have met the safety standard and the slope is safe from landslide potential. Therefore, although the designed retaining wall has already been applied, horizontal drainage installed on the retaining wall is needed. If the drainage does not work optimally, the retaining wall can only increase safety factor but the landslide potential is still high.
6. Conclusions

From the field survey in Code River, a crack in the embankment, land subsidence, movement in the gabion in the downslope, deformation in the slope and malfunction of embankment drainage were found. In the river, erosion in the upper part of the river, and reduction of river bed perimeter occurred. In the housing area located on the upper part of the embankment, a crack 1-5 cm wide was also occurred. The riverbank slope was not in a stable condition, with safety factor value of less than 1.5 and deformation of more than 10 mm. Factors affecting the slope stability are external loads, groundwater depth, and seismic load. In all simulation phases by using finite element model, the deformation area is mostly in the masonry surrounding area and masonry structure. The masonry area experienced the highest compression and deformation due to great pressure in the slope crest. As mitigation measures, the retaining walls in term of cantilever wall and gabion are proposed as they are safe from overturning, sliding, bearing capacity failure, piping, and failure due to seismic force. Retaining wall construction modelling with slope flattening shows an increase in safety factor value and a decrease in deformation. However, in a high groundwater level, the
safety factor does not meet the safety standard, and therefore, a horizontal drainage to dissipate the water from the embankment body is necessary. The drainage must work effectively and optimally to reduce groundwater level so that the safety factor increases and landslide potential is reduced.

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Certificate of Participation

This is to certificate that

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