Assessment of Ground Movement at Cigombong Bridge by Using Finite Element Analysis: A Case Study in West Java, Indonesia

A Arafianto and P P Rahardjo

Department of Civil Engineering, Parahyangan Catholic University - Indonesia Corresponding author: arafianto@unpar.ac.id

Abstract. Ground movement phenomena were detected at abutment A2 of Cigombong Bridge, causing the abutment to hit a girder. A thorough investigation has been performed by inspecting the structure condition and nearby landslide occurrences directly on site. Additional borings with Standard Penetration Test (SPT) were also conducted to evaluate the actual soil condition in the abutment area. Moreover, monitoring of subsurface movement was performed regularly with inclinometers. Such additional borings and inclinometers were located at the road elevation beside the wing wall and the existing ground surface near the abutment footing. During the installation of inclinometer pipes, soil identification and SPT were conducted. Consequently, the soil condition and SPT N-value profiles can be obtained. It was established that the original soil condition is very soft clay with an SPT N-value of 1 to 4 blows/30 cm. Furthermore, the inclinometer monitoring results showed that lateral displacements were identified in both inclinometers. Based on these findings, a 3D finite element analysis was then performed to investigate the effects of ground movement on the abutment and the foundations. The results showed a possibility of pile crack and overstressed pile due to the ground movement induced by compression of soft soil.

1. Introduction

In 2018, ground movement phenomena were detected at abutment A2, causing the abutment to hit a girder. A thorough investigation has been performed by inspecting the structure's condition and nearby landslide occurrences directly on site. Additional borings with Standard Penetration Test (SPT) were also conducted to evaluate the actual soil condition in the abutment area. Moreover, monitoring of subsurface movement was performed regularly with inclinometers.

The purpose of this paper is to present the importance of well-performed geotechnical instrumentations for ground movement assessment. The inclinometers give the essential information on the depth of the ground deformation as well as its value. Moreover, the piezometers also provide valuable groundwater level data to evaluate whether the ground surface is fully saturated.

Finite element analysis was then performed to understand better the mechanism of ground movement that might happen. A three-dimensional analysis is mandatory since a plane strain section cannot represent the slope geometry and original ground surface. Furthermore, a three-dimensional effect can also be taken into account in the analysis to provide a more realistic result.
2. Site Condition

Cigombong Bridge is one of the main bridges in Bogor-Ciawi-Sukabumi Toll Road (section I), located in the South of Bogor City. The area of Cigombong is specifically situated between two active volcanoes, namely Mount Salak and Mount Pangrango. Therefore, the ground surface of the bridge area is hilly. The location of the Cigombong Bridge is shown in Figure 1.

![Figure 1. Satellite view of Cigombong Bridge (Source: Google Earth)](image)

A site visit was conducted on March 23, 2018, accompanied by the contractor and the owner. Investigation of the bridge focused on the east side of the abutment A2, where the deformation occurred. Figure 2 shows a drone view of the surrounding area and the site situation at the time of the site visit.

![Figure 2. Location and situation of the investigated abutment](image)

It can be seen from the figure that the fill embankment is quite thick, which is about 12 m. On the east side of the abutment, a small river is located near the toe of the fill embankment. Also, the fill embankment height in front of the abutment is almost as high as the girder.

Assessment of ground movement was started by examining the structural condition of the abutment. The site investigation has shown that the bearing pad at the east side of the abutment has been deformed significantly and lifted. Examination of the abutment condition on the west side cannot be conducted due to the difficulty of the access road. However, the structure and the bearing pad on the west side were in good condition, based on the contractor's information. Figure 3 shows the structural condition on the east side of the abutment.
3rd International Conference on Sustainable Infrastructure
IOP Conf. Series: Earth and Environmental Science 832 (2021) 012023   doi:10.1088/1755-1315/832/1/012023

Figure 3. Structural inspection on the east side of the abutment [1]

3. Original Ground Condition
An essential data of topographic map was used better to comprehend the ground surface contour before the abutment construction. The analyzed topographic map and cross-section of the abutment is shown in Figure 4.

Figure 4. Analyzed topographic map and cross-section view of the abutment [1]

Based on the above figure, it can be concluded that the footing of the abutment on the east side is sitting on the fill area, while the west side on the cut area. The fill thickness on the east side (underneath the abutment footing) is about 5 m. This considerable fill thickness generates higher stress to the soil beneath.
Another necessary information that can be used to evaluate the ground condition is by using a local geological map. This map provides starting information regarding the soil or rock condition at the project site. Figure 5 shows the bridge located on the geological map. Based on the Indonesia local geological map of Bogor, Java, the bridge is located at a geological layer of $Q_{vpo}$, which is stands for Quaternary Volcanic of Pangrango formation. The layer consists of old deposits, lahar, and lava, which is categorized as volcanic soils.

![Figure 5](image1.png)

**Figure 5.** Site location on a local geological map

### 4. Geotechnical Instrumentation and Additional Soil Tests

Further evaluation of the ground movement phenomena was performed by installing inclinometers at the top area behind the abutment and the bottom of the abutment in the existing ground. During the installation of inclinometer pipes, soil identification and SPT were conducted. Consequently, the soil condition and SPT N-value profiles can be obtained. The location of geotechnical instrumentation and borings is shown in Figure 6.

![Figure 6](image2.png)

**Figure 6.** Layout of borings and geotechnical instrumentations [2]

Three inclinometers and piezometers were installed, two on the east side and one on the west side. The main reason behind this configuration was to examine whether the ground movement involves a deep failure mechanism. The instrumentation on BH-IN-03 is installed to check whether there is movement on the other side of the abutment. However, since the deformation was minimal, the inclinometer results on BH-IN-03 is not considered in this study.

#### 4.1. Additional boring results

Additional soil tests of boring were conducted approximately one month after the site visit in April 2018. There were only two borings performed at the road elevation behind the wing wall (BH-IN-01) and the existing ground surface near the abutment footing (BH-IN-02). Figure 7 shows the soil stratification in the abutment area.
Based on these borings, it was established that the original soil condition is identified as very soft to soft clay, with an SPT N-value of 1 to 4 blows/30 cm. The thickness of the soft soil is about 7 – 10 m in both boring locations. These results show that the abutment is sitting on the soft soil. Thus, the ground movement was strongly expected due to the soft soil layer's compression induced by backfilling.

4.2. Geotechnical instrumentation results
Monitoring of inclinometers and piezometers were performed weekly for 3 months, starting from May 4 to August 2 2018. The purpose of these regular monitoring is to give continuous progress of the subsurface deformation and an early warning system. Monitoring documentation is shown in Figure 8.
4.2.1. Inclinometer results

The inclinometer is one of the most common apparatus used to monitor subsurface movement. An inclinometer measures a casing’s inclination from vertical. Successive inclination measurements, when processed, measure lateral deformations with depth. In addition, an inclinometer can also measure not only the horizontal deformation that parallel to the slope direction but also perpendicular to the slope direction. Figure 9 shows the sketch of a standard inclinometer system.

Figure 9. Portable borehole inclinometer system [4]

In this specific site, the inclinometer results show excellent and consistent data from reading to the subsequent reading. Moreover, the slip surface location from both inclinometers can be easily noticed, as shown in Figure 10. Please note that IN-BC-01 corresponds to location BH-01 and ditto for IN-BC-02.
Figure 10. Inclinometer monitoring results [2]

The above results show that the maximum measured deformation, at A-A direction, which is parallel to the slope direction, on both locations is about 5 mm. Deformation in the B-B direction of IN_BC-01 shows a lower value than in the A-A direction, which is about 2 mm. It means that the direction of the deformation is more dominant in the A-A direction. In contrast with IN-BC-01, the deformation in both directions is more or less the same, about 5-6 mm. Both inclinometers show a distinct slip surface location; 17 m in the IN-BC-01 and 10 m in IN-BC-02.

4.2.2. Piezometer results

There are various types of piezometers available in the market, and each type has its purpose. In the study site, stand-pipe piezometers were installed to monitor the groundwater level. The results of the monitoring are shown in Figure 11.

Figure 11. Stand-pipe piezometer monitoring results [2]

The piezometer results show relatively constant groundwater during the monitoring period of May to August 2018. Groundwater level at BH-02 (represented by piezometer PZ-02) is always at zero, which means that the soil is fully saturated. An anomaly of groundwater level fluctuation was found at PZ-01 on July 14, 2018. However, the fluctuation can be neglected, and the average groundwater level can be taken as 9.5 m below the existing ground (below the abutment base).
5. Prediction of Ground Movement Mechanism

Based on the ground condition and monitoring results, it can be concluded that the ground movement was induced by compression of the soft soil due to backfilling. The thickness of the fill embankment on the east side of the abutment is as high as 14 m. This considerable load is approximately equivalent to a pressure of 22 tons/m². Also, the inclinometers confirm that the lateral movement existed and gradually increase with time. Figure 12 shows a cross-section of the abutment, complemented with additional borings and instrumentation results. A slip surface prediction is also drawn in the sketch to show the possible deep failure mechanism.

![Figure 12. Slip surface prediction based on additional borings and instrumentation results](image)

Besides the ground movement, it is also interesting to elaborate on the possible twisting of the abutment. Site findings show that the bearing pad at the east side of the abutment has been deformed significantly and lifted, contrasting with the bearing pad at the west side. This phenomenon occurred because the footing of the abutment on the east side is sitting in the fill area, while the west side is in the cut area. Thus, it can be concluded that the movement of the abutment was more significant on the east side than the west side. Figure 13 shows the possible twisting movement of the abutment.

![Figure 13. Possible movement of the abutment](image)
6. Three-dimensional Finite Element Analysis
Most slope stability analyses are performed on a two-dimensional model even though the slope's shape in the field is three-dimensional. In this particular problem of Cigombong Bridge, three-dimensional analysis is warranted since the ground surface is not level and the fill embankment on the east side of the abutment is thicker than the west side.

6.1. Analysis method, model configuration, and input parameters
Slope stability analysis in the 3D finite element model is conducted using finite element-based software, Midas GTS NX. The model consists of soil layers, groundwater conditions, structural elements (abutment and piles foundation), and soil-structure interactions. Soil layers and the abutment are modeled as 3D solid elements. The embankment's real geometry on the field can be created, including the granular backfill behind the abutment. Figure 14 shows the configuration of the model, which comprises triangular meshes.

![Finite element model configuration](image1)

Figure 14. Finite element model configuration

Based on the as-built drawing, the foundation of the abutment is driven piles with a diameter of 500 mm and a length of 14 m. A total of 4×21 piles are modeled with beam elements instead of solid elements. Beam elements were chosen to enable the calculation of forces of the pile. Additionally, to model the girder, it is decided to model it with pinned boundary conditions. The isometric and front view of the details in the abutment is shown in Figure 15.

![Structural element of abutment and piles; girder modeled as pinned boundary conditions](image2)

Figure 15. Structural element of abutment and piles; girder modeled as pinned boundary conditions
The geotechnical parameter is mostly derived from the correlation with SPT N-value and other common correlations used in geotechnical practice. A linear-elastic stress-strain behavior was adopted for modeling the structural elements such as piles and abutment. As for the soils, a Modified Mohr-Coulomb model was selected. This model is a more detailed material model than the Mohr-Coulomb model, and the elasticity modulus can be set at different values for loading and unloading. The lists of structural and soil parameters are shown in Table 1 and Table 2.

| Table 1. Structural parameters (Linear-Elastic Model) |
|-----------------------------------------------|
| Name | E (kN/m²) | ν | γ (kN/m³) |
|------|-----------|---|-----------|
| Concrete f’, 52 MPa, Driven Pile | 3.40E+07 | 0.3 | 26 |
| Concrete f’, 30 MPa, Abutment | 2.57E+07 | 0.3 | 25 |

| Table 2. Soil Parameters (Modified Mohr-Coulomb Model) |
|-----------------------------------------------|
| Name | ν | γ (kN/m³) | γsat (kN/m³) | Es0 ref (kN/m³) | Eav ref (kN/m³) | Enur ref (kN/m³) | c (kN/m²) | φ (°) | K0,NC | m | Ry |
|------|---|---------|-------------|----------------|----------------|----------------|------|------|-------|---|---|
| Granular Backfill | 0.3 | 21 | 22 | 90000 | 100000 | 50000 | 1 | 50 | 0.23 | 0.5 | 0.9 |
| 1. Soft Clay (N=3) | 0.35 | 15 | 16 | 3800 | 19000 | 2100 | 3 | 30 | 0.50 | 1 | 0.9 |
| 2. Clayey Silt (N=15) | 0.3 | 17 | 18 | 19000 | 57000 | 10500 | 15 | 31 | 0.48 | 0.5 | 0.9 |
| 3. Clayey Silt (N=30) | 0.3 | 18 | 19 | 38000 | 114000 | 21000 | 30 | 33 | 0.46 | 0.5 | 0.9 |
| Embankment (N=8) | 0.3 | 18 | 19 | 10180 | 30500 | 5600 | 12 | 25 | 0.58 | 0.5 | 0.9 |

In Midas GTS NX, soil response due to axial and lateral loading can be considered by inputting the values of ultimate shear force, shear stiffness modulus, and normal stiffness modulus in an interface parameter. The ultimate shear force corresponds to the maximum shear strength of the soil (τmax) as the same as in t-z curves, and shear stiffness modulus (Kt) corresponds to the gradient of the t-z curve. The normal stiffness modulus is the same concept as the lateral subgrade reaction modulus, calculated from the general p-y analysis. Table 3 summarizes the parameters for the interface element.

| Table 3. Pile interface parameters |
|-----------------------------------------------|
| Name | Ultimate Shear Force (kN/m²) | Shear Stiffness Modulus, Kt (kN/m³) | Normal Stiffness Modulus, Kn (kN/m³) |
|------|-------------------------------|-------------------------------------|-------------------------------------|
| Pile-Soil Interface | 70 | 7000 | 1600 |
| Pile-Raft Interface | 2.15E+09 | 1.00E+08 | 1.00E+09 |

6.2. Construction stage and analysis type
In the finite element analysis, it is possible to simulate the construction stage in the field. The staged construction defined in the calculation is as follows:

- Initial condition; that is the calculation of the initial stress in the soil mass
- Construction of working platform and pile installation;
- Abutment construction;
- Backfilling and girder installation (by activating pinned boundary conditions on the abutment)

The analysis type adopted in this study is stress analysis, and only long term condition is considered. Thus, the material type is drained, and all parameters are expressed in effective stress parameters.
6.3. Analysis results
The first output commonly checked in the stress analysis, especially in the loading problem, is the displacement contour. Based on the calculation, the predicted long-term settlement of the embankment is about 90 cm. This value is relatively close to the basic settlement calculation with the coefficient of volume compressibility ($m_v$) parameter. The maximum settlement is located in the south of the abutment, in which embankment at the particular area is 12.5 – 13 m. Figure 16 shows the contour of the total displacement of the fill embankment.

![Figure 16. Total displacement of fill embankment](image)

Displacement of abutment and piles is the next item to be checked. Since the top of the abutment is constrained with the pinned boundary conditions, it can be seen that the displacement in the area is zero, and the abutment footing is displaced. As expected, the displacement on the east side of the abutment is more significant than the west side. The maximum displacement on the piles and abutment are 5.5 cm and 4.4 cm, respectively. The deformed mesh view of the pile and abutment is shown in Figure 17.

![Figure 17. Total displacement of abutment and piles (deformed mesh view)](image)
Besides checking the displacement, it is also mandatory to check the pile forces. If the long-term settlement is achieved, the bending moment in the piles could exceed its capacity. The pile maximum bending moment can be as high as 305 kN-m, while the typical crack moment capacity of the 500-mm diameter driven pile is about 120 – 140 kN-m. Figure 18 shows the bending moment ($M_y$) profiles in each pile. It can be seen that the bending moment of piles on the west side of the abutment is minimal compared to those on the east side.

Figure 18. Bending moment in the y-direction ($M_y$) on piles

Since the fill embankment is considerably high on the east side, it is expected that the induced stress both in the vertical and lateral directions is high. Hence, it is necessary to check the piles’ axial force, whether the piles are overstressed. Figure 19 shows the axial force profiles in each pile. It can be seen that the maximum axial force on the pile is as high as 1085 kN. If the pile bearing capacity is calculated, the ultimate axial capacity is approximately 970 kN. This result shows that there is a possibility of the pile being overstressed in the field.

Figure 19. Axial force on piles
7. Conclusions Summary
Based on the interpretation of monitoring results and three-dimensional finite element analysis, it can be concluded that on-site investigation and topographic map analysis provide essential information on the possible ground movement mechanism at abutment 2 of Cigombong Bridge and additional soil tests and well-performed geotechnical instrumentations have been proven to be successful in showing that the ground movement was due to the compression of the soft soil layer induced by backfilling. In addition, three-dimensional finite element analysis is warranted for the particular problem of Cigombong Bridge because the ground surface is not level, and the fill embankment is thicker on the one side. The analysis results show a possibility of pile crack and overstressed pile due to the ground movement.

8. References
[1] PT. Geotechnical Engineering Consultant 2018 Final Report: Stability Evaluation of Cigombong Bridge (in Indonesian) (Bandung: PT. Geotechnical Engineering Consultant)
[2] PT. Geotechnical Engineering Consultant 2018 Geotechnical Instrumentation Inclinometer and Piezometer Monitoring Report (in Indonesian) (Bandung: PT. Geotechnical Engineering Consultant)
[3] PT. Testana Indoteknika 2018 Soil Investigation Report (in Indonesian) (Jakarta: PT. Testana Indoteknika)
[4] Abramson L W, Lee T S, Sharma S and Boyce G M 2002 Slope Stability and Stabilization Methods (New York: John Wiley & Sons, Inc.) p 615

Acknowledgments
The authors are grateful to the director and staff of PT. Geotechnical Engineering Consultant for sharing and allowing the project data to be used in this study. The authors are also thankful to PT. Midasindo Teknik Utama for the donation of Midas GTS NX license to Universitas Katolik Parahyangan to perform this research successfully.