Analysis of rock mass behavior with Empirical and Numerical method for the construction of diversion tunnel, Laos

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Abstract. Tunneling construction in the mountain area is a challenge for engineers and geotechnicians because of instability due to the presence of discontinuities. The objective of this paper is the modeling of surrounding rock masses for the stability of the diversion tunnel to predict the behavior of rock masses during the excavation process for the Nam Phoun hydropower station project in Laos. Field investigation and laboratories test was realized; Empirical methods as Rock mass designation and Geological Strength Index were performed, rock masses were classified in three categories (RM-1, RM-2, and RM-3); in situ stresses were obtained from existing equations, numerical modeling was performed by the 2D plane strain finite element code Phase2 developed by Rocscience, using Generalized Hoek-Brown criterion for each type of rock masses. The results of numerical modeling show the strength zones of stresses and deformations around the tunnel and predict the instabilities around the tunnel during excavations processes. Thus, for all rock’s masses, it will be necessary to consider an analysis for the supports design before the excavation’s process. The findings of this study allow a clearer understanding of the importance to assess a predictive analysis of slope stability during the feasibility phase of a project by engineers to have an idea of instabilities and its significant in preventing the impact on the cost of the project.

1. Introduction

The construction of structures in mountain environments has always been a major challenge for engineers and geotechnicians. In most cases, the excavated ground presents risks of failure related to the behavior of the rock masses due to their heterogeneity and anisotropy. In the case of the construction of deep or shallow tunnels, road, a dam in a massif with discontinuities, great attention should be paid to the stress redistribution, strain, displacement and behavior of the rock mass, which is control by the number of discontinuities set and their characteristics (orientation, appearance, and spacing), the characteristics of rock’s matrix (e.g. resistance, deformability, weathered) and groundwater condition; In most of the case, the detail on these parameters are absent [1]. But, taking into account these factors during the design of the structure by engineers is very important; This makes it possible to evaluate the risks, the avoid any impact on the investment cost of the project. However, with the progress in the rock engineering area, some empirical methods and numerical methods have been developed by many researchers. The empirical methods allow obtaining the data on engineering properties of rock mass such as shear strength properties, in situ stress, deformation modulus, the general state of discontinuities, hydrological condition of the rock mass, the quality of rock mass and others parameters controlling the behavior of rock mass. But in the case of tunneling designing, empirical methods present some limitations in the evaluation of the performance of support systems, stress distribution and the deformation around the tunnel [2]; In the aim to solve this problem, the numerical method is used by engineers. These methods have many
advantages in the evaluation of rock mass behavior and it provides quick modeling of the studied problem and the lowest financial cost.

The purpose of this study is the modeling of surrounding rock masses for the stability of diversion tunnel in the aim to predict the behavior of rock masses during the excavation process, based on the empirical methods and numerical method for the Nam Phoun hydropower station project in Laos. In this paper, RQD proposed by [3] and GSI were used to classify the rock masses and the finite element method was used for modeling the behavior of rock masses code called phase 2. v.8.0 developed by Rocscience [4]. The project is under a feasibility study, and the diversion tunnel will be constructed on the right bank slope. To accomplish the task, field investigation, measurements, and laboratory test were performed on the different samples collected from the dam site to determine the engineering properties of rock masses.

2. Geology of the study area
The dam site in this study is located at Nam Phoun River near the Nam Phoun Village in Paklay District of Xayaburi Province in Northwestern Laos, approximately 2.6km upstream of the convergence points of the tributary Nam Gnam River. The latitude and longitude coordinates of the dam project location are 18°27’48.23”N and 101°28’04.45”E (Figure 1).

Figure 1. Location of the study area [5]

Diversion tunnel should be constructed at the right bank and is a total of 635.7 m long; inlet floor elevation is 243m and outlet floor elevation 242m; the cross-section is portal shaped, 6.0 m wide and approximately 7.0 high. Field investigation performed in the study area showed that the geology of bedrock in the dam area is mostly comprised of tuff, tuffaceous sandstone and tuffaceous slate (CPz2) from Upper Devonian to Lower Carboniferous. The dam site area has the major presence of Quaternary overburden which is mostly composed of alluvial and diluvial-eluvial deposits. The upper part of
overburden on both banks is composed of diluvial-eluvial gravelly clayey soil and lower part eluvial gravelly soil. The upper part is composed of deluvial-eluvial gravelly clay and lower part eluvial gravelly soil. The bedrock at diversion channel is lithologically composed of tuff, tuffaceous sandstone, and tuffaceous slate.

3. Rock mass classification

In the literature, there are various rock mass classification system developed in the field of rock, civil and mining engineering by many researchers such as Rock mass Quality Designation (RQD), Rock Mass Rating (RMR), Geological Strength Index (GSI), Tunneling quality index (Q-system) proposed by [2-6-7-8] and others, as New Australian Tunneling Method (NATM), Rock Structure Rating (RSR) for assessment and classification of rock mass; this provides some necessary input data for performing the numerical modeling of the rock mass. In this study RQD, and GSI were used because of its flexibility in terms of input parameters.

Rock Quality Designation (RQD) is defined as the ratio of the sum of the length of pieces of core longer than 10cm to a total length of the core and expressed by the following equation:

\[
RQD = \frac{\sum \text{length of core pieces > 10cm}}{\text{total length of core}} \times 100\%
\]  

(1)

Table 1 illustrate the classification of rock mass according RQD. The Geological Strength Index (GSI) is estimated using qualitative and quantitative factors depending on the petrographic nature of the rock and its degree of invoicing.

| RQD value | Rock mass quality | Distribution | Lithologies | Rock Mass |
|-----------|------------------|-------------|-------------|----------|
| 90-100    | Excellent        | -           | Tuff, tuffaceous sandstone, tuffaceous slate | RM-1     |
| 75-90     | Good             | 65%         | Tuffaceous slate, Tuffaceous phyllite | RM-2     |
| 50-75     | Fair             | 25%         | Tuffaceous slate, Tuffaceous slate, tuffaceous sandstone, tuff | RM-3     |
| 25-50     | Poor             | 10%         | -           | -        |
| <20       | Very poor        | -           | -           | -        |

Thus, this system was established to estimate the resistance of the rock mass, by reducing the resistance of the intact rock for the different geological conditions. For this system, the maximum resistance of a rock depends on the properties of the intact blocks and also depends on the freedom of the blocks to slide and rotate under a range of conditions of imposed stresses; figure 2 illustrates the chart of GSI [9].
In situ stress

In rock engineering, in situ stress is determined by direct and indirect methods. Indirect methods, in situ stress, are determined by the methods like flat jack, overcoming and under coring, and hydraulic fracturing. These methods are costly time-consuming, the procedures used in the determination of these stresses are difficult, and the result may be questionable [10-11-12]. In the indirect method, the stresses (vertical and horizontal) are determined through empirical models developed in rock engineering by the following equations:

\[ \sigma_v = \gamma \cdot H \]  

(2)

\[ \sigma_h = \left( \frac{v}{1-v} \right) \sigma_v + \beta E_{rm} G \left( \frac{H+100}{1-v} \right) \]  

(3)

Where the unit weight of rock mass and H is the overburden, \( \nu \) is Poisson's ratio, \( \beta \) is the coefficient of thermal expansion which the value for rocks is \( 8 \times 10^{-6} \) /°C, \( E_{rm} \) is the young’s modulus of intact rock in MPa, \( G(\text{C/m}) \) is the thermal gradient of rock (However, the simplified relationship of equation (2) for the determination of the horizontal stress is):

\[ \sigma_h = \left( \frac{v}{1-v} \right) \sigma_v \]  

(4)

In this study, the vertical and horizontal stresses were determined using equations (2) and (4) for each type of rock mass; the result is illustrated in the table 2.
4.1 Numerical modeling

Numerical modeling has been developed and applied in rock engineering because of its ability to better simulate actual slopes failure mechanisms [13]. Great progress has been realized in the last three decades in rock mass analysis. This allows us to classify numerical methods into the continuum and discontinuous [14-15]. In the continuum modeling, plastic softening and damage are the most techniques for capturing failure and localization [16-17]. The continuum method includes the Finite element Method (FEM), Finite Difference Method (FDM), and the Boundary Element Method (BEM). The discontinuous methods include Distinct Element Methods (DEM) and the Discontinuous Deformation Analysis (DDA). All these methods provide an approximate solution to problems. However, the finite element method (FEM) is one of the most used in the field of landslide and slope stability analysis; it provides more advantages such as:

a. the stress-strain relation of soil or rock is considered and thus more accurate mechanical behavior can be computed, such as non-linear deformation and the influence of water and earthquakes;
b. No assumptions are applied in advance related to the interslice forces and their directions, or the shape or location of the slip surface. The critical slip surface is determined automatically, and the slope fails naturally;
c. Complex slope geometries can be addressed, and parametric studies can be conducted [18-19].

In this study, to predict the behavior of surrounding rock masses around the diversion tunnel during the excavation process, a two-dimensional hybrid finite element model code Phase2 [4] is used. The modeling concerns each type of rock mass identified around the tunnel site project.

| Rock mass | Unit weight (MPa) | Modul of elasticity (MPa) | Poisson’s ratio (ν) | Generalized Hoek -Brown parameters | Vertical stress (MPa) | Horizontal stress (MPa) |
|-----------|------------------|--------------------------|------------------|-----------------------------------|---------------------|------------------------|
| RM-1      | 27.45            | 10000                    | 0.28             | 2.60 0.0067 0.5040 13 55 0.21 0.816 |
| RM-2      | 22.5             | 40000                    | 0.34             | 0.821 0.0012 0.514 7 40 0.2 0.10 |
| RM-3      | 19.61            | 40000                    | 0.40             | 0.402 0.00013 0.5437 7 20 0.1568 0.104 |

5. Results and Discussion

Rock masses modeling was carried out for the construction of the diversion tunnel of the Nam Phoun hydropower station project site in this paper. The purpose of this study was to perform the modeling of surrounding rock masses for the stability of the diversion tunnel in the aim to predict the behavior of rock masses during the excavation process. Field investigation, measurements, and laboratory tests were conducted in the study area according to IRSM methods [20]. RQD and GSI as an empirical method were used.

According to RQD classification, the surrounding rock mass around the diversion tunnel site is the following distribution: 65% fair, 25% poor and 10% very poor (Table 1). The GSI was used as an input parameter for numerical modeling and the result is illustrated in Table 2.

The Finite element method was performed to model the rock masses using a commercial finite element code Phase2 developed by Rocscience. The input parameters for the numerical modeling include uniaxial compressive strength, stresses (vertical and horizontal), cohesion, deformation modulus, Poisson’s ratio, modulus elastic of intact rock, unit weight (Table 2). Each category of rock mass was modeling according
to elastoplastic behavior using Generalized Hoek-Brown criterion and the tunnel modeling is 2D considering plane strain problem. It’s important to notify that, the numerical modeling was selected for a sequential excavation, and the field stress using in the model was gravity loading for all rocks mass.

5.1. Numerical modeling for RM-1

The result of the Finite element modeling of the Rock mass-1 is illustrated in figure 3. Before excavation, the virgin sigma 1 value was 0.21 MPa. After excavation, the maximum sigma 1 value is 0.16 MPa at the crown, 0.96 MPa at sidewalls and the maximum value is 1.6 MPa at the corner sides (Figure 3a). The value of sigma 3 was 0.081 MPa before excavation; However, after excavation, the value of sigma 3 values is respectively 0.01 MPa at the crown, the base, and the sidewalls. After the comparison of the stress’s virgin sigma 1 and sigma 3 before and after excavation, this is shown the increasing virgin sigma 1 of 1.39 MPa and the decreasing of virgin sigma 3 of 0.071 MPa. It’s important to notify that for this section, the maximum stress concentration is developed at the corner sides of the tunnel (Figure 3b).

The maximum total displacement which constitutes the maximum deformation is 7.65e-004 m, situated at the base of the tunnel after excavation model (Figure 3c); Although the displacements are not so important around the tunnel the thickness of plastic zone is important at the base (yields zone is 100%) as shown in the Figure 3d. So, it's important for safety during excavations operations for this tunnel to prevent an analysis concerning the design of the support system.

5.2. Numerical modeling for RM-2

In this section, the result of finite element modeling of surrounding rock mass in the tunnel is illustrated in figure 4. Before excavation, the virgin stress sigma 1 was 0.2 MPa. After excavation, the virgin stress sigma 1 is respectively 1.80 MPa at the crown, 0.30 MPa at the sidewalls, and 0.60 MPa at the base of the tunnel (Figure 3a). The virgin stress sigma 3 before excavation was 0.1 MPa; However, after excavation, the virgin stress sigma 3 is 0.23 MPa at the crown, 0.04 MPa respectively at the sidewalls and the base of the tunnel (Figure 3b). The difference between the maximum virgin stress sigma 1 before and after the excavation is 1.6 MPa, and for virgin sigma 3 is 0.03 MPa. These results showed the increasing of stresses sigma 1 and sigma 3 after excavation. Although, for this section, the maximum concentrations for stresses are at the crown and the base of the tunnel.

The maximum deformation is 1.80e-004 m, concentrated at the sidewalls and the crown (Figure 3c). This is can explain the maximum plastic zones at the crown (yields zone 80%) and the base of the tunnel (yields zone 100%) according to the figure 3d; But, for more safety, it will be necessary to establish the support accord to the quality of rock mass.

![Figure 3. RM-2; (a) Sigma 1 contours; (b) Sigma 3; (c) Maximum displacement; (d) yielded elements.](image-url)
The finite element modeling of rock mass 3 is represented in figure 4. The analysis of the virgin stress \( \sigma_1 \) and \( \sigma_3 \) before excavation was respectively 0.15 MPa and 0.10 MPa. After excavation, the virgin stress \( \sigma_1 \) is 0.36 MPa at the crown of the tunnel, and 0.12 MPa at the sidewalls (Figure 4a). The virgin stress \( \sigma_3 \) is 0.4 MPa at the crown, the sidewalls, and the base of the tunnel (Figure 4b). According to the results, there is an increase of stresses, respectively 0.21 MPa for \( \sigma_1 \) and 0.3 MPa for \( \sigma_3 \) after excavation around the tunnel.

![Figure 4. RM-3; (a) Sigma 1 contours; (b) Sigma 3; (c) Maximum displacement; (d) yielded elements.](image)

According to figure 4c, the maximum displacement is 1.71e-004 m in this section. The maximum vectors deformation is focused around the tunnel; This is can be explained by the strength thickness of the yield’s zones (at 100%) around the tunnel (Figure 4d). According to the results of yields elements analysis, the support design will be necessary for the rock mass during the construction processes.

6. Conclusion

In this paper, the empirical and numerical methods were used to modeling the behavior of rock masses in the aim to predict the stability of the tunnel during the excavation operations. Stresses redistribution and deformations around tunnel were analyzed before and after excavation for each type of rock masses. Finite element analysis code Phase\(^2\) developed by Rocscience was used for modeling of surrounding rocks around the tunnel, the generalized Hoek-Brown criterion was used for the models. Based on the empirical and numerical modeling analyses performed for the construction of the diversion tunnel, the following conclusions can be drawn: a) For the RM-1, the stresses analysis showed the increase of the virgin \( \sigma_1 \) of 1.39 MPa and the reduced of the virgin \( \sigma_3 \) of 0.071 MPa after excavation in the modeling; the total displacement was observed as 7.65e-004 m where the yield zone was observed as 100% more focused at the base of the tunnel model. b) For the RM - 2, the stresses analysis performed showed the increase of the virgin \( \sigma_1 \) of 1.6 MPa and \( \sigma_3 \) of 0.03 MPa after excavation, the maximum deformation was observed as 1.80e-004 m, at the sidewalls and the crown of the tunnel model where the thickness of yielded elements were respectively 80% and 100%. c) For the RM- 3, the stress analysis performed showed the increased value of the virgin \( \sigma_1 \) of 0.21 MPa and 0.3 MPa for \( \sigma_3 \) after excavation around the tunnel model. The maximum deformation was observed as 1.71e-004 m and the large thickness yielded zones as 100% around the tunnel; and for all Rock masses in this case of study, it will be necessary to perform the analysis concerning the support design before and during the construction of the diversion tunnel for more safety. The findings study allowed a clearer understanding
of the importance to assess a predictive analysis of slope stability during the feasibility phase of a project by engineers to have an idea of instabilities and its significant in preventing the impact on the cost of the project and the protection of human lives.

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