Stability Analysis of Shallow Depth Tunnel in Weak Rock Mass: 3D Numerical Modeling Approach

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Abstract The paper aims at understanding the behavior around a shallow depth tunnel in weak rock mass. Practically such behavior is difficult to completely follow or even estimate in field. To solve the issue, the paper applied modeling using Abaqus/CAE software. In this software, finite element method was used to build a 3D model for the tunnel and its surrounding rock mass. The model was analyzed for stresses and their corresponding strains distribution in addition to displacement vectors before and after tunnel excavation. Results indicate that stresses and strains around the studied tunnel are redistributed due to tunnel excavation. The principal stresses are concentrated on tunnel floor, while shear stress at the tunnel corners. The displacement severely affect tunnel roof. It has also been found that the most affected zone is central floor portion of tunnel at a damage number of 0.75.

Keywords: shallow tunnel, stress distribution, strain distribution, failure, damage number


1. Introduction

Shallow tunnels have numerous applications in urban areas. They are used for subsurface structures, public utilities, in addition to human transportation equipment. Recently, there is a significant increase in shallow tunnel construction worldwide due to rapid growth of human needs for transportation, intensive energy and infrastructure development besides the high cost and unavailability of land space in metropolitan cities [1]. It is a major issue if these shallow tunnels are to be excavated in heavily jointed weak rock mass [2,3,4]. The stability of such tunnels is a challenging task. For well understanding of these tunnels stability, various studies were performed before and after excavation of rock mass, including geological [5,6] geophysical [7], geotechnical [8] applying the suitable technique for each studied parameters. Recent studies have considered numerical modelling as a technique for assessment of tunnels stability [4,9,10]. The technique was adopted due to its relatively low cost, fastness and ability to deal with different categories of the surrounding rock mass properties [11]. The numerical modelling technique also has the ability to show different responses such as stresses, strains and displacements. The shallow tunnel in weak rock mass induces both lateral and vertical disturbances, which may cause serious damages for both surface and sub surfaces structure [12,13,14]. Prediction of stability of these tunnels is very important for safety reasons especially they are usually surrounded by weak rock mass. Therefore, this paper is mainly dealing with numerical assessment of shallow tunnel in weak rock mass.

2. Tunnel Description and Its Modelling

Abaqus/CAE software has been used for this study. The tunnel 3D model with half portion has been utilized for analysis assuming symmetry around its vertical axes [9,10]. Figure 1 shows 3 D cross sectional view of a segment from the under study tunnel. The segment dimensions are assumed 50*50*50 m which represents more than three times of the tunnel dimensions. The segment dimensions were thus assumed in the light of previous finding of Verma and his co-workers where they found that at that ratio of segment to tunnel dimensions, the effect of tunnel excavation usually is usually null [15]. It is clear that the tunnel is of shallow depth (17 m depth) with horseshoe shape. The horseshoe shape was selected in this study due to its practical uses for shallow engineering constructions [16]. Moreover, technically, the stress distribution around horseshoe shape tunnel is better as compared to that of other shapes [10].

The model main components included: tunnel surrounding material properties and their expected constitutive behavior, initial boundary and loading conditions, in addition to mesh generation [9,10,17,18,19].

Table 1 shows the considered jointed weak rock mass properties. In the model, it has been assumed that an elasto-plastic behavior is expected. Generalized Hoek and Brown criteria is used for material model and Mohr-Coulomb failure criterion is for failure determination. The
The equivalent material condition is shown in Table 2. The Generalized Hoek and Brown Failure Criterion for rock mass is expressed as

$$\sigma'_1 = \sigma'_3 + \sigma'_{ci} \left( m_b \left( \frac{\sigma'_3}{\sigma'_{ci}} \right) + s \right)$$

(1)

Where,

- $\sigma'_1$ & $\sigma'_2$ = major and minor effective principal stress,
- $\sigma'_{ci}$ = uniaxial compressive strength of intact rock, and
- $m_b$ = reduced value of the material constant $m_i$.

And $m_b$ is given by the following equation

$$m_b = m_i \exp \frac{GSI - 100}{28 - 14D}$$

(2)

Where,

- $m_i$ = Hoek-Brown rock material constant to be found from triaxial tests on rock cores.

In Equation (5) and (6), $s$ and $a$ are constants for the rock mass given by the following relationships.

$$s = \exp \frac{GSI - 100}{9 - 3D}$$

(3)

$$a = \frac{1}{2} + \frac{1}{6} \left( e^{-GSI/15} - e^{-20/3} \right)$$

(4)

Where, $D$ = disturbance factor which depend on the rock mass has been subjected by blast damage and stress relaxation and it varies from 0 to 1.

In the model, meshing is developed from nodes and elements and each element and node behaves according to a prescribed linear or nonlinear stress/strain response according to applied forces or boundary condition [19]. The geometry of the model is discretized into 10,000 elements (Figure 2). The stresses, strains, and displacements have observed and analyzed before and after tunnel excavation.

### Table 1. Properties of the jointed weak rock mass surrounding the understudy tunnel

<table>
<thead>
<tr>
<th>Rock parameters</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>2200</td>
<td>Kg·m$^{-3}$</td>
</tr>
<tr>
<td>Young's Modulus</td>
<td>20</td>
<td>GPa</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>Uniaxial Compressive strength</td>
<td>28</td>
<td>MPa</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>1.5</td>
<td>MPa</td>
</tr>
<tr>
<td>Cohesion</td>
<td>3</td>
<td>MPa</td>
</tr>
<tr>
<td>Angle of internal friction</td>
<td>27</td>
<td>Degree</td>
</tr>
</tbody>
</table>

### Table 2. Condition for equivalent material model for Hoek and Brown criteria

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>GSI</td>
<td>55-60</td>
</tr>
<tr>
<td>$m_i$</td>
<td>12-15</td>
</tr>
<tr>
<td>D</td>
<td>0</td>
</tr>
</tbody>
</table>

### Figure 2. 3D model showing the segment of the tunnel area discretised into mesh elements (10000 elements)

### 3. Results and Discussions

#### 3.1. Stresses Distribution Analyses

Figure 3 presents the principal stress distribution around the tunnel. It shows that principal stress ranges from -0.325 to +0.115 MPa. One can notice that the maximum tension is met in the tunnel floor (+0.115MPa). On the other hand, the shear stresses distribution around the tunnel is shown in Figure 4. It depicts that shear stress varies from -0.248 to +0.281 MPa. The high shear stresses are concentrated in the tunnel side corners. Generally, this types of condition occurs when the overburden is so less and the material is medium to weak condition.
The hitherto discussed stresses (either principal or shear) represent the after tunnel excavation case. However, it is of importance to compare these stresses with their original ranges before tunnel excavation especially at areas corresponding to tunnel floor, roof and its corners. This comparison is shown in Figure 5. It indicates that all stresses before excavation were compression. After excavation, stresses at the tunnel corner are still compression but with smaller values. However, stresses at tunnel roof and its floor have been changed to become tension. Higher tension stress encountered in tunnel floor (+0.115 MPa) as before excavation but under high compression. The floor stress after tunnel excavation has been changed 166 % compared to before tunnel excavation. These findings agree with previously published results regarding behaviour of weak rock mass tunnel. In 1993, Ducan reported that stress redistribution around tunnels excavated in weak rock masses may lead to failure of the tunnel if it is not well supported [20]. In another investigation, Hoek (1998) showed that failure of a horseshoe shape tunnel in a highly stressed poor quality rock mass was initiated at the tunnel floor, its corners and extended to the sidewall.

3.2. Strain Distribution Analyses

Figure 6 shows the principal strain distribution around the tunnel. It shows that principal strain ranges from -8*10^{-10} to +1.57*10^{-5}. These strains are met in the tunnel floor, corners and its side walls. On the other hand, the shear strains distribution around the tunnel is shown in Figure 7. It depicts that shear strains varies from -3.1*10^{-5} to +3.51*10^{-5}. The high shear strains are concentrated in the tunnel bottom and top corners.
3.3. Displacement Analyses

The up to moment analyses shows stresses and strains behavior around the studied tunnel. However, still one important item needed for more understanding of the tunnel stability that is the resultant displacement vector around the tunnel. This section of the paper covers that item (Figure 8). It shows that some of the tunnel elements are subjected to low resultant displacement (tunnel floor) while some other elements can suffer from a maximum of 1.23 mm resultant displacement (tunnel roof). This is due to redistribution of stresses after excavation and the creation of free faces. In addition, the design of tunnel in isotropic material model condition and stress field is assumed almost hydrostatic. In such case, tunnel shape controls the stress and deformation condition and tunnel is planner in invert so such maximum stress and deformation is observing in invert (Panthee et al. 2016a).

Figure 8. Resultant displacement vector around the tunnel as shown from the 3D model cross section

Figure 9. Failure zone of tunnel model for the different considered damage numbers as shown by the tunnel 3D model
3.4. Failure and Damage Analyses

After analysing stresses, strains and resultant displacement around the considered tunnel, it becomes of value to understand the possible failure and damage areas in the tunnel structure. For this part of the study, the maximum principal stress theory has been considered [16,22]. According to this theory, the failure will occur when the maximum principal stress reaches the value of the strength at elastic limit of rock and tension strength is generally consider for rock failure due to weakness of rock tension compared to its principal stress (Kainthola et al. 2012; Sazid & Singh 2013; Panthee et al. 2016b). This was followed by identifying the deactivated mesh elements in the model. The failure criterion is the damage number which represents the ratio of maximum principal stress to strength of the tunnel surrounding rock mass. Thus it indicates the probability of rock mass failure [10]. Different values for the damage number were considered (0.75, 0.6, 0.45, 0.3 and 0.15) and used in the tunnel model. This is the way of assessment the probability of rock failure. Abaqus/CAE has capability to deactivated element analysis of road cut slopes using Hoek and Brown failure systems during tunnel excavation to avoid failure.

Figure 9 (a-d) shows the failure zone of tunnel model for the different considered damage numbers. At a damage number of 0.75 (Figure 9a), the tunnel floor central part is subjected to failure. If the damage number was lowered to become 0.6 (Figure 9b), more failure is expected to occur in the tunnel floor starting from the floor centre and extending outwards. Complete failure of the tunnel floor, in addition to, some failure in the tunnel lower corners may take place if the damage number has reached 0.45 (Figure 9c). Tunnel Floor, corners and part of its side walls will fall if the damage number reached 0.3 (Figure 9d). At lowest considered damage number, 0.15 (Figure 9e and Figure 9f), failure zone covers tunnel floor, its corners and sidewalls (Figure 9e) moreover it can extend to the ground surface of the tunnel (Figure 9f). Therefore, it is recommended to make the adequate supporting systems during tunnel excavation to avoid failure.

4. Conclusion

In this paper, the stability of shallow tunnel in weak rock was numerically analyzed using Abaqus/CAE. The results showed that stresses, strains and displacements are redistributed due to tunnel excavation. The maximum principal stresses zone is in the tunnel floor, while the tunnel roof has maximum displacement vectors. Also, it has been found that at high damage numbers (0.75), failure occurs at the tunnel floor and extending upwards. With decreasing the damage number, the failure extends to the tunnel bottom corners and side walls. At low damage number approaching 0.15, failure can reach to the ground surface above the tunnel. Therefore, a supporting system becomes a must for shallow depth tunnels in weak rock mass.

References


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