Performance based support design for horseshoe-shaped rock caverns using 2D numerical analysis

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\textbf{ABSTRACT}

Excavation in rock may change the stress field and induce excavation damaged zones (EDZ) in the surrounding rock mass. To consider the development of the EDZ, the support design could be evaluated using the convergence-confinement method (CCM). In this paper, an efficient approach is proposed to evaluate the support design based on rock cavern performance. 2D plane strain models are adopted to simulate the excavation effects of horseshoe cross-section rock caverns using the progressive core replacement method. The performance of rock cavern is investigated using CCM. Parametric studies are carried out to analyze the effects of rock condition and sequential excavation. It shows the roof displacement is not changing significantly during excavation if \( Q > 10 \). It also presents the sequential excavation can reduce the range of the EDZ, but there are no obvious relationships for different subdivision methods. Using the data from numerical analysis, the relationships among the rock conditions, the sequential excavation parameters and the cavern performances are mapped using artificial neural network (ANN). An evaluation chart for the support design of a rock cavern is proposed by integrating the ANN models into EXCEL software. A case study is presented to verify the accuracy of the proposed method. It illustrates the feasibility of the proposed approach for practical applications with much less computing time.

\begin{itemize}
\item \textbf{1. Introduction}
\end{itemize}

An accurate knowledge of geological conditions is favorable for rock cavern excavation design. In the design stage, there is uncertainty due to naturally spatial variable phenomena and lack of knowledge or understanding. The uncertainty also comes from the excavation which may affect the stress field and engineering properties of the surrounding rock mass. As shown Fig. 1, the influence zones due to excavation in rock are normally classified as Excavation disturbed Zone (EDZ), Excavation Damaged Zone (EDZ), and Highly Damaged Zone (HDZ) (Siren et al., 2015). The rock mass in the EDZ and HDZ are often fully yielded with very limited self-support capacity. The normal distance from the EDZ edge to the excavation surface is often defined as damage depth (\( D_p \)) to characterize the damage of the rock mass. Another commonly used parameter to represent the damage due to excavation is the convergence on the boundary of the excavation surface. The two parameters are often used for economic evaluations for the rock cavern performance and supports designs (Kwon et al., 2009; Zhang and Goh, 2015; Hijazo and González De Vallejo, 2012; Siren et al., 2015; Feng et al., 2018).

To reduce the damage depth and improve the self-support capacity of the surrounding rock mass, the sequential excavation method (SEM) is often used for excavation in rock by deliberately controlling and adjusting the stress and deformation field. One of the major steps in the SEM process is the selection of the sequential excavation parameters, such as the subdivision of cavern cross-section, the round length (or maximum unsupported excavation length), and the support installation locations (or support installation time). The sequential excavation parameters and the geological conditions determine the selection of the support systems which are usually classified using empirical methods, i.e. the RMR support system (Bieniawski, 1989) and the Q-system (NGI, 2015). The commonly used support elements include the rockbolt, steel set, shotcrete, concrete lining or a combination of the above.

The support system design has to consider its installation time and their interaction with the surrounding rock mass. The convergence confinement method (CCM) is a widely used method to assess the stress relaxation in the excavation surface, the pressure on support and the progressive expansion of the EDZ at different excavation steps (Sinha, \( \cdots \)).
Excavation disturbed Zone (EdZ)
- Reversible damage
- Minor property changes

Excavation Damaged Zone (EDZ)
- Significant property changes
- Fracturing development

Highly Damaged Zone (HDZ)
- Macro-scale fracturing or spallng
- Significant property changes
- Wedge/Surface instability

Fig. 1. Influence zones due to excavation in rock (modified after Siren et al., 2015).

Fig. 2. Characteristic curves for excavation in rock (modified after Lü et al., 2012).

Numerical simulations for excavation in rock have been performed by many researchers to determine the characteristic curves (Karakus, 2007; Pellet et al., 2009). As a full 3D numerical analysis is usually too time-consuming for the preliminary support design, the suitable 2D finite element (FE) plane strain models are often used to consider the 3D excavation effects (Karakus, 2007; Cai, 2008; Janin et al., 2015; Kitchah and Benmebarek, 2016). The key difficulty in the application of the 2D model is how to determine the correspondence between the simulation stage and distance from the excavation face (Alejano et al., 2010). To solve this problem, an improved Longitudinal Displacement Profile (LDP) has been proposed by Vlachopoulos and Diederichs (2009, 2014).

In this paper, the excavation in rock was simulated using the 2D FE plane strain models. Parametric studies were conducted to investigate the influence of SEM parameters and the geological conditions. Using the numerical results, the ANN models were built to identify the relationships among the geological conditions, the sequential excavation parameters and the cavern performances. An evaluation chart was proposed which would provide an evaluation of a support design for the rock caverns. A case study was also conducted to illustrate the process of using the proposed evaluation chart.

2. Theoretical background

2.1. Characteristic curves

The characteristic curves of CCM include GRC, LDP and SCC. The typical characteristic curves for excavation in rock are shown in Fig. 2.
The GRC gives the relationship between the pressure on ground \(p_0\) and the displacement at excavation surface towards the opening (displacement, \(u\)). The in-situ ground pressure before excavation is denoted as \(p_0\). With the increasing of displacement, the pressure on ground decreases linearly in elastic ground or nonlinearly in plastic ground. The Point \(D'\) in GRC is the threshold that the rock mass starts to loosen and thus the supports have to be installed before it. The LDP presents the relationship between the displacement \((u)\) and the distance from the excavation face \((x)\) which can be used to establish the displacement at excavation locations in the SEM process. The SCC is the plot of the pressure on support \((p)\) versus the displacement \((u)\) curve. The stiffness of the support determines the slope of the SCC. As shown in Fig. 2, the SCC of the rigid Support B has higher slope than that of the flexible Support A.

The intersection of the SCC and displacement axis (point \(N\)) could be used to determine the installation position of the support using LDP (point \(N'\)). The intersection between SCC and GRC means the support pressure and the ground stress are balanced which could be used to determine the pressure on the supports. As shown in Fig. 2, supports A, B, C have to sustain \(p_{s,B}\) and \(p_{s,ss}\) respectively. As the required pressure on support \(p_{s,B}\) is the smallest among the three and much less than the support capacity \(p_{s,max}\), the flexible support (Support A) installed at point \(N\) gives the highest factor of safety. The allowable displacement \(u_{max}\) is another parameter to restrain the support design which indicates that a rigid support (Support B) installed at point \(N'\) is more proper due to \(u_0 < u_{max} < u_A\). In this way, the characteristic curves can be used to evaluate the design of the support system.

2.2. Performance functions

To represent the performances, the support system has to satisfy the following three design criteria, i.e. displacement criteria, support capacity criteria and rockbolt length criteria. The performance functions (limit state functions) of the displacement criteria \(g_1(x)\), the support capacity criteria \(g_2(x)\) and the rockbolt length criteria \(g_3(x)\) are shown as,

\[
g_1(x) = u_{max} - u
\]

\[
g_2(x) = p_{s,max} - p
\]

\[
g_3(x) = L_0 - (D_p + L_a)
\]

where \(x\) is the vector of random variables, \(u_{max}\) is the allowable displacement, \(u\) is the displacement at the excavation surface to the opening, \(p_{s,max}\) is the bearing capacity of the support, \(p\) is the pressure on support, \(L_0\) is the length of rockbolt, \(D_p\) is the damage depth of EDZ from the excavation surface, and \(L_a\) is the anchored length of rockbolt which is approximately 1.2 m.

The displacement coordinate of the intersection between SCC and GRC is the displacement at the excavation surface to the opening where the support system is fully functioned. However, the displacement at the installation position of support \(u'\) is different from that at the intersection of SCC and GRC. As shown in Fig. 2, support \(B\) will provide support \(p_{s,b}\) at point \(B\) with its displacement of \(u_B\). The displacement at installation point \(N\) is \(u_N\). The relationship between the displacement at the installation point \(u'\) and that the support system is fully functioned \(u\) is:

\[
w' = u - \Delta u
\]

and

\[
\Delta u = \frac{p_i}{K_i}
\]

where \(p_i\) is the support stress and \(K_i\) is the support stiffness.

The bearing capacity \(p_{s,max}\) in Eq. (2) is not easy to be determined as the group effects of the support system are hard to be defined. It is usually assumed \(p_{s,max}\) as the sum of the bearing capacity of each individual support element (Hoek, 2007). The support capacities of three commonly used support systems, i.e. the steel set, the rockbolt and the shotcrete lining, and their stiffness on a circular tunnel with radius of \(r_0\) have been proposed by Hoek (2007) and summarized in Table 1. The length of rockbolt \(L_0\) in Eq. (3) is determined by cavern span \(B\) when the rockbolt is used to suspend the failure zone to the natural arch. An empirical equation to estimate \(L_0\) in moderately jointed hard rock masses has been proposed by 

\[
L_0 = 1.40 + 0.184B
\]

3. Analysis of horseshoe cavern using numerical method

3.1. Numerical models

A horseshoe-shaped rock cavern was successfully constructed in Singapore with span \(B\) of 20 m and a height \(H\) of 27 m. The 2D FE plane strain model is conducted based on the dimensions of this horseshoe cavern using RS\textsuperscript{2} program (Rocscience Inc., 2010), as shown in Fig. 3a. The length and the width of the horseshoe cavern are denoted as \(H\) and \(B\), respectively. The distance of the cavern wall to the model boundary is set to 4H. To study the effects of the sequential excavation, the full-face (FF) excavations and the subdivisions are considered in the numerical analysis.

The cavern is assumed under an isotropic stress state of 10 MPa. Five ground classes are assumed based on Q-value, i.e., very good \((Q = 40)\), good \((Q = 10)\), fair \((Q = 4)\), poor \((Q = 1.0)\) and very poor \((Q = 0.1)\). The elastic moduli of the rock mass before peak are calculated as (Bieniawski, 1984),

\[
E_m = 2RMR - 100 \quad (RMR > 50) \tag*{(7)}
\]

\[
E_m = 10^4((RMR - 10)/40) \quad (RMR \leq 50) \tag*{(8)}
\]

where RMR is the Rock Mass Rating (RMR) and calculated as (Bieniawski, 1984),

\[
RMR = 9.0 \ln Q + 44 \tag*{(9)}
\]

The UCS of the rock mass is (Serafim and Pereira, 1983; Palmstrøm, 2000),

\[
UCS = RMR \tag*{(10)}
\]

The Geological Strength Index (GSI) of the rock mass is (Marinos et al., 2005),

\[
GSI = RMR - 5 \tag*{(11)}
\]
where $\sigma'_1$ and $\sigma'_2$ are the maximum and minimum effective principal stresses at failure, respectively, $m_0$ is the value of the Hoek-Brown constant $m$ for the rock mass, $s$ and $a$ are constants which depend upon the rock mass characteristics, and $\sigma_{ci}$ is the UCS of the intact rock samples.

For the residual properties of rock mass, such as $E_m$, UCS, GSI, $m_0$, $m_s$, $s$ and $a$, the linear interpolation method is used to calculate their values based on the intervals of GSI proposed by Alejano et al. (2010). All the properties of the rock mass used for the numerical analysis are summarized in Table 2.

The excitation effects in rock are simulated using the progressive core replacement method or stiffness reduction method proposed by Swoboda (1979). The excitation core and surrounding rock mass are initially assigned with same mechanical properties and calculated to get the initial balanced stress state. The properties of the excavation core are then replaced and assigned by softened unstressed elastic material. The surrounding rock mass will converge, while the stress filed will rebalanced. The process of replacement is repeated till the excavation core is removed from the model. Totally ten stages are used in this simulation to simulate the excavation process. Point M1 right above the middle of the roof surface (out of excavation core) is set to record the normal stress on the ground ($p_i$). The normal stress $p_i$ versus roof displacement $u$ is the GRC as shown in Fig. 4.

Another circular tunnel model with diameter $D = H$ as shown in Fig. 3b is used to determine the LDP curve for horseshoe cavern by assuming it is the same as that of the circular tunnel (Vlachopoulos and Diederichs, 2014). The damage depth ($D_p$) of each simulation stage is recorded. The simulation stages are associated with the locations from the excavation face along the tunnel axis based on the empirical equation (Vlachopoulos and Diederichs, 2009) as:


Table 2
Geotechnical parameters for numerical models.

<table>
<thead>
<tr>
<th>$Q$</th>
<th>RMR</th>
<th>Before Peak</th>
<th>Residual</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UCS (MPa)</td>
<td>GSI</td>
<td>$E_m$ (GPa)</td>
</tr>
<tr>
<td>0.4</td>
<td>35.75</td>
<td>35.75</td>
<td>30.75</td>
</tr>
<tr>
<td>1</td>
<td>44.00</td>
<td>44.00</td>
<td>39.00</td>
</tr>
<tr>
<td>4</td>
<td>56.48</td>
<td>56.48</td>
<td>51.48</td>
</tr>
<tr>
<td>10</td>
<td>64.72</td>
<td>64.72</td>
<td>59.72</td>
</tr>
<tr>
<td>40</td>
<td>77.20</td>
<td>77.20</td>
<td>72.20</td>
</tr>
</tbody>
</table>

Fig. 3. Numerical models using RS$^2$ program for (a) horseshoe cavern, (b) circular tunnel, and (c) horseshoe cavern under subdivision.
conditions ($Q < 4$), the roof displacement at the excavation face cannot satisfy the allowable value.

The damage depths versus the distances from the excavation face curves are shown in Fig. 5b. As expected, the smaller the $Q$-value the deeper the damage depth is generated in the surrounding rock mass. The damage depth increases nonlinearly with respect to the distances from the excavation face. For the cavern span of 20 m, the rockbolt length is recommended of 5.0 m according to the support categories of the $Q$-system (NGI, 2015). According to Eq. (3), the depth of EDZ from the excavation surface is calculated as 3.8 m which can be used to separate the purposes of the rockbolt as suspension and arching. For $D_p > 3.8$ m, the rockbolt might not connect to the undamaged rock mass which indicates the spacing of the rockbolt has to be reduced to assist the formation of artificial arch in the surround rock mass.

The normal stress versus roof displacement curves (GRC) for the horseshoe cavern excavated in five ground classes are plotted in Fig. 5c. It shows the normal stresses reduce nonlinearly with respect to the roof displacements. The smaller the $Q$-value the larger normal stresses are generated in the surrounding rock mass and thus larger support pressures are required. Eq. (2) could be used to assess the support safety once the maximum support capacity of the support is known.

### 3.2.2. Effects of SEM parameters

The effects of the subdivision of cavern cross-section and the round length or the maximum unsupported excavation length are investigated using numerical method. The horseshoe caverns with span $B$ of 5 m, 8 m, 10 m, 15 m, 20 m, 25 m and 30 m excavated in five different ground conditions represented by $Q$-value of 40, 10, 4, 1 and 0.4 are considered. The ratios of the damage depth to span ($D_p/B$) versus $Q$-values are plotted in Fig. 6a which shows nonlinear relationships. The larger the round length the larger the ratio $D_p/B$. The roof displacement versus normal stress curves for the subdivisions using 2-sections, 3-sections, 4-sections and full face (FF) are shown in Fig. 6b. It can be seen that the normal stress reduces nonlinearly with respect to the roof displacement. However, there is no obvious relationship for the above four subdivision processes. However, the normal stress versus $u/B$ plots for certain subdivision excavation and $Q$-value could indicate the differences of the subdivision method, see FF and 3-sections excavated in rock mass with $Q = 1.0$ in Fig. 6c. More advanced function is still required to present the relationship between the SEM parameters and the cavern performance.

### 4. Prediction of cavern performance using ANN

The ANN models are built to identify the relationships among the geological condition parameters, the excavation design parameters and the cavern performances obtained from the numerical analyses. The ANN is the multi-layer feed forward back-propagation network which has been widely used in rock engineering for data analysis to find their complex relationships (Zhao and Ren, 2002; Zhao et al., 2008; Tiryaki, 2008). In this study, a 4-$n$-1 structure is used to map the relationships between a set of SEM design parameters $P_i$ (i.e., ground class $P_1$, width of top heading $P_2$, height of top heading $P_3$, round length $P_4$) and support performance $O_j$ (i.e., normal stress $O_1$, damage depth $O_2$ or roof displacement $O_3$) as shown in Fig. 7. The $n$ varies in the three models to predict the support performance $O_j$ based on the training and testing errors.

The transfer function is tangent sigmoid transfer function, denoted as ‘tansig’ in Fig. 7, for both the hidden layer and the output layer which can be expressed as,

$$f(P) = \frac{1}{\arctan(P) + 1}$$

where $f(P)$ is tangent sigmoid transfer function, $P_i$ is the inputs, $i = 1$ for ground class, $i = 2$ for width of top heading, $i = 3$ for height of top heading, and $i = 4$ for round length.

There are 700 data generated from the numerical analysis. Approximately 70% of them are randomly chosen as training data while the rest of them are used for verification. The input $P_i$ and output $O_j$ are normalized using the minimum and the maximum magnitudes of the
parameters based on the numerical results as summarized in Table 3. The equation to normalize parameter $P_i$ is shown as,

$$P'_i = \frac{2(P_i - P_{i,\text{min}})}{P_{i,\text{max}} - P_{i,\text{min}}} - 1$$

(15)

where $P_{i,\text{min}}$ and $P_{i,\text{max}}$ are the minimum and the maximum magnitudes of the parameter $P_i$, respectively.

The network is essentially trained using optimization methods by adapting the weights and biases of neurons and minimizing the mean square error between the predicted and the target values. The performance of the ANN models for training the numerical results to predict the damage depth and the normal stress are shown in Fig. 8a and b, respectively. The comparisons between target and predicted values of the damage depth and the normal stress are shown in Fig. 9a and b.
respectively. The results show very good agreements between the predictions and the testing data.

5. Development of an evaluation chart based on ANN models

To directly use the ANN models for preliminary support design, an evaluation chart is proposed and coded using Microsoft Excel as shown in Fig. 10. The evaluation chart contains the following five parts, the inputs of cavern geometry and ground class (A. Inputs), the prediction of cavern performance using ANN models (B. Prediction), the SEM design parameters (C. SEM Design), the support design parameters (D. Support Design), and the support performance based on the predicted results (E. Performance Functions).

The cavern geometry and ground class in part A have the maximum and minimum boundaries as summarized in Table 3. For part B, the proposed three ANN models are adopted to generate the response surfaces and predict the damage depth and the stress on support. Based on the above ANN models, the support performance \( O_j \) (i.e., normal stress \( O_1 \), damage depth \( O_2 \) or roof displacement \( O_3 \)) are calculated as,

\[
O_j = \frac{\sum_{i=1}^{n} w_j P_i + b_j}{w_j}
\]

where: \( w_j \) is the weights of the neuron \( j \), and \( b_j \) is the bias of the neuron \( j \).

Part C lists the SEM design parameters \( P_i \) (i.e., Q-value \( P_1 \), width of heading \( P_2 \), height of heading \( P_3 \) and round length \( P_4 \)). The width of heading \( P_2 \) and height of heading \( P_3 \) are calculated based on the divisions of span and height respectively. If no subdivision is considered, \( P_2 \) is the cavern span, and \( P_3 \) is the cavern height. The round length \( P_4 \) is set according to the NHI (2009). These parameters are the inputs for the
ANN models to predict the normal stress and the roof displacement to plot the GRC. In Part D, the estimations of support capacity provided by the end-anchored rockbolt, the shotcrete linings and/or the steel set are given according to Hoek (2007). Note that all three support elements are assumed to act independently, and the bearing capacity and the stiffness of the compound support system are the accumulation of their bearing capacity and the stiffness respectively (Özsan and Başarır, 2003). The SCCs of different support types are plotted with GRC to present the ground-support interaction in the diagram. The intersection between GRC and SCC indicates the displacement and the support pressure when the equilibrium is achieved. Part E gives three performance functions as shown in Eqs. (1) to (3) to indicate the support performances.

The evaluation chart could be used to estimate the support safety. A shotcrete lining with thickness \( t_c = 0.1 \) m is selected as an example. The cavern is full-face excavated in rock with UCS and \( E_c \) of 35 MPa and 35 GPa, respectively. The calculated support capacity criteria \( g_2(x) > 0 \) means the support is suitable for the ground conditions and cavern size. As shown in Table 4, the shotcrete lining support with \( t_c = 0.1 \) m could only be effective in good rock conditions with \( Q \geq 10 \) or narrow span caverns with \( B < 10 \) m. The data used to generate Table 4 are also shown in Fig. 10 where the GRC and SCC curves are also included to illustrate the calculation process.

The evaluation chart could be used to assess the functions of the patterned rockbolt to support rock cavern. The damage depth can be predicted using the ANN model. Example of the calculated rockbolt length criteria function \( g_3(x) \), see Eq. (3), is shown in Table 5. The functions of the patterned rockbolts for rock cavern support could be generally separated as suspension element when \( g_3(x) > 0 \) and arching

Table 3

<table>
<thead>
<tr>
<th>( P_1 - Q )-value</th>
<th>( P_2 )-Span, m</th>
<th>( P_3 )-Height of heading, m</th>
<th>( P_4 )-location, m</th>
<th>( O_1 )-inner pressure, MPa</th>
<th>( O_2 )-damage depth, m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. 40</td>
<td>40</td>
<td>54</td>
<td>50</td>
<td>10</td>
<td>50</td>
</tr>
<tr>
<td>Min. 0.1</td>
<td>5</td>
<td>-5</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

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![Fig. 7. Architecture of the ANN models.](image)

![Fig. 8. Performances of the ANN models to train the numerical results to predict (a) the damage depth and (b) the normal stress.](image)
element when \( g_3(x) \leq 0 \) (Li, 2017). Table 5 shows the patterned rockbolts are suspension elements for \( 4 < Q < 10 \) and \( 20 < B < 30 \) m. The patterned rockbolts are arching elements for the rest cases. The rockbolt lengths are much less than the sum of damage depth and anchored length, \( g_3(x) < -2 \) m, for \( Q = 0.4 \) and \( B > 8.0 \) m.

The evaluation chart could be used to optimize the subdivisions of the excavation cross section. The division of span and height would be optimized using displacement criteria \( g_1(x) \). This could be achieved using "Solver" function by minimizing cell D22, and changing cell E11 and cell E12 with their magnitudes of integers and < 5 (not more than 5 subdivisions). The width and height of heading are obtained by dividing the span based on the optimized results. As shown in Fig. 10, cells E11 and E12 are the optimization results for the FF excavation. The width and height of heading are then obtained as shown in cells C11 and C12, respectively, in Fig. 10. The displacement after balance is \( u = 0.018 \) m which is close to the allowable displacement \( u_{\text{max}} = 0.02 \) m. It should be noted that the evaluation chart provides an estimation of the ground-support interaction for preliminary design as the ANN models are build based on the 2D numerical modellings which might not be exactly representing the excavation process and time effect.

![Fig. 9. Comparisons between the targets and the predicted values of (a) the damage depth and (b) the normal stress.](image)

![Fig. 10. Evaluation chart for the preliminary support design.](image)
6. Case study

The evaluation chart is applied to evaluate a real case design of Kaletepe tunnel presented by Sari and Pasamehmetoglu (2004). The highway tunnel with a wall height of 9.6 m, a width of 12.7 m and a length of 2.5 km was excavated under a hill with the maximum overburden of approximately 300 m. The test results of core specimen taken from six borehole sections and site investigations were used as the input parameters to access the geological conditions. The support recommendation and excavation guide preliminary support design proposed by Sari and Pasamehmetoglu (2004) are shown in Table 6. The tunnel length was divided into seven sections (Section 1 to 7), along its axis for the preliminary support design. Only Sections 2, 5 and 7 are selected in this study to illustrate the applications of the proposed evaluation chart. An allowable displacement is assumed as $u_{\text{max}} = 0.02$ m. The support capacities are based on those listed in Table 1.

For Section 2, the length and the spacing of the rockbolt are 3.9 m and 0.8 m, respectively, which are the average data given in Table 6. The sequential excavation method is top heading and benching. The thickness of the shotcrete lining is 0.25 m. The calculated GRC and SCC are shown in Fig. 11a. The intersection between SCC and GRC determines the displacement and the support pressure which are 0.024 m and 0.207 MPa, respectively. However, the displacement of 0.024 m is larger than $u_{\text{max}} = 0.02$ m indicating the support design should be optimized. Moreover, the SCC curve of shotcrete in Fig. 11a shows the pressure on support is much less than its bearing capacity which means the support is not used wisely.

Fig. 12a gives the calculated GRC and SCC curves of the shotcrete lining and rockbolt support with optimized design parameter, such as round length of 5 m and thickness of shotcrete lining of 0.15 m. The intersection point between SCC and GRC shows the support system could restrain the displacement right of 0.02 m within its bearing capacity.

The GRCs and SCCs are calculated using the evaluation chart to assess the support design proposed by current study and Sari and Pasamehmetoglu (2004) for Section 5 and Section 7. The support parameters are also given in Table 6. The calculated GRCs and SCCs of the support designs for Section 7 and Section 5 are shown in Fig. 11b and c, respectively. The intersections between SCC and GRC show the displacements are smaller than the allowable displacement $u_{\text{max}} = 0.02$ m which means the support system could satisfy the requirement. For Section 5, the intersection between SCC and GRC in Fig. 11c shows the rockbolt has no effective support which means the shotcrete is strong enough to provide the support pressure. The spot rockbolt could be used to provide the support pressure as shown in Fig. 12c.

To further evaluate the performance of the support design parameters proposed by current study and Sari and Pasamehmetoglu (2004), series of numerical simulations are carried out. The displacement contours around the excavation face without support and with support by proposed method and those by Sari and Pasamehmetoglu (2004) are shown in Table 7. The maximum displacement and the damage depth of EDZ from numerical analysis and evaluation chart are also listed in Table 7 for comparison purpose. It can be seen that the support designs proposed by current study and Sari and Pasamehmetoglu (2004) can restrain the roof displacement within $u_{\text{max}} = 0.02$ m. The revised support design using evaluation chart proposed in this study adopts less supports. The roof displacement and the damage depth of EDZ resulted in numerical analysis and evaluation chart are further plotted in Fig. 13a and b, respectively. The average differences between the numerical analysis and evaluation chart to calculate the maximum displacement and the damage depth of EDZ are 12% and 4.5%, respectively.
Table 6
Support design parameters.

<table>
<thead>
<tr>
<th>Ground condition</th>
<th>Proposed by Sari and Pasamehmetoglu (2004)</th>
<th>Current research according to evaluation chart</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Section 2</td>
<td>Section 7</td>
</tr>
<tr>
<td>Ground condition</td>
<td>Poor</td>
<td>Fair</td>
</tr>
<tr>
<td>SEM parameter</td>
<td>Round length, m</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>Sequential excavation</td>
<td>Top heading &amp; benching</td>
</tr>
<tr>
<td>Rockbolts in pattern</td>
<td>Length, m</td>
<td>3.9</td>
</tr>
<tr>
<td></td>
<td>Spacing, m</td>
<td>0.5–1.0</td>
</tr>
<tr>
<td>Shotcrete linings</td>
<td>Thickness, m</td>
<td>0.2–0.3</td>
</tr>
</tbody>
</table>

Fig. 11. Calculated GRCs and SCCs using evaluation chart to assess the support designs proposed by Sari and Pasamehmetoglu (2004) for (a) Section 2, (b) Section 7, and (c) Section 5.

Fig. 12. GRCs and SCCs to evaluate the revised support designs for (a) Section 2, (b) Section 7, and (c) Section 5.
7. Conclusions

A support design method for horseshoe cross-section rock caverns is proposed in this paper with considerations of the progressive damage of the rock mass using the 2D numerical modelling and the artificial neural network (ANN). The excavation effects of the rock cavern are simulated using the 2D finite element plane strain models. The performances of the rock cavern during excavation are investigated based on the convergence-confinement method (CCM).

Parametric studies are conducted to analyze the effects of the Q-values on the EDZ development of rock cavern with a span of 20 m and a height of 27 m under isotropic stress of 10 MPa. It is found that the roof displacements versus the distances from the excavation face curves increase nonlinearly when $Q < 4$ and become insignificantly for $Q > 10$. Furthermore, the smaller the Q-value the deeper the damage depth is generated in the surrounding rock mass. The spacing of the rockbolt has to be reduced for poor ground classes ($Q \leq 1$) to assist the formation of the artificial arch in the surround rock mass. The effects of sequential excavation parameters are also investigated by changing the size of rock cavern and the subdivision of cross-sections. It is found that the subdivision can reduce the range of the EDZ but there are no obvious relationships for different subdivision methods. More advanced
Fig. 13. Comparisons between the results from numerical analysis and evaluation chart (a) displacement and (b) damage depth of the EDZ.

function is still required to present the relationship between the SEM parameters and the cavern performance.

The ANN models are built using the numerical results to map the relationships among the rock mass condition, the sequential excavation parameters and the cavern performances. Totally 700 data are generated from the numerical analysis. Approximately 70% of the data are randomly chosen as training data and the rest of them are used for verification. Good agreements between the predictions and the testing data are obtained. An evaluation chart is proposed by integrating the ANN models into the EXCEL software. The proposed evaluation chart provides an effective method to evaluate the support safety, the functions of the patterned rockbolt and the optimization of subdivisions of the excavation cross section. The evaluation chart is evaluated to evaluate a real case design of Kaletepe tunnel. Comparing to the preliminary design presented by Sari and Pasamehmetoglu (2004), the support designs evaluated by the proposed evaluation chart can restrain the roof displacement. The good agreement between the numerical results and the results of evaluation chart shows the proposed evaluation chart could be used in practical applications and save the time of numerical modelling. It also should be noted that the evaluation chart is feasible to estimate the support for the preliminary design as the ANN models are built based on the 2D numerical results which might not be exactly representing the complex excavation process and time effect.

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References


