Preliminary analysis on excavation stability of Beishan Underground Research Laboratory

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Abstract. According to construction requirements of the first underground research laboratory (URL) for high-level radioactive waste (HLW) disposal in China, preliminary analysis on the excavation stability was carried out. Based on in-situ stress conditions and preliminary design of the URL, excavation induced stresses were revealed by numerical simulation. Then, through coordinate conversion, the excavation induced stresses were converted into local coordinate system perpendicular to the tunnel. Finally, an analytical method that compares rock strength with stress conditions is used to evaluate the excavation stability of surrounding rock in the local coordinate system. Results of this study show that: due to the high strength of Beishan granite and low level of the regional in-situ stress, the Beishan URL is highly likely to remain stable after excavation, the surrounding rock can hardly be destroyed by both tensile and compressive stress. Relatively unstable parts of the URL are the bottom of the shaft, rotation parts of the -560m-platform and some corners. These parts should be paid attention to in the future excavation process.

1. Preface
HLW refers to the waste produced by nuclear industry that has strong radioactivity and high toxicity. Deep geological disposal is currently recognized as a feasible method for HLW disposal internationally, which means to build an underground repository to bury the waste body for tens of thousands years to ensure safety. Construction of the URL is a necessary step to verify reliability of the repository$^{[1]}$.

Given the situation that China's first URL project has now been approved, research work on the URL excavation stability should be carried out as soon as possible.

2. Background data

2.1. Site condition
Beishan area located in Gansu province is the preferred preselection area for HLW disposal in China. This area presents a typical Gobi landform, which has the following characteristics: flat terrain, bare bedrock and sparse vegetation.

China’s first URL is located in the Beishan Xinchang site. So far, five boreholes have been drilled in the site. Statistical results of drill core RQD values showed that rock integrity in this area is excellent.

2.2. In-situ stresses$^{[2]}$
A single-loop hydraulic fracturing system was used to measure the in-situ stresses of the URL site. The test results of several boreholes are very close, and the horizontal stress increases linearly with the increase of depth, indicating that the geological structure of Xinchang rock mass is very simple. Fitting equations of maximum($\sigma_{hi}$) and minimum($\sigma_{h}$) horizontal principal stresses with depth(H) are as follows:

$$\sigma_{hi} = 0.0208H + 4.910$$  \hspace{1cm} (1)

$$\sigma_{h} = 0.0152H + 2.728$$  \hspace{1cm} (2)

Statistical analysis shows that the direction of $\sigma_{hi}$ in the Xinchang rock mass is dominated by NEE (average value: N55°E) within depth of 600m. This is also an important input for the determination of boundary conditions in subsequent numerical simulations.

Based on density of Beishan granite, the intermediate principal stress (self-weight stress $\sigma_i$) of the rock mass can be obtained as:

$$\sigma_i = 0.0267H$$  \hspace{1cm} (3)

2.3. Mechanical parameters of Beishan granite[3]

About 42m granite core were selected from 5 boreholes, and standard specimens were elaborately prepared. Based on preliminary measurement results, it was found that the rock density is insensitive to the change of depth, the average density of Beishan granite is about 2.67g/cm³. TAW-2000 testing machine was used to carry out strength and deformation tests of Beishan granite. The test results are shown in Table 1. All these parameters are not sensitive to depth.

| Parameter                      | Abbreviation/ Unit | Average value | Range Minimum value | Minimum value |
|--------------------------------|--------------------|---------------|---------------------|---------------|
| Elastic modulus                | E / GPa            | 60            | 50                  | 70            |
| Poisson’s ratio                | $\nu$ / -          | 0.25          | 0.20                | 0.28          |
| Uniaxial compressive strength | UCS / MPa          | 170           | 110                 | 235           |
| Tensile strength               | TS / MPa           | 11            | 6                   | 15            |

2.4. Crack initiation stress($\sigma_{ci}$)

$\sigma_{ci}$ is one of the key stress thresholds in the process of rock failure under compression condition, which can be used as the lowest stress limit for rock damage[4]. In view of the fact that ISRM has not yet proposed a method to solve $\sigma_{ci}$, we used AE[5] and LSR[6] method to determine it, and mean value of the two independent methods was used as the final solution. The results indicate that $\sigma_{ci}$ shows an linear increasing trend with the increase of UCS. The ratio of $\sigma_{ci}$ to UCS is about 0.45.

2.5. Preliminary design of the URL

Layout of the URL adopts the form of “ramp & shafts & platforms” (see Figure 1). Three vertical shafts are respectively personnel lift shaft, air inlet shaft and air outlet shaft, which all go straight down to the bottom(-560m) of the URL. Design of the ramp adopts curved spiral descending form. Specific parameters of the design are as follows: slope 1:10, section dimension 7m, turning radius 400m. In addition, the two platforms are located at depths of -240m and -560m, which are the main test areas.
3. Process of numerical simulation

3.1. Simplification
In view of the fact that the URL has not yet been excavated, the surrounding rock conditions have not been obtained in detail, numerical simulation at this stage needs to be simplified. Therefore, during numerical simulation process, the rock mass was considered isotropic homogeneous. In the simulation process of this paper, the effect of cracks, groundwater and rock creep will not be considered.

3.2. General consideration
Stress state of a circular tunnel can be solved by both analytical and numerical methods. Taken layout of the URL into consideration (Refer to Figure 1, small tunnel stretches for several kilometers), if we directly use numerical simulation method to solve the stress conditions after excavation and simulate damage of the surrounding rock, it will involve massive calculations. In addition, based on elasticity assumptions mentioned above, from the perspective of obtaining the surrounding rock stress conditions and evaluating the stability of the URL, we cannot obtain more information by using numerical methods than analytical methods. In view of the above reasons, we would like to use a method of numerical simulation combined with analytical method to carry out the stability analysis in the subsequent simulation process.

3.3. Analytical solution
Taking any cross-section of a circular tunnel as research object, the far-field stress conditions can be transformed into two principal stresses on the plane perpendicular to the tunnel axis in the near field (plane-strain problem, see Figure 2), so that the three-dimensional (3D) problem can be converted into a two-dimensional (2D) problem. After this, it will be very efficient to use the analytical method to evaluate the stability of the tunnel.

The following is the specific process of using the analytical method to solve the stress around a circular tunnel (see Figure 3 for reference):

Under the maximum principal stress, the radial stress ($\sigma_{rr}$), tangential stress ($\sigma_{t\theta}$) and shear stress ($\tau_{t\theta}$) around a circular tunnel can be calculated as:

\[
\sigma_{rr} = \frac{\sigma_1}{2} \left(1 - \frac{a^2}{r^2}\right) + \frac{\sigma_2}{2} \left(1 + 3 \cdot \frac{a^2}{r^2} - 4 \cdot \frac{a^2}{r^3}\right) \cos 2\theta \tag{4}
\]

\[
\sigma_{t\theta} = \frac{\sigma_1}{2} \left(1 + \frac{a^2}{r^2}\right) - \frac{\sigma_2}{2} \left(1 + 3 \cdot \frac{a^2}{r^2}\right) \cos 2\theta \tag{5}
\]

\[
\tau_{t\theta} = -\frac{\sigma_1}{2} \left(1 - 3 \cdot \frac{a^2}{r^2} + 2 \cdot \frac{a^2}{r^3}\right) \sin 2\theta \tag{6}
\]
Under the minimum principal stress, the radial stress ($\sigma_3$), tangential stress ($\sigma_{3\theta}$) and shear stress ($\tau_{3\theta}$) around a circular tunnel can be calculated as:

$$\sigma_3 = \frac{\sigma_1}{2} (1 - \frac{a^2}{r^2}) - \frac{\sigma_3}{2} (1 + 3 \cdot \frac{a^4}{r^4} - 4 \cdot \frac{a^2}{r^2}) \cos 2\theta$$  \hspace{1cm} (7)$$

$$\sigma_{3\theta} = \frac{\sigma_1}{2} (1 + \frac{a^2}{r^2}) + \frac{\sigma_3}{2} (1 + 3 \cdot \frac{a^4}{r^4} \cos 2\theta)$$ \hspace{1cm} (8)$$

$$\tau_{3\theta} = -\frac{\sigma_3}{2} (1 - 3 \cdot \frac{a^4}{r^4} + 2 \cdot \frac{a^2}{r^2}) \sin 2\theta$$ \hspace{1cm} (9)$$

Superimpose the above equations, we can get the stress conditions around the tunnel:

$$\sigma_\theta = \frac{\sigma_1 + \sigma_3}{2} (1 - \frac{a^2}{r^2}) + \frac{\sigma_1 - \sigma_3}{2} (1 + 3 \cdot \frac{a^4}{r^4} - 4 \cdot \frac{a^2}{r^2}) \cos 2\theta$$  \hspace{1cm} (10)$$

$$\sigma_\theta = \frac{\sigma_1 + \sigma_3}{2} (1 + \frac{a^2}{r^2}) - \frac{\sigma_1 - \sigma_3}{2} (1 + 3 \cdot \frac{a^4}{r^4} \cos 2\theta)$$ \hspace{1cm} (11)$$

$$\tau_{\theta} = -\frac{\sigma_1 - \sigma_3}{2} (1 - 3 \cdot \frac{a^4}{r^4} + 2 \cdot \frac{a^2}{r^2}) \sin 2\theta$$ \hspace{1cm} (12)$$

Substituting $r = a, \sigma_\theta = 0$ and $\tau_{\theta} = 0$ into the above equations, we can get the stress conditions on the tunnel excavation surface:

$$\sigma_\theta = \sigma_1 + \sigma_3 - 2(\sigma_1 - \sigma_3) \cos 2\theta$$  \hspace{1cm} (13)$$

According to Equation (13), we can find that the minimum tangential stress (compression is positive) on the tunnel boundary appears at the waist ($\theta = 0^\circ$ or $180^\circ$). Putting $\theta = 0^\circ$ into the equation, we get $\sigma_{\theta-0^\circ} = 3\sigma_1 - \sigma_3$. It can be seen from this result that if the ratio of the maximum principal stress ($\sigma_1$) to the minimum principal stress ($\sigma_3$) around a circular tunnel is greater than three ($\sigma_1/\sigma_3 > 3$), tensile stress ($\sigma_\theta$) will occur. And if $\sigma_1$ is bigger than $T_S$ of the rock, tension failure will occur.

The maximum tangential stress on the tunnel boundary appears at the top or bottom ($\theta = 90^\circ$ or $270^\circ$). Putting $\theta = 90^\circ$ into the equation, we get $\sigma_{\theta-90^\circ} = 3\sigma_1 - \sigma_3$. It is obvious that if this value ($3\sigma_1 - \sigma_3$) is greater than UCS of the rock, compression failure will occur.

![Figure 2. 2-d simplification of stress state around a circular tunnel.](image)

![Figure 3. Decomposition of the stress around a circular tunnel](image)
3.4. Process
Based on above considerations and theoretical basis, specific steps of numerical simulation in this paper can be described as follows:

1. **Build mesh model and initial stress field**. According to the layout of the URL, scope of the model is determined, specifically: a rock mass 2300m from east to west, 1900m from north to south and 850m from ground surface to the bottom. The positive directions of the X, Y and Z axis are respectively east, north and up. Based on in-situ stress test results (See Section 2.1) and indoor compression test results (See Section 2.3), parameters and boundary conditions of the model can be set. Subsequently, the initial stress field is constructed through self-balance of the model.

2. **Import main structure of the URL into the model**. The URL layout drawing must be converted to a dense and uninterrupted polyline format to be recognized by FLAC3D and imported into the model for calculation. Therefore, we need to carry out the following works: ① In order to facilitate the subsequent processing of the main structure of the URL, the design drawing of URL was divided into four parts according to different geometric characteristics. The four parts are respectively: shaft, ramp, -240m-platform and -560m-platform. ② Use CAD embedded tools to convert the centerlines of the above four structures into spline curves. ③ Adjust the segmentation accuracy and re-convert the spline curves into dense and continuous polylines. ④ Load the polylines into FLAC3D (see Figure 4).

3. **Spatial stress conversion and stability analysis**. Initial stress state of the mesh model is expressed by maximum, minimum and self-weight stress fields, which is inconvenient for stress relief and 2D analysis. Therefore, it is necessary to perform spatial stress conversion. The specific processes are as follows: ① Use FISH language to capture stress tensor information of the zones in the mesh model. ② Convert the principal stress tensors into the global coordinate system, which means decomposing the principal stresses along the coordinate axes. The stress state will be expressed in the form of normal and shear stresses. ③ Convert the normal and shear stresses into the local coordinate system, which means decomposing the normal and shear stresses along the URL main structure direction and on the plane perpendicular to it. ④ Release the normal stresses and shear stresses along the axis of the URL tunnel. ⑤ On the 2D plane perpendicular to the axis of the tunnel, perform stress conversion to obtain the state of the principal stress field. At this point, the conversion of stresses from 3D to 2D and from global coordinate system to local coordinate system is completed. Combined with the analysis method mentioned in Section 3.3, stress state and failure tendency of surrounding rock at different positions in the URL can finally be obtained.

![Figure 4. Mesh model with the main structure of the URL.](image)

4. Stability analysis

4.1. **Principal stress state of the URL in the local coordinate system**
Principal stress state of the main structure of the URL in the local coordinate system after excavation is outlined in Figure 5 and Figure 6. It can be seen that the principal stress of the rock mass after excavation does not vary smoothly with the depth going down in the local coordinate system. This is because the far-field principal stress in the local coordinate system is not affected by a single factor, but by the combined influence of depth (overall stress level) and axial direction (stress component). In addition, we can also find that in the URL depth range, the maximum value of \( \sigma_1 \) is about 19.67MPa, which is close to \( \sigma_1 \) in the global coordinate system. However, the maximum value of \( \sigma_3 \) is significantly different, about 15.26MPa, which is closer to \( \sigma_2 \) in the global coordinate system.
4.2. Possibility of tensile and compressive failure

According to Section 3.3, the results of constructing $\sigma_1/\sigma_3$ contours and $(3\sigma_1-\sigma_3)/$ UCS contours in local coordinate system are outlined in Figure 7 and 8. It is observed that the maximum value of $\sigma_1/\sigma_3$ is about 1.51, which appears at the shaft position. This value is far from reaching the condition ($\sigma_1/\sigma_3>3$) for tensile stress to occur, which means excavation of the URL will not cause tension damage. Relatively large values of $(3\sigma_1-\sigma_3)/$ UCS appear at the rotation position of the 560-meter platform and the bottom of the shaft, but the maximum value does not exceed 0.27, which means that the main structure of the URL is unlikely to undergo compression failure either.

![Figure 5. Maximum principal stress distribution of the URL in local coordinate system (unit: MPa).](image1)

![Figure 6. Minimum principal stress distribution of the URL in local coordinate system (unit: MPa).](image2)

![Figure 7. Distribution characteristics of $\sigma_1/\sigma_3$ of the URL in local coordinate system.](image3)

![Figure 8. Distribution characteristics of $(3\sigma_1-\sigma_3)/$ UCS of the URL in local coordinate system.](image4)
4.3. Re-analysis of destruction tendency

In order to deepen the stability analysis of the URL, we introduced a new parameter called crack initiation index (CII) to measure the destruction tendency of the surrounding rock. CII represents the ratio of the maximum compressive stress level on the excavation boundary(3σ₁-σ₃) to the minimum damage strength of the surrounding rock(σ₃). The CII values were calculated and then outlined in Figure 9. It is observed that the CII value of the tunnel area after URL excavation is less than 0.60, which means if the rock mass is in good integrity the possibility of rock mass failure would be small. The CII index is of great value for judging the failure trend of surrounding rock, so further stability analysis was carried out for the four parts of URL. The analysis results are as follows:

(1) **Shaft.** Maximum value of CII occurs at the bottom of the shaft, which is close to 0.6, indicating that the bottom of the shaft is the area with the highest risk of instability and needs to be paid attention to in the URL excavation process.

(2) **Ramp.** As the depth increases, CII shows an increasing trend, but fluctuates at the rotating part. Therefore, in the actual engineering excavation process, it is necessary to pay attention to the change of excavation-induced stress when downward and rotation occur at the same time. In addition, the maximum CII value of the entire ramp is greater than 0.47, and some large values are distributed at the bottom of the ramp and the final rotation position. It can be predicted that if some unfavorable conditions (such as cracks, groundwater, etc.) are found in these places, the occurrence of limited rock mass instability will become a high probability event.

(3) **-240m-platform.** Structure of the -240m test platform is not complex and the CII value is between 0.18 and 0.23. Stress-induced rock mass instability is unlikely to happen in this part.

(4) **-560m-platform.** The -560m platform is of high importance in the URL and requires special attention. Therefore, we have partially enlarged the contour map of this area, as shown in Figure 10. It can be seen that the -560m-platform consists of three parts, namely: the public part(①), the rotation part (②) and the straight tunnel part (③).

① **The public part.** Layout of the public area is relatively complicated, and it is difficult to fully express the behaviors of the surrounding rock during excavation using simplified simulation methods. Therefore, some supplementary analysis of excavation stability needs to be done in the future. In addition, according to the the current design, a lot of transitional corners are set at the tunnel connection. These corners have high CII values(close to 0.5), which cannot be ignored in future stability analysis process.

② **The rotation part.** There are abundant stress conditions in this area(CII value from 0.3 to 0.5), which is very suitable for carrying out underground experiments under different stress conditions. The maximum CII value of the -560m-platform appears in this part, which means the stress concentration part of this place has the highest risk of instability during the excavation process.

③ **The straight tunnel part.** This part can be used to expand the URL space in the future. According to the current design, the CII value is small. Therefore, the excavation risk here is not high.

![Figure 9. CII distribution characteristics of the URL](image-url)
Figure 10. CII distribution characteristics of the -560-platform

5. Conclusions
Based on preliminary design and rock mass conditions of the first URL in China, analysis of the excavation stability was carried out by numerical simulation combined with analytical method. The main conclusions are as follows:

(1) According to the current conditions, excavation of the URL will not cause tensile stress on the excavation boundary, and of course it will not cause tension damage of the surrounding rock.

(2) Maximum stress on the excavation boundary of the URL main structure appears at the bottom of the shaft, but the ratio of this value to the UCS of the surrounding rock does not exceed 0.27, indicating that excavation is unlikely to cause compression failure of the URL.

(3) Crack initiation index (CII) was introduced to measure the destruction tendency of the surrounding rock of the URL. The result shows that relatively high values of CII are mainly located at: the bottom of the shaft (close to 0.6), the rotation part of the -560m-platform (close to 0.5) and some tunnel corners (close to 0.5). In the future excavation process, more attention should be paid to these areas.

Note: This simulation result is only valid for the current design. Layout of the URL may change in the future, but the simulation process proposed in this paper can still be used.

6. References
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