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Tunnel face stability investigation by means of 3D numerical analysis and hand calculations

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ABSTRACT: The first underground radioactive waste repository (NRWR) in Hungary is located near the village of Bátaapáti in the Mórágy Granite Formation. The repository serves as the final storage of the low and intermediate level radioactive wastes (LILW) resulting from the operation of Paks Nuclear Power Plant (NPP). The present paper discusses the stability issue of the tunnel face, including investigation of the Western Exploratory Tunnel (WET) of the NRWR. The face stability has been analyzed assuming that the WET crosses a fault-zone, which dominantly consists of weak, highly fractured rock mass. There is no well known method to calculate tunnel face stability in thin fracture zones. In this paper an attempt was made by using the method by Leca & Dormieux (1990) to assess the stability of tunnel-face during penetration of a fractured zone. The relevance of the hand calculation was verified with the 3D finite difference model, where the exect tunnel geometry, face support and fracture zone is directly considered. The calculations were performed without reinforcement as well, in order to show the efficiency of the support. Furthermore, the effect of groundwater pressure on the tunnel face has been also assessed by conducting all analysis.

1 NATIONAL RADIOACTIVE WASTE REPOSITORY PROJECT IN BÁTAAPÁTI

The Paks Nuclear Power Plant (Hungarian: Paksi Atomerőmű), is the first and only operating nuclear power plant in Hungary. The establishment of the NPP had presented new problems concerning the management of radioactive waste. The temporary storage problem of low- and intermediate level waste were resolved by the Püspökszilágy radioactive Waste Treatment and Disposal Facility (RWTDF) until 2005 when the 5040 m^3 capacity was not adequate. There were many geological investigations carried out between 1997-1999 and 2002-2003 in order to be able to decide where to host the new long-term disposal of low- and intermediate level waste from Paks. In mid- 2005, after considering the economic, environmental and technical aspects the village of Bátaapáti was chosen as the potential location. The project has been financed by RHK Ltd. (Public Limited Company for Radioactive Waste Management). [Kovacs et al. 2012]

1.1 Investigated area

The radioactive waste repository consists of several facilities. General layout of the facilities and the investigated WET can be seen in Figure 1.
The *WET* starts at the 31st meter of the chamber-connecting tunnel and passes 75 meters with a 33 m^2 cross section. The tunnel slope is 5‰ towards the chamber-connecting tunnel. The WET is designed to be able to deepen few pre-drill holes in the I-N1, I-N2 emplacement chambers at the west wing of the chamber field I, and later to explore the neck- and emplacement chambers.

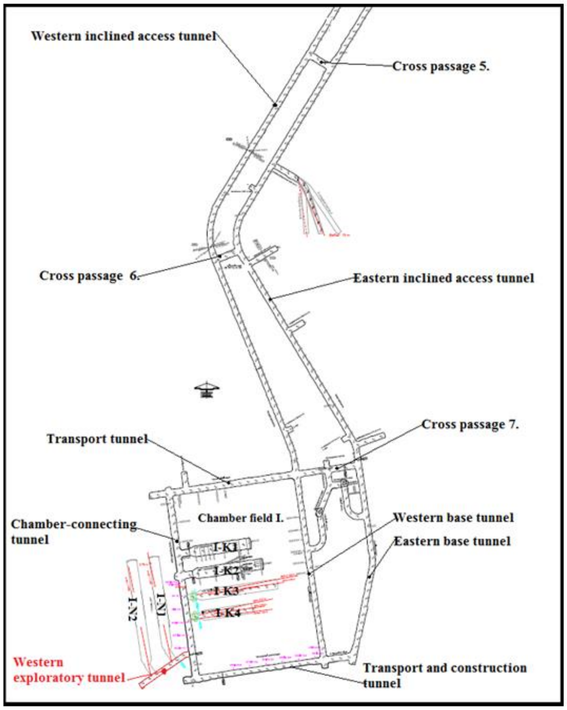

Figure 1. General layout of the repository (Kovacs et al. 2012)

2 PROBLEM STATEMENT, KEY FACTORS INFLUENCING THE STABILITY

During face stability analysis, one might distinguish tunnel face with and without support due to their effect in increasing the factor of safety of the excavation face. This is the reason behind the widespread use of face supporting bolts. The potential instability of an underground opening can pose danger to both workers and equipment.
The stability of the investigated drill and blast tunnel face depends on (Schubert & Goricki 2004):

- The ground type characteristic
- The geometry of the tunnel
- The excavation face support elements
- The groundwater conditions

2.1 Ground type characteristic

The geological and tectonical environment of the western exploratory tunnel was revealed by the a borehole (BeR-15). On the first half of the core significant lithological variability was observed, with monzonite-, syenite- and potassium- feldspar-schliers while on the second half the monzogranite became dominant. Based on preliminary data, there is a major structure around the tunnel, which called Patrick fault zone. The rock mass classification was carried out according to the Q-system (Barton et al. 1974). Table 1 shows the ranges of the rock mass classes.

Table 1. Ranges of rock mass classes (Toth et al. 2014)

Q rock mass classes	Ranges of the Q rock mass classes
I	Q > 10.0
II A	10.0 ≥ 4.0
II B	4.0 ≥ 1.0
III	1.0 ≥ 0.1
IV	0.1 ≥ 0.02
V	0.02 ≥ Q

Table 2 shows the laboratory tested intact rock parameters. In case of greater fault zones such as the analyzed Patrick fault zone, the core zone and its borders consists of weak, very fractured, weathered, soil-like rock, this would be a class V. rock mass, and which should be handled by a soil mechanics approach during the design phase.

Table 2. Intact rock parameters (Toth et al. 2014)

Rock mass class	UCS mass	E mass	γ mass	mb	s	a
	Uniaxial compressive strength	Young-modulus	Unit weight	Constants by Hoek, Kaiser and Bawden (1995)		
	[MPa]	[GPa]	[kg/m³]	[-]	[-]	[-]
I	15.72	38.30	2606.4	3.8850	0.0151	0.5024
	11.94	30.94	2574.7	3.2677	0.0088	0.5034
II	7.99	21.14	2528.9	2.5467	0.0041	0.5056
III	4.64	11.73	2469.3	1.8406	0.0015	0.5104
IV	2.33	5.70	2399.4	1.2578	0.0005	0.5214
V	0.81	2.70	2312.2	0.7818	0.0001	0.5522

2.2 Geometry and support elements

The stability of the excavation face depends on the depth of the tunnel with respect to the ground surface as well. The WET is located 240 m below the ground level. In the present paper, 18 steel rock bolts were taken into consideration on the face as support system. Their tensile capacity is 246 kN. Their systematic allocation can be seen on Figure 2.

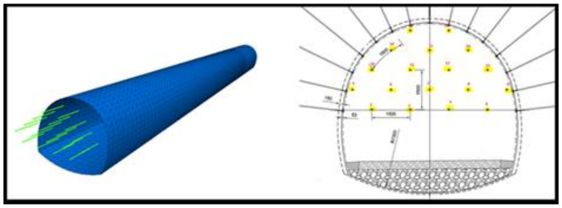

Figure 2. Layout of rock bolts

2.3 Groundwater conditions

The aquifer water has been lowered in the vicinity of the tunnel shafts. Significant groundwater pressure is not expected although to illustrate groundwater pressures effect on the tunnel safety in case of unexpected water leaking a separate calculation was performed. The water pressure was taken into consideration with a value of 0.3 MPa.

3 METHODS USED FOR STABILITY ANALYSIS

To obtain the factor of safety there are several methods used. A number of authors have described the failure mechanism of the tunnel face (Atkinson 2007). This paper aimed to compare three different methods.

- The limit analysis, upper-bound approach by the hand calculation (Leca & Dormieux 1990)
- A FLAC three dimensional model based on the finite difference method were examining the plastic failure of the tunnel face. (Mollon et al. 2009)
- The gravity driven, wedge failure was investigated with several UNWEDGE 3D models.

In this paper, an attempt is made to get the factor of safety, which is intended to ensure that the structure is not near its ultimate state.

The hand calculation was performed based on the limit analysis approach by Leca and Dormieux (1990) under drained and undrained conditions and it analysed the dual-cone active failure. During the calculation the rockbolts were considered as distributed loading on the tunnel face. The ground was controlled by the Mohr-Coulomb criterion. The shear strength of the soil is characterized by the cohesion (c = 3881 kPa) and the friction angle (φ = 23.84°).

The second method used is based on the finite difference method, dealing with the plastic failure as well. A 3D model was created by others (Toth et al. 2014) during the design phase of the performed detailed analysis of the face support using the FLAC 3D program by Itasca Consulting Group (version 5.01). Several changes were introduced into the model by the authors. The Patrick fault zone's properties were redefined to Mohr-Coulomb constitutive model. the rockbolts were applied as cable elements. Calculation of the factor of safety was executed by reducing the strength parameters of the soil. (φ-c reduction).

The third calculation method is the so called limit equilibrium method. A three-dimensional stability analysis and visualization program, called UNWEDGE 3D (version 4.0) was used to examine the tunnel face of the WET. The size and shape of potential wedges in the rock mass, hence the factor of safety value of the underground opening, depend on the geometry of the tunnel, furthermore on the orientation and properties of the significant discontinuity sets. See Table 3.

Table 3. Joint orientations in the investigated area (Toth et al. 2014)

Joint orientations in the medial cross-section of the WET		
Number of joints	Dip	Dip direction
1	76⁰	287⁰
2	37⁰	248⁰
3	65⁰	259⁰
4	37⁰	354⁰
5	51⁰	286⁰

The analysis is based on the worst case occurring. In the present calculations, the factor of safety must be a minimum of 1.5, which is an arbitrarily defined design requirement. A simplification was used in the UNWEDGE models. The 18 rockbolts were integrated in the model on the full face area, not only on the upper half of it, because the program cannot solve this type of layout of bolts.

It was expected that the results of the hand calculations and the 3D model will be consistent as both methods were examining the plastic failure of the tunnel face. Whereas the program UNWEDGE investigated the structurally controlled stability. The emerging failure mechanism is which results in a lower factor of safety.

4 RESULTS

According to the hand calculation, the factor of safety is 5.9 and 7.0 without and with support, respectively. Considering the water pressure on the tunnel face the factor of safety is 3.0 and 4.1.
The results of the three dimensional numerical model is consistent with the results of the hand calculation. The predicted factor of safety is approximately 5.4 and 5.7 without and with support, respectively. Considering the water pressure 3.0 without and 3.4 with support which is the most useful result, as the by FLAC 3D program and hand calculated factor of safety values are fairly the same. Hence, simple hand calculation such as the upper bound approach by Leca and Dormieux (1990) can be used to characterize face stability in fractured zones as well.Table 4 and Figure 3 summarize the results of all the investigated cases.

Table 4. Results for tunnel face stability investigation

	Analized conditions			
Factor of safety	Without support	With support	Without support considering water	With support considering water
FS according to hand calculations	5.9	7.0	3.0	4.1
FS according to FLAC models	5.4	5.7	3.0	3.4
FS according to UNWEDGE models	181	194	1.0	1.6

Based on these results, it can be concluded, that the tunnel face is stable against plastic failure. It should be noted, that the hand calculation is applicable only if the tunnel geometry can be approximated with a circular tunnel. If the tunnel geometry is more complex or the interaction of several tunnels is questioned, the three dimensional modelling is needed to be used.
The calculations made by UNWEDGE indicates that the wedge shaped blocks at the tunnel face are stable against gravity induced, structualy controlled failure (factor of safety is above 180). Applying 0.3 MPa constant water pressure on the face causes a huge decrease in the factor of safety values, although using rockbolt support the face can be stabilized.

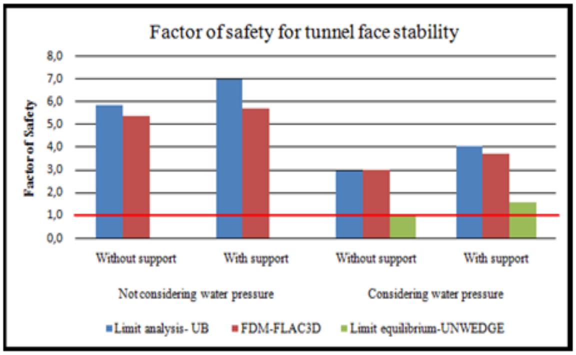

Figure 3. Factor of safety values (under 8.0) for tunnel face stability

It can be stated that the plastic failure of the ground in front of the tunnel is the governing failure mode if the water pressure is not considered. With 0.3 MPa water pressure on the face the wedge type of failure is the governing failure. The rock support certainly increases the stability of the tunnel face, as the difference is not significant. Face support is needed to maintain stability, if the presence of 0.3 MPa water pressure is expected. The stand-up time of the face was out of the scope of this paper, so it is possible that face stabilization would be needed if the face stability needs to be maintained for decades.

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