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Uncertainties and Complexities of the Geological Model in Slope Stability: a Case Study of Sabzkuh Tunnel

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Abstract

Slope stability analysis is a geotechnical engineering problem characterized by many sources of uncertainty. In slope stability computations, some of these sources are encountered, such as geological details missed in the exploration program, estimation of soil properties that are difficult to quantify and many other relevant factors. Therefore, accurate reproduction of the spatial variability in the field could be essential to decrease uncertainty. The Sabzkuh-Choghakhor water conveyance tunnel is currently under construction using the NATM and TBM tunnelling methods in the Zagros Mountains in south-western Iran. In the slope stability assessment of the Sabzkuh tunnel portal, despite adequate geotechnical investigations, field studies have not been performed with appropriate accuracy. A landslide and collapse has occurred in a part of the portal due to tunnel excavation. In this paper, the importance of having a precise and predetermined schedule for selecting site location, monitoring, complexities of the geological model, uncertainty and its effects on the stability of the trench were investigated and the necessity of comprehensive slope management was emphasized.

Keywords: *collapse monitoring, ground complexity, uncertainty, slope stability.*

1. Introduction

Uncertainty is inherent in geotechnical design [1, 2]. Geological anomalies, inherent spatial variability of soil properties, scarcity of representative data, changing environmental conditions, unexpected failure mechanisms, simplifications and approximations adopted in geotechnical models and human mistakes in design and construction are all factors contributing to uncertainty [3]. Therefore, geotechnical engineering almost always has to deal with uncertainty, whether it is formally acknowledged or not [4].

Slope stability is one of the most important and delicate problems in civil engineering, which particularly encountered in large and important construction projects, such as dams, highways and tunnels. Slope stability analysis, an important research field of engineering disaster prevention and mitigation, is used to determine the most dangerous slip surfaces (potentially the most dangerous sliding surface) and the safety factor of the slope [5]. Therefore, the main aim of slope stability analysis is to design a slope that is stable, economical and has the least possible chance of failure [6].

The Sabzkuh-Choghakhor water conveyance tunnel under construction in the southwest of Iran is 10,617 metres in length and 0.01% in gradient (Figure.1). The tunnel is going to be excavated by a double-shield tunnel boring machine (DS TBM). The information obtained from the site, including field reconnaissance as well as geotechnical and geophysical studies revealed that the tunnel from the beginning to 0+390 km (T1) is located in the alluvium. So, taking into consideration the application range of DS TBM and its limitation for working in alluvial soils, this part of the tunnel is being excavated by conventional methods [7, 8, 9, 10, 11].

Obviously, a flatter slope in the geometric design of a trench leads to more stable conditions with regard to sliding or collapsing; however, in the Sabzkuh tunnel project, it was not the optimal and most economical solution due to space limitations and not taking (Demising) the land from its owner. Therefore, despite these limitations to the geometric design of the trench, the height of the tunnel entrance has been considered at over 32 metres with benches four metres wide and eight metres high (2H: 1V & 1.5H:1V).

The stability of this portal was examined by using SLIDE 5.0 and FLAC/Slope software and it was found that the slope is stable in both static (FS: 1.5) and pseudo-static (FS: 1.1) conditions. An appropriate safety factor for the stability of the trench was obtained in the short-term (during construction) in accordance with the assumption of saturated soils in the static mode, as well as in the pseudo static mode based on the assumption of a horizontal peak acceleration of 0.251 g for return periods of 75 years and a lifetime of 50 years. After excavating 35 metres of tunnel and confronting unexpected ground conditions, a face collapse initiated and developed rapidly. This problem caused a failure in the tunnel portal area. After sliding and collapsing in the portal of the tunnel we found that an error occurred at one stage of the geological field investigation, geotechnical data or in the trench design. Thus, regardless of the primary data and assumptions, all steps were reviewed and soil sampling and laboratory tests were performed again to identify the cause of the collapse.

This collapsing is due to uncertainties in the input parameters in stability analysis. Thus, the present study is concerned with answering the question of whether the assumptions used in designing the construction process are matched with observations.

2. Difference Between Geological and Geotechnical Surveying

As slope stability analysis is not a simple task, it requires knowledge from other disciplines. Therefore, a joint effort from geologists, geotechnical engineers and seismologists is required to tackle this problem.

A comprehensive geological model is absolutely fundamental to any slope design. Without such a model, slope designers have to resort to crude empiricism and the usefulness of such designs, with the exception of very simple pre-feasibility evaluations, is highly questionable.

According to the site investigations and two exploratory boreholes, the geology of the portal included the contemporary alluvial and debris. The boreholes log and the profile used in the trench stability analysis are shown in Figure 2a.

After geological mapping of the inside and outside of the tunnel route, reinterpreting the primary geotechnical reports and comparing between these results and those obtained from the boreholes logs and primary geological surveying at the first stage was completed. Fundamental differences were identified, particularly in the diversity and the slope angle of soil layers. Then, in accordance with the USCS, the new samples, which were taken from the trench, were classified. According to the new results of soil classification, the number of soil categories increased from three (ML, CL-ML, SC-SM), in the initial interpretation (Fig. 2a), to six categories (CL, CH, ML, CL-ML, SC-SM, GP-GM) (Fig. 2b).

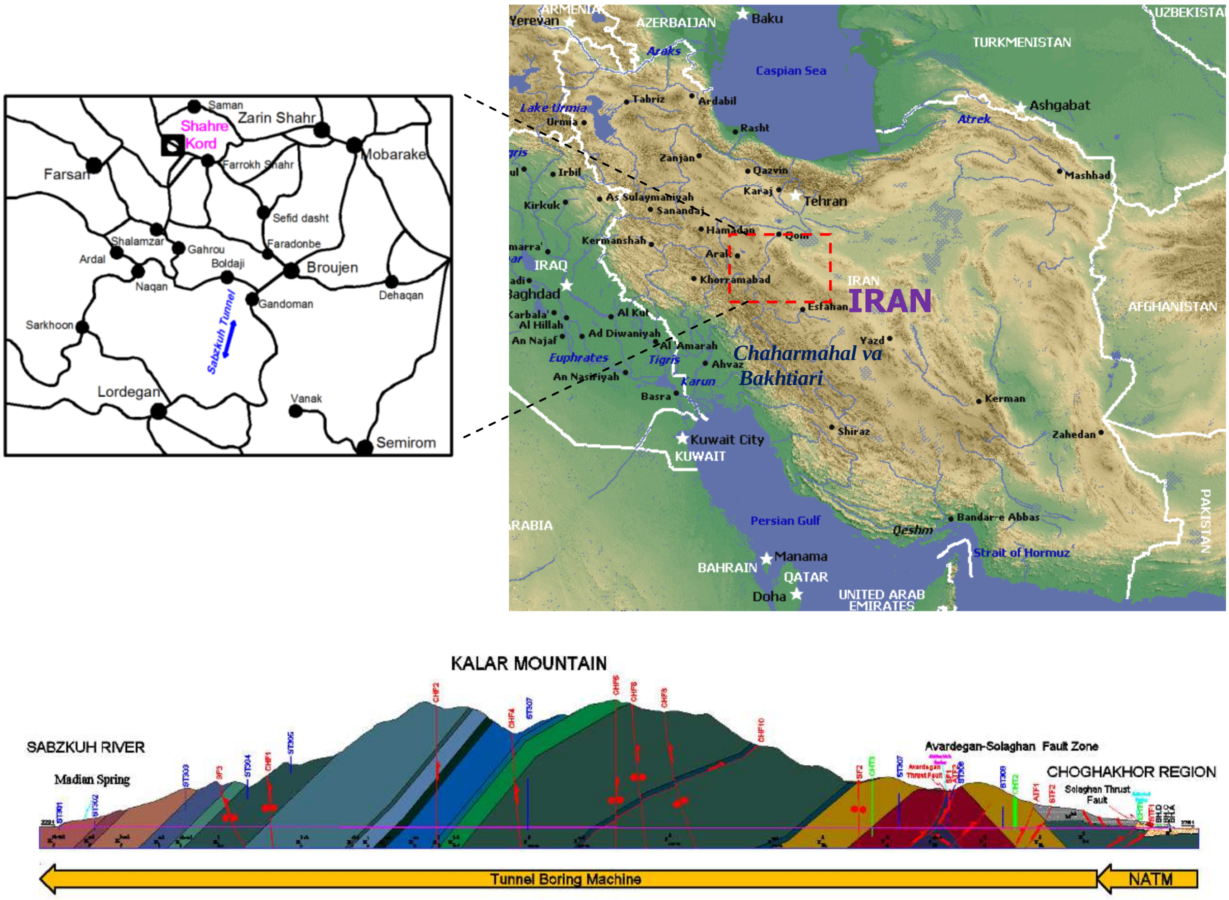
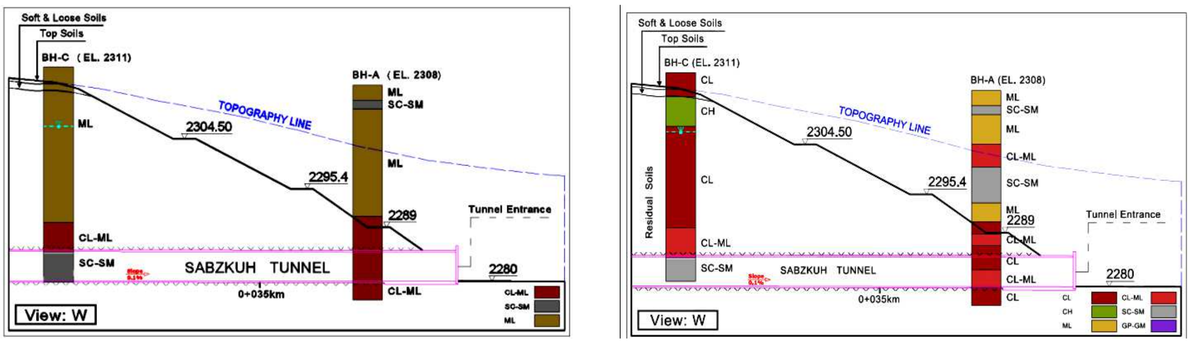


Fig.1. Sabzkuh tunnel location (above) and longitudinal geological profile (below) [7].



(a) (b)

Fig. 2. Boreholes log: (a) initial; (b) modified [9].

The old sliding surface was then detected at the both sides of the portal by surface surveying (SS: 75-80 / 355-005). Due to the existence of muddy soils at the surface, the geometrical characteristics of alluvial layers were immeasurable. Thus, on the waterway path leading to the portal and also to the collapsed zone in the trench, hidden old alluvial layers under contemporary alluviums were observed (50-55/005-010 (Dip/Dip Direction)).

In order to complete the geotechnical investigations, geoelectric studies using the CRP method were performed in several profiles within the trench. From these profiles the movement of bed rock was easily detectable as a result of sliding. The CRP profile and the sliding surface of the trench are shown in Figures 3 and 4, respectively.

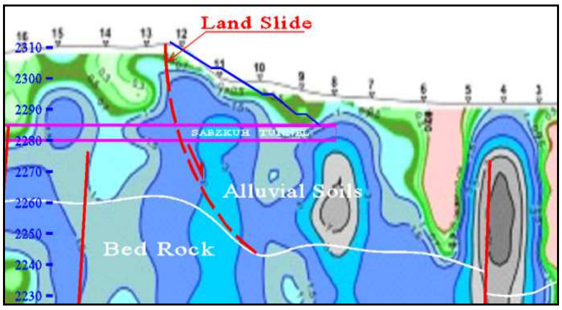


Fig. 3. The CRP profile in portal [8].

Fig. 4. The trace of slide surface on the portal.

At the final stage, the physical and mechanical parameters of soil samples were reassessed by in-situ re-sampling from the soil of cavity and in-situ field tests, and the new geotechnical test results were interpreted by statistical approaches.

The preliminary geotechnical properties of the soil (highlighted in the primary trench stability analysis), in accordance with the previous geotechnical reports and performed tests, as well as the results of the new classification based on the USCS standard (ASTM D-2487) with physical and mechanical parameters of the trench soil, are given in Table 1. Figure 5 shows the previous modified profiles (Fig. 2a) and Figure 6 plots the occurred collapse at the trench.

Table 1. Physical and mechanical parameters of the trench soil.

NO.	SOIL UNIT	γ (GR/CM^3)	(DEG.)	C (KG/CM^2)
1	CL	1.71	22	0.49
2	CH	1.64	19	1.1
3	ML	1.76	26.7	0.18
4	CL-ML	1.73	24	0.35
5	SC-SM	1.88	31.3	0.035
6	GP-GM	2.1	34	0.01
7	SLIDE SURFACE	1.8	65	0.0

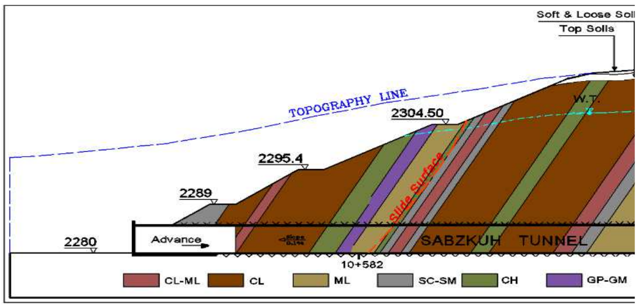


Fig. 5. The modified geology profile [9].

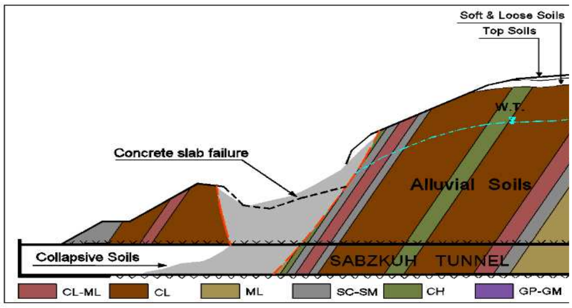


Fig. 6. Tunnel collapsing and the failure of concrete slab [7].

3. Monitoring

The monitoring data can provide a basis for understanding the situation and understanding warning signs in slopes [12]. Therefore, great effort has been paid around the world every year to monitoring and analysing the stability of soil and rock slopes [13].

Wyllie and Mah noted that: "because of the unpredictability of slope behaviour, slope monitoring programs can be of value in managing slope hazards, and they provide information that is useful for the design of remedial work" [14].

Measurement of the surface displacement is important in slope failure prediction models and in conducting risk assessments [15]. Slope monitoring using total station and in situ field observations are common methods to investigate landslides and monitor slope instability.

After completion of the trench construction and before the beginning of the tunnel excavation, many cracks were observed on the concrete slabs, as well as at the construction and expansion joints. Therefore, an adequate monitoring scheme was implemented before and during the tunnel construction in order to monitor the interaction between the tunnel and the trench movements. The monitoring scheme around the study area included the following key elements:

A) Monitoring of crack displacements

Major pre-existing cracks on the concrete slabs were monitored using a series of glass slides to review the development progress of these cracks.

Unfortunately, the veinlet structural fractures were identified on the glass slides. Consequently, due to the sensitivity of the trench stability issue and propagation of cracks, topographical control was conducted.

B) Topographical control

The use of total station surveying instruments for monitoring the movement of structures with good results have been reported by many authors, such as Radovanovic and Teskey in 2001, Hill and Sippel in 2002, Kuhlmann and Glaser in 2002, as well as Zahariadis and Tsakiri in 2006 [16, 17, 18, 19].

Pins and control points fixed into the potentially unstable ground were surveyed. A total of nine bench marks were installed at the various points of the trench (Fig. 7) and monitoring of these points were inserted at the work plan of the surveying unit.

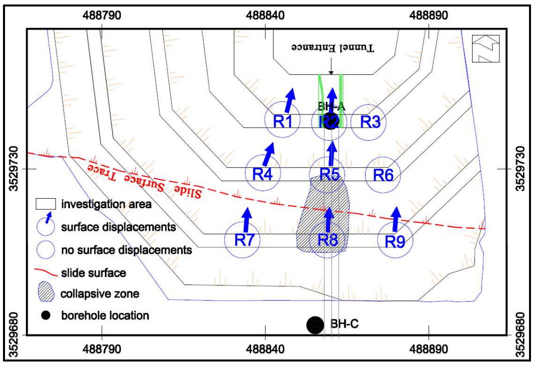


Fig. 7. The location of the monitoring station and overall tendency of displacement along X, Y and Z [8].

The preliminary estimation of the displacement direction and assessment of the trench stability can be identified as the two main objectives for continuous trench monitoring. After data acquisition, it was determined that the pins at the seven stations had moved in one direction of the X, Y, Z directions from north to northeast, but there was no movement in the R3 and R6 stations (Fig. 8).

The soil of some parts of the trench consists of a certain clay type, thus the swelling is predictable as a result of increasing saturation of the soil due to groundwater and rain-fall that caused soil uplifting.

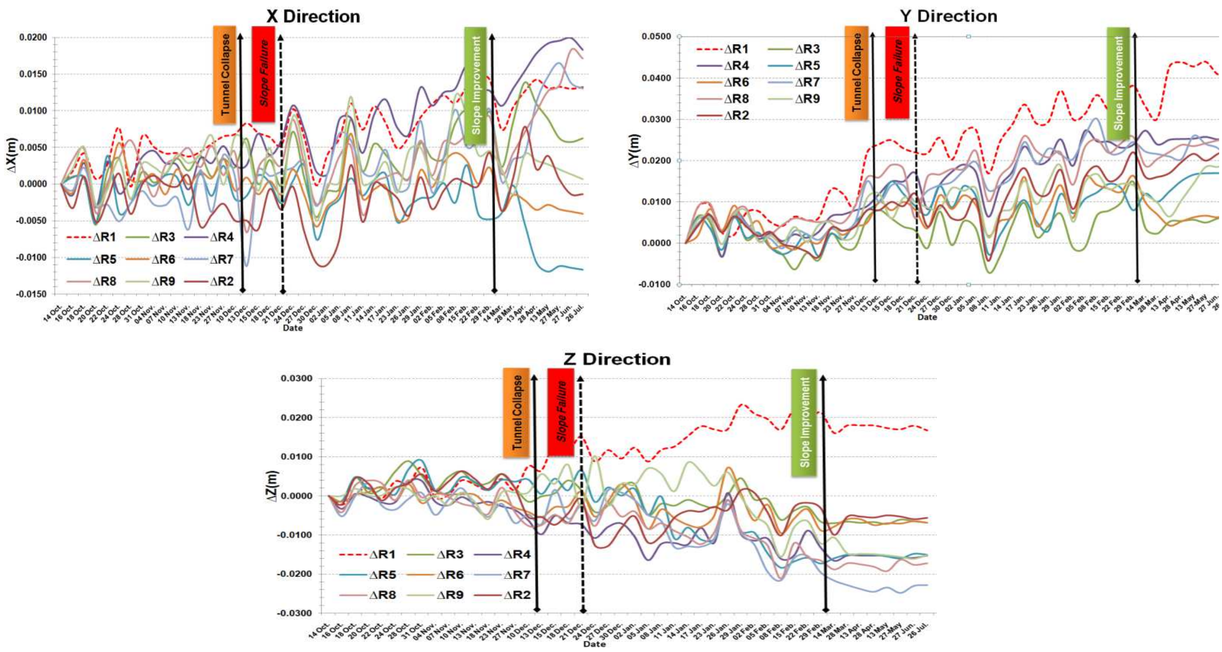


Fig 8. The displacement graphs of the installed pins along X, Y, Z. As observed in the Z direction despite the trench subsidence, R1 station was moved upward. The soil at this location contains montmorillonite, which has the capability of swelling (Modified [8]).

4. Process of the trench collapse

The displacement of installed pins was recorded with millimetre precision before collapsing. The first signs of sliding were observed after the initiation of tunnel excavation from 35 metres, where four metres of the tunnel crown collapsed in eight hours and 1500 m^3 of soil collapsed six times in one month (Fig. 9). The monitoring program was simultaneously conducted in shorter, yet, more accurate periods. The results showed that an increasing movement of the trench ultimately caused the failure of the concrete slabs.

It is obvious that bad engineering judgment in the design of trench parametres will occur as a result of inaccurate field investigations.

The high tunnel advance rate, stresses releasing, existence of aqueous soils and layers with high degree angles caused mass soil sliding on the slide surface. Figure 10 offers a flowchart representation of the collapsing factors for this trench.

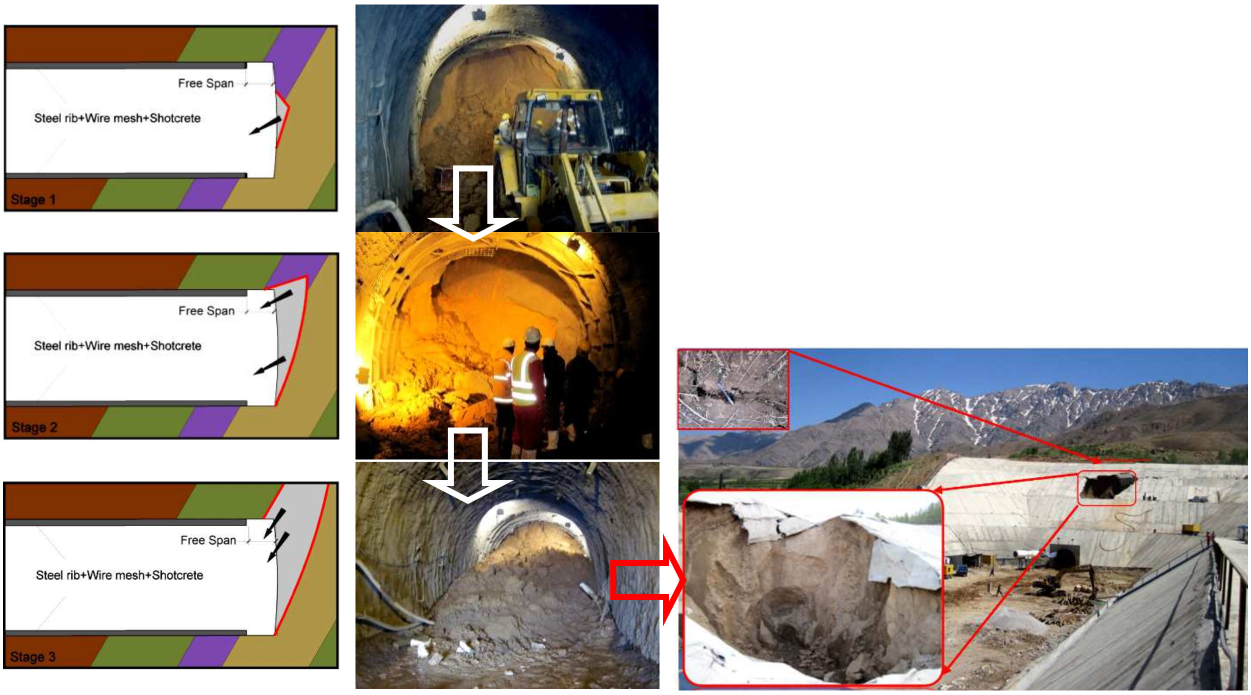


Fig. 9. Stages of Trench failure [7].

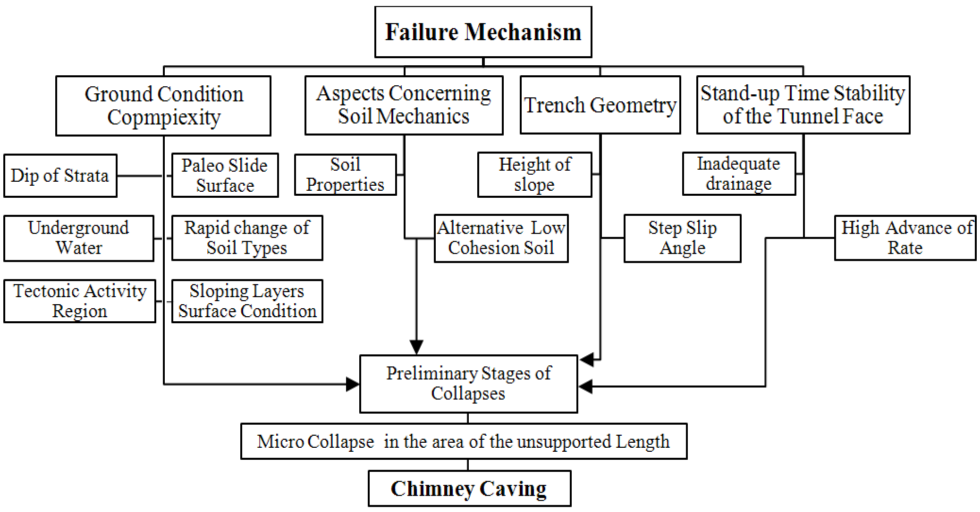


Fig. 10. Flowchart of the trench failure mechanism [8].

5. Trench stability reanalysis

According to the results obtained from geological and geotechnical studies as noted earlier, geometric design and the trench stability were reviewed and reanalysed based on our new studies.

Evidently, the slope stability analysis is based on simplified assumptions; consequently slope designing depends heavily on field tests and investigations.

Determining soil properties is the key factor in stability analysis. On the other hand, soil properties vary spatially even within homogeneous layers due to depositional and post-depositional processes that cause variation in properties [20]. Nevertheless, most geotechnical analyses adopt a deterministic approach based on single soil parameters applied to each distinct layer. The conventional tools for dealing with ground heterogeneity in the field of geotechnical engineering have been applied under the use of safety factors and by implementing local experience and engineering judgment [21]. Calculations are performed based on the safety factors for unstable (FS $\leq$ 1), boundary (FS =1) and stable (FS> 1) states, in most analyses, though with little attention to the precision of soil properties and the geometric of the layers.

Despite the uncertainties in the slope stability problems, geotechnical engineers still avoid adopting the probabilistic techniques in their analyses; thus, most of the design regulations such as Eurocode 7 (2004) consider conservative properties for soils and relatively large safety factors in the slope stability analyses. In recent years, slope stability analyses have widely utilized numerical models that closely approximate failure processes and more accurately represent actual landslide conditions [22].

Simplified assumptions in 2D slope stability methods have led to factors of safety that differ from the more rigorous 3D slope stability analysis methods. The 3D slope stability analysis method establishes a compatible velocity field and obtains the factor of safety by the energy and work balance equation [23].

Drawing on the above guidelines and by using modified geological profiles and new geotechnical data, a 3D numerical model of the portal of the Sabzkuh tunnel with pervious geometry was produced. The analysis methods of slope stability include the rigid body limit equilibrium method (RBLEM) [24, 25], stereographic projection method [26], finite element method (FEM) [27, 28, 29], discrete element method (DEM) [30], discontinuous deformation analysis (DDA) [31, 32, 33] and numerical manifold method (NMM) [32, 33], etc. These methods have both advantages and disadvantages [34].

Numerical modelling software packages, such as FLAC^{3D} (Fast Lagrangian Analysis of Continua, which uses the explicit finite differentiation formulation), can contribute to solving problems that consist of several stages, large displacements and strains, non-linear material behaviour and unstable systems, and even cases of yield/failure over large areas, or total collapse.

The model geometry for the analyses using FLAC^{3D} [35] is illustrated in Figure 11. The model was 50 metres wide and 81 metres long. The floor of the tunnel was located 15 metres above the base of the model and the overburden height varied between three metres to 23.5 metres. A D-shaped tunnel with a width of 5.6 metres and height of 5.51 metres was excavated to the collapsing zone. The unsupported tunnel length was one metre. For each step, the excavation was advanced by one metre and the pre-support shotcrete (25 cm thick with two layers of welded wire mesh $\phi 8$) and steel arc support (2IPE140) were installed at the specified distances from the tunnel face.

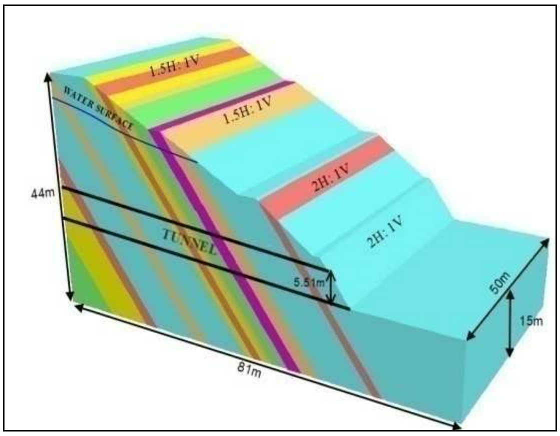


Fig. 11. The model geometry based on geology profile for the analyses [8].

The tunnel was constructed in weak soil conditions that are modelled as Mohr-Coulomb materials with properties given in Table 1. The initial stress state corresponds to gravitational loading with the following relation between vertical and horizontal stresses: $2\sigma_{zz} = \sigma_{xx} = \sigma_{yy}$.

The pre-support shotcrete was modelled with zones. Note that the steel arch support was not explicitly modelled and its effect was combined with that of the shotcrete. Assuming 20GPa for modulus of elasticity and 0.2 for Poisson's ratio and using the relations presented by Hoek et al. in 2008, the equivalent modulus of elasticity for the support system would be 23.2GPa [36].

Before tunnel excavation, the factor-of-safety (FOS) was calculated using the strength reduction method. A value of 1.11 was calculated for the FOS under wet conditions, indicating that the slope was stable. However, as observed after beginning the excavation of the tunnel, displacements increased and in the vicinity of the sliding surface the maximum relative displacement was 3.358e-001m. Figure 12 plots the resulting failure surface due to tunnel excavation by the displacement contour. Before the sliding surface area, displacements were below the permitted limit. Installation of a support system, as soon as possible, leads to preventing surface displacement during the excavation stage. However, the trench collapsed near the sliding surface before the support system was installed. The velocity vectors calculated during the excavation stage and the maximum values are in the range of 7.823e-007 ms^{-1}. These values indicate that the soil collapsed in wet conditions. Furthermore, field observations confirmed that this phenomenon may occur.

Figure 13 plots shear strain-rate contours, which allow the failure surface to be identified.

In essence, the approach assumes that field data (such as in-situ stresses, material properties and geological features) will never be completely known. It is futile to expect the model to provide design data, such as expected displacements, when there is massive uncertainty in the input data. However, a numerical model is still useful in providing a picture of the mechanisms that may occur in particular physical systems, but as it can be observed, if extensive field data are available, then these may be incorporated into a comprehensive model that can yield design information directly.

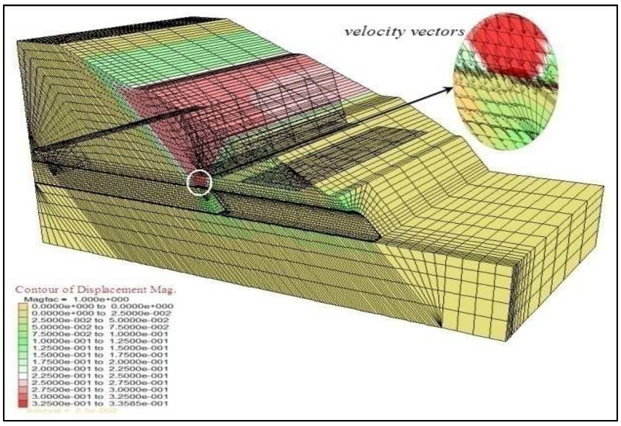


Fig. 12. Displacement contours and velocity vectors in the model at the failure state [8].

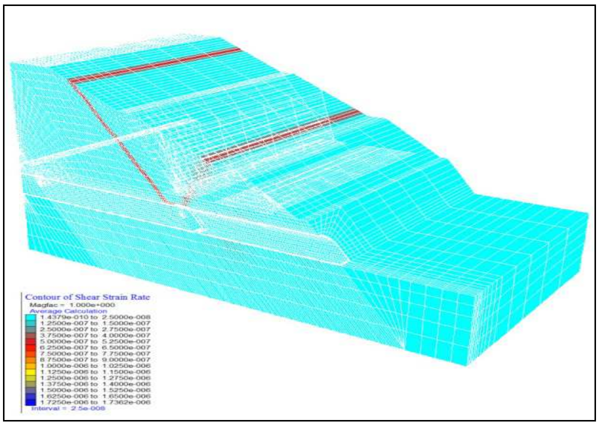


Fig. 13. Shear strain-rate contours in slope model at the last non-equilibrium state [8].

6. Slope management

In any economic slope, a variety of instabilities may be present at various locations in the slope at any time. A skilled geotectonic engineer should be able to manage these features successfully. The absence of any failures is a sign of over-conservative slope design, and hence, inefficient project management. It is, therefore, an absolute requirement that engineering geologists and geotechnical engineers work together all the time in order to ensure that:

1. the appropriate data are collected,
2. the appropriate analyses are carried out,

3. the slope designs are clearly conveyed to, and understood by, the mine planners and operators, and

4. well-conceived slope monitoring programmers are established to monitor the service performance of the slope throughout its life. Contingency plans must also be drawn-up to deal with the inevitable surprises that will occur from time to time.

A slope design is based upon the best possible evaluations of the soil types and characteristics, the structural geology and the groundwater conditions in the slope. Even the best slope designs require some averaging of all of this information and local variations in geology or groundwater conditions are not always incorporated into the design. These local variations can have a significant impact on slope stability and, depending on the location of this instability, may have important consequences for the performance of the slope. Advance warning of these slope instability problems is very important and monitoring of slope movement has proved to be the most reliable method for detecting slope instability. The more accurate this measurement the earlier the developing problem can be detected.

Tools for slope displacement monitoring are well developed and are used routinely on many civil engineering projects in which large slopes exist or are being excavated. These are generally based upon observations on numerous targets placed at carefully selected locations on the benches of the slope.

Simple visual observation by geologists and geotechnical engineers is a tool that is frequently ignored. The development of tension cracks and the appearance of bench faces is due to slope instability. If these are observed routinely and recorded systematically, a ‘feel’ for the behaviour of the slopes can gradually be developed. This is important information that can be taken into account when signs of significant slope instability appear and when discussions on remedial measures and contingency planning take place [37].

7. Conclusions

This study employed the three methods of field investigation, monitoring and numerical modelling. The field survey, monitoring and numerical simulation results revealed landslide failure mechanisms, the area of the potentially unstable slope and sliding depth, which otherwise are difficult to obtain. The conclusions are as follows:

1. using field investigation to estimate the parameters of soil can compensate for the deficiencies in overall representativeness of mechanical tests, and

2. numerical simulation analysis verification of the history of the disaster area and estimated results can yield conservative calculations of the potential landslide area. Numerical simulation analysis can also simulate the behaviour of the landslide mass sliding through soil, which can be delineated according to the scope and depth of the high potential landslide.

In analyses and the design process of the portal of the Sabzkuh tunnel, a geotechnical model was planned, performed and analysed independently, regardless of a geological model. A lack of correlation between these two models, as well as the complicated ground situation and uncertainties provided some inappropriate information for designers and affected the analytical model. This problem caused a failure in the tunnel portal area.

This case study demonstrates once again the importance of early detection or prediction of potentially problematic zones (via probe drilling and monitoring) in slope stability, particularly through mixed or difficult ground conditions characterized by alternating layers, faulting and landslides. As a result of the inherent uncertainty in the slope design, the presence of experienced geologists and geotechnical engineers who can predict and deal with such instability problems effectively and promptly guarantees the successful completion of civil engineering projects.

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