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A Numerical Investigation of Sinkhole Subsidence Development over Shallow Excavations in Tectonised Weak Rocks: The Dolaei Tunnel's Excavation Case

Ebrahim Fathi Salmi · Majidreza Nazem · Anna Giacomini

Abstract Roof collapses during the constructions of underground excavations in weak rocks is a serious problem. In particular, excavations in shallow depths and in incompetent rocks may initiate the caving of the overburden materials and sinkhole formation. Sinkholes have significant environmental impacts and more importantly, they threaten the stability of surface and subsurface infrastructures above the excavations. This study investigates the formation of sinkhole subsidences in shallow excavations in poor and problematic rocks. The progressive collapse and sinkhole subsidence in the Dolaei road tunnel is considered as a case history. Understanding the geological and geotechnical characteristics of rocks is the fundamental step for analysing the mechanisms of instability. Rock mass characteristics are reviewed, and the most effective factors impacting on the tunnel's stability are identified and discussed. The role of the method of excavation and support in controlling ground movements are assessed through numerical modellings. The effect of pre-support as a practical technique for controlling ground movements and preventing sinkhole formation in weak rocks is also discussed. Outcomes of this study indicate that the rock mass surrounding the Dolaei tunnel consists of highly tectonised and foliated metamorphic rocks. Schistosity and foliation considerably impact on the strength and deformability of the rock mass. This study shows that the geological characteristics of the rocks in the Dolaei tunnel had substantial effect on the collapse and sinkhole formation. The numerical findings also reveal that employing pre-supporting techniques, particularly forepoling, using a staged excavation and applying composite support systems (consisting of rock bolts and reinforced shotcrete) are practical remedies to prevent the progressive collapse, and to avoid the formation of sinkholes during excavation in weak rocks in shallow depths.

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1 Introduction

The analysis of the mechanisms of ground movement and subsidence formation over civil and mining excavations is a very challenging task. A precise estimation of subsidence is usually required for selecting an appropriate mitigation system to control and reduce the undesirable excavation impacts (Bell

et al. 1988; Ghorbani et al. 2012; Karakus and Fowell 2006; Selby 1999; Wittke 1988). The magnitude and duration of a subsidence depends on numerous factors, such as the geometry of the underground opening, the method of excavation, the overburden thickness, the types and mechanical characteristics of the surrounding ground (Bell et al. 1988; Karakus and Fowell 2003, 2006). Generally, excavations modify the in situ stress conditions as well as the hydrogeology of the area with consequent movements of the surrounding rocks and soils towards the excavation. Particularly, the excavation of a shallow tunnel in weak ground results in the formation of a failure zone behind the tunnel face, which can lead to significant surface subsidence (Bell et al. 1988; Peila 1994). In very soft ground, the failure zone may propagate towards the ground ahead of the tunnel face which usually results in sinkhole subsidence (Bell et al. 1988; Jacobs 1975).

Although numerous studies have been performed to predict the magnitude of surface subsidences caused by tunnelling, underground mining activities, or even fluids withdrawal (Karakus 2007; Kratzsch 1983; Schina and Charalampidou 2014; Selby 1999; Whittaker and Reddish 1989), there are only a few studies about the mechanisms of sinkhole formation over shallow excavations (Karfakis 1987; Oliveira 2009a, b; Piggott and Eynon 1978; Singh and Dhar 1997; Whittaker and Reddish 1989). In this study, the formation of sinkhole subsidence in the Dolaei tunnel in weak metamorphic rocks is investigated as a case study. The main geological and geomechanical characteristics of the surrounding rocks are reviewed in order to estimate the rock mass properties and their mechanical behaviour. Then, the obtained information is used to find a practical method to prevent the sinkhole formation. This study shows that a rock mass mechanical behaviour plays a substantial role in the formation of sinkholes over excavations.

Designing a proper method of excavation and support systems is a useful technique to reduce the tunnelling induced ground movements. Different empirical approaches, mainly based on rock mass classification methods, are used in this study for designing the primary method of excavation and selecting the appropriate support measures. The proposed design aims to reduce and control the ground movements and prevent sinkhole formation. Moreover, the role of the method of excavation and support in controlling the ground movements are assessed through the numerical modelling. Two-dimensional (2D), as well as three-dimensional (3D), numerical simulations are conducted to study the mechanisms of instability and sinkhole development. Numerical modelling is also used to modify the primary design of the support measures. Additionally, the role of the method of excavation and support on preventing the sinkhole subsidence is investigated through the numerical modelling. The effects of the pre-support technique on stabilising the tunnel and decreasing the support pressure in fault zones is also discussed and analysed.

2 Case Study

The Dolaei tunnel represents a unique example of tunnelling in difficult ground conditions which has led to the formation of a very large sinkhole subsidence. The tunnel was mainly constructed to facilitate transportation between Touyserkan and Hamadan in Iran. The Dolaei tunnel is located at 5 kilometres south-east of Touyserkan, is 675 m long and has a cross-section area of ~ 72 m^2. The tunnel span is approximately 10 m and its height is about 8 m (Ghasemi et al. 2007; Nazeri et al. 2004). The maximum overburden of the tunnel is also around 90 m.

Tunnelling was performed from both north and south portals. Conventional drilling and blasting method was used to construct the tunnel. The length of a single excavation round depended on the blasting performance and varied in different geological and geotechnical conditions. The average single excavation round in good rock masses in the Dolaei tunnel was around 1.5–2.0 m. However, tunnelling through the fault zones was very difficult and imposed a relatively short excavation round and an extremely low rate of excavation. Figure 1 shows the tunnel location and a view of the sinkhole. Although the tunnel is relatively short, its excavation was very challenging due to the very difficult ground conditions (Ghasemi et al. 2007; Nazeri et al. 2004). The contact metamorphic rock mass surrounding the tunnel (see Sect. 3) can be defined as "weak–very weak", and it includes intersecting fault zones and weak planes that considerably impacted on the stability of the tunnel (see Fig. 2).

During a progressive roof collapse in 2000, approximately 290,000 m^3 of broken rock and fault gouge

(A)

(B)

(C)
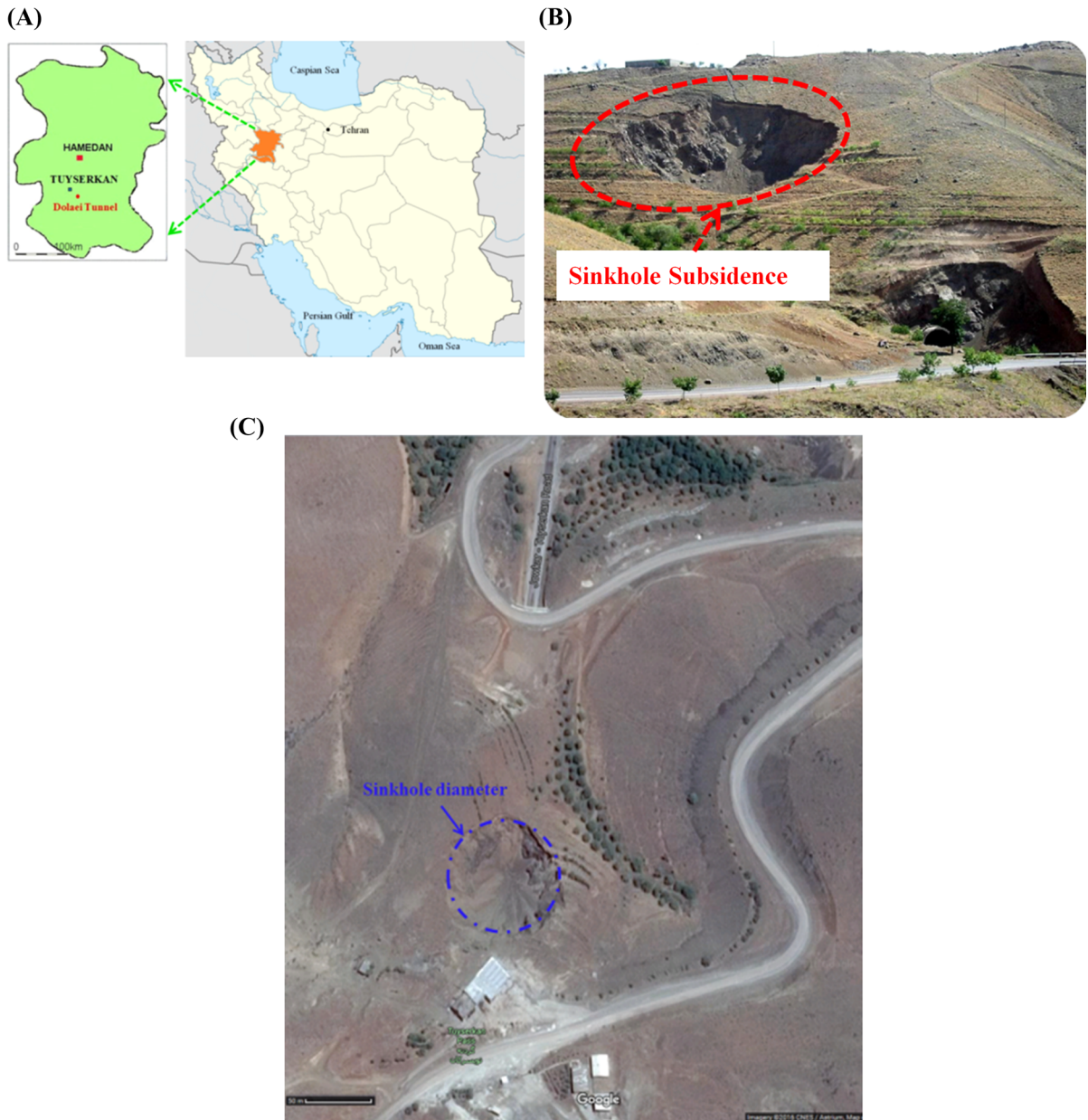
Fig. 1 **a** The location of the Dolaei tunnel, **b** and **c** views of the sinkhole subsidence (Google Map 2016)
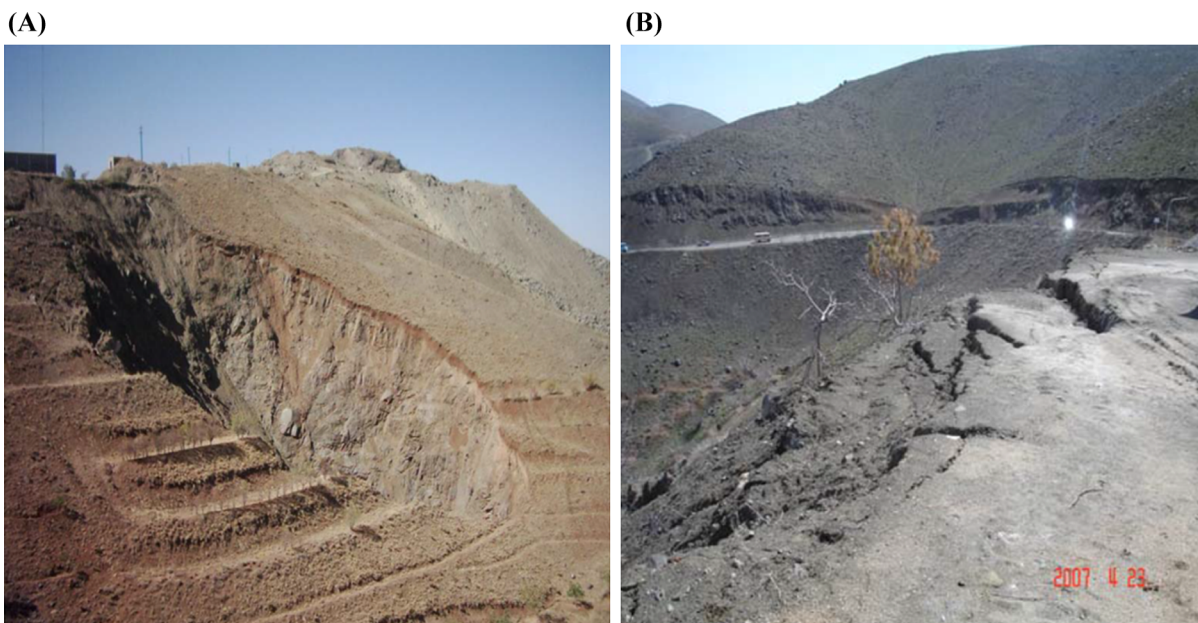
caved into the tunnel, and formed a huge sinkhole above the tunnel (Ghasemi et al. 2007; Nazeri et al. 2004). The sinkhole crater was huge and can be easily observed in satellite images, as depicted in Fig. 1c. The average overburden height in the sinkhole area was around 50 m. However, the inclined topography and the progressive caving led to a slope failure (see Fig. 3). On the other side, rock masses in the area were highly tectonised and weathered with a very low

repose angle. Therefore, the dimension of the sinkhole dramatically increased and its diameter at the surface exceeded 50 m.

The bulking of the caved materials played a pronounced role in the upward expansion of a cavity and the sinkhole's formation, as the caved debris may chock the cavity and halt the upward void migration (Bell and De Bruyn 1999; Piggott and Eynon 1978; Whittaker and Reddish 1989). The low durability of

North Portal   South Portal

F1, F2, F3 and F4 are the main discontinuities and faults
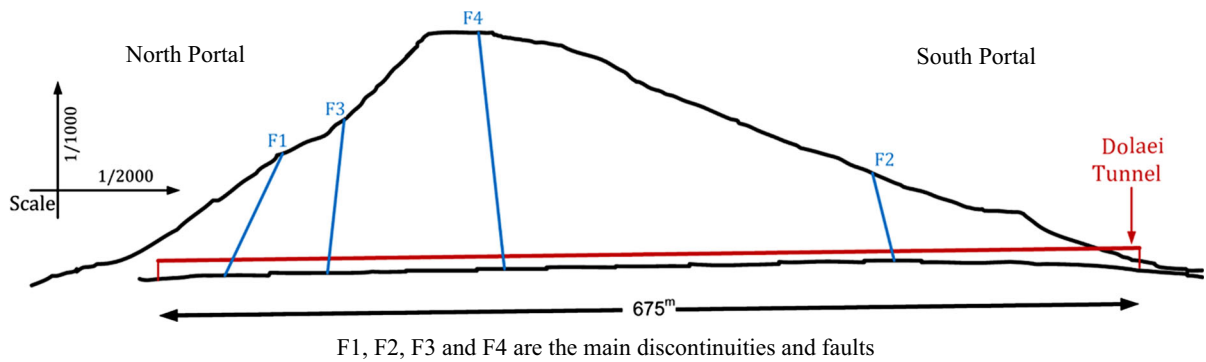
Fig. 2 Schematic section of the Dolaei road tunnel. Adopted from Ghasemi et al. (2007)

(A)   **(B)**

Fig. 3 Subsidence induced slope failure above the Dolaei tunnel. Adopted from Ghasemi et al. (2007)

the caved materials is also reducing the bulking factor and increasing the potential of sinkhole formation (Cole and Figg 1987). To construct the Dolaei tunnel, the collapsed debris had to be withdrawn from the excavation. The void, therefore, migrated upward to the surface and formed a huge sinkhole above the tunnel (see Fig. 4).

The collapse of the Dolaei tunnel not only damaged the supporting structure and the tunnelling equipment, but imposed significant financial costs associated with the withdrawing of the debris and the infilling of the generated sinkhole. Additionally, the collapse treatment was extremely difficult, and it caused almost 2 years of delay in the construction plan, as reported

by Ghasemi et al. (2007). The total construction finally took over eight years (Ghasemi et al. 2007). Medium and heavy steel ribs (IP 16 and 18, 2 m spacing), shotcrete (15 cm) and 5 m long rock bolts (1.5 × 1.5 m spacing) were used as the temporary support measures to control the tunnelling induced ground movements. To ensure the long-term stability of the tunnel, an in situ reinforced concrete lining was also used as the permanent support. Utilising heavy supports was very time consuming and expensive. It is also notable that stabilising the tunnel face in fault zones was also difficult. To prevent progressive failure of the face and tunnel crown, rock bolts were employed in the tunnel (Haghshenas et al. 2013;

Jalilvand and Haghshenas 2013; Jalilvand et al. 2014). Figure 5 shows the permanent support measures in the Dolaei tunnel.
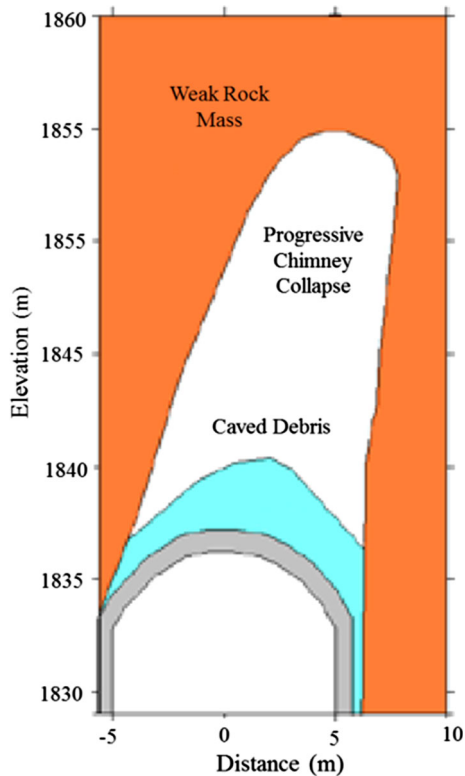
Fig. 4 Schematic upward expansion of the cavity above the Dolaei tunnel. Modified from IRIMPO (2016)

3 The Geological and Structural Settings of the Dolaei Tunnel

The geological and geotechnical characteristics of a tunnel must be scrupulously investigated before the design and analysis of underground excavations. This is because the mechanical characteristics of rocks and discontinuities are among the key factors controlling an excavation responses (Aydan et al. 2014; Bieniawski 1989; Hoek 2007).

To the best of the authors' knowledge there is no evidence of any detailed site investigations, prior to the excavation of the Dolaei tunnel, for the route selection in order to avoid facing fault zones and complex ground conditions. This lack of information was partially responsible for the unexpected difficulties faced during the tunnelling operation. The misinterpretation of the field characteristics and an overestimation of the strength of rock mass have been among the main suspects for the collapse and instability in this project.

It is notable that some research was carried out after the occurrence of sinkhole subsidence to investigate the problem in the Dolaei tunnel. In particular, the geological features of the tunnel area were studied and reported in the relevant literature (Ghasemi 2008; Ghasemi et al. 2007; Jalilvand et al. 2014; Nazeri et al. 2004; Sharifi et al. 2011; Yousefi Rad et al. 2013). However, the relations between the geotechnical characteristics of the rocks and the sinkhole formation have not been well addressed or clarified in any of

(A)   **(B)**

Fig. 5 The permanent lining of the tunnel. Adopted from T.S. Courtesy of Aria (2016)

these studies. Additionally, the roles of effective factors, such as tunnelling method, support system and pre-supporting, on controlling ground movements were ignored in the previous studies. It seems, therefore, essential to meticulously assess the geotechnical characteristics of the rock mass around the tunnel and their effects on the mechanisms of instability. The rock mass's mechanical characteristics are used as the main input parameters when performing the detailed numerical investigations. The results of this stage, therefore, directly impact on the outcomes of the subsequent numerical analysis.

The area of the tunnel possesses a wealth of geological and geotechnical data, mainly because of the construction of several roads as well as Sarabi Dam in this region (Ghiasvand 1999; Sharifi et al. 2011; Uromeihy and Ghiasvand 1999). In particular, the geological and geotechnical characteristics of the igneous and metamorphic rocks in Hamadan province have been well investigated and documented in the relevant references (Fereidooni et al. 2015; Ghiasvand 1999; Khanlari et al. 2014a, b). Unfortunately, these studies were overlooked in the design of the tunnel as majority of them were conducted after starting the tunnelling project. The authors will, however, use and combine the available previous studies here to assess the geotechnical characteristics of rocks in the Dolaei tunnel area. The geotechnical data are then used to derive the mechanical properties of rock masses.

3.1 Geological Characteristics of the Tunnel Area

The Dolaei tunnel is located in the Sanandaj–Sirjan tectonic zone, which occupies the area between the main Zagros trust in the southwest and the Urumiyeh–Bakhtar volcanic belt in the northeast (Koleini 2012). The mineralogy and petrology of the metamorphic rocks of Hamadan province, where the Dolaei tunnel is located, have been very well studied and reported in the related references (Fereidooni et al. 2015; Ghiasvand 1999; Khanlari et al. 2014a; Sharifi et al. 2011). Based on these investigations, contact metamorphic rocks (e.g., garnet schist, andalusite schist, staurolite schist, andalusite–sillimanite schist, andalusite–staurolite schist, amphibolites schist and hornfels schists) are the dominant rocks in the area (Ghasemi 2008; Ghasemi et al. 2007; Ghiasvand 1999; Sharifi et al.

2011; Uromeihy and Ghiasvand 1999). The rock masses in this region have been affected by the Alvand Batholith. In other words, the pressure and temperature of this batholith has directly controlled the level of the metamorphism (Ghasemi et al. 2007; Sharifi et al. 2011). Schistosity and foliation is, therefore, one of the dominant characteristics of the rock mass in the tunnel area (see Fig. 6). This shows that the rock mass's behaviour represents a high level of directional dependency (Fereidooni et al. 2015; Ghasemi et al. 2007; Khanlari et al. 2014a; Nazeri et al. 2004).

Lamination and schistosity are among the main characteristics of sedimentary and metamorphic rocks. These special characteristics can considerably impact on the mechanical parameters of rocks including their stiffness and strength (Fereidooni et al. 2015; Khanlari et al. 2014a; Saroglou et al. 2004a, b). In particular, several studies were conducted to investigate the role of schistosity in the mechanical behaviours of similar metamorphic rocks in the area (Fereidooni et al. 2015; Ghasemi 2008; Ghiasvand 1999; Khanlari et al. 2014a, b; Uromeihy and Ghiasvand 1999). The overall results indicate that the texture of the rocks is highly anisotropic and foliated (see Fig. 6). The laboratory results also showed that the maximum strength of the rock is achieved when the loading direction is perpendicular to the schistosity plane, while the minimum strength is obtained when the angle between the direction of the loading and the schistosity is around 30° (Fereidooni et al. 2015; Ghiasvand 1999; Khanlari et al. 2014a, b; Uromeihy and Ghiasvand 1999).

The laboratory findings also revealed that the level of metamorphism, particularly the types and amounts of minerals generated during the process of metamorphism, had affected the physical and mechanical properties of the rock specimens (Ghiasvand 1999; Ghiasvand and Uromeihy 2010; Khanlari et al. 2014a; Sharifi et al. 2011; Uromeihy and Ghiasvand 1999). Generally speaking, rocks that have undergone a complete process of metamorphism, or experienced re-metamorphism, are stronger than those that have been partially affected by metamorphism. Additionally, the laboratory studies indicated that the existence of water significantly decreased the strength of the anisotropic rocks in the area of the study (Fereidooni et al. 2015; Ghiasvand 1999; Ghiasvand and Uromeihy 2010; Khanlari et al. 2014a).

Fig. 6 Rock mass schistosity in the tunnel area. Courtesy of Ebadi (2011) and Ghasemi et al. (2007)

3.2 Characteristics of Faults and Fault Zones

Field observations (Ghasemi et al. 2007; Nazeri et al. 2004) indicated that the Dolaei tunnel intersects four faults (see Fig. 2). Faulted zones are complex structures that exhibit heterogeneous geotechnical conditions. Large deformations and instability problems, leakages and the rush of underground water and broken materials toward the excavation are normally expected during tunnelling in faulted rocks (Dalgıç 2003; Hoek 2007; Kimura et al. 1987; Palmström 1995; Schubert 2008). The structures and compositions of fault zones are important parameters in the stability of tunnels (Dalgıç 2003; Schubert 2008; Schubert et al. 2006).

Tunnelling in fault zones is usually challenging, and is preferably avoided. This means that the most appropriate location, where intersections with fault zones is the least probable, needs to be identified for constructing underground structures (Schubert et al. 2006). Unfortunately, no detailed field investigation was performed prior the tunnelling operation to find the location of any faults in the area of the Dolaei tunnel. Therefore, the presence of the fault zones is taken into account in the present study and will be discussed in further details in Sect. 7.5. The presence of faults is also considered with regards to the observed issues caused by the presence of water flows into the tunnel face during the tunnel's construction. The intersecting faults represent preferential path for water flow and, therefore, are critical points in the tunnel construction. The potential for water rushing in such locations strongly contributes to the decrease of the rock mass's strength by reducing the strength of the intact rock materials, as well as the shear strength of the discontinuities (Aydan et al. 2013; Vásárhelyi and Ván 2006).

Being exposed to humidity and variations of temperature caused intensive physical and chemical weathering which highly impacted on the strength of the rocks in the tunnel area. Therefore, the contribution of deterioration to the tunnel's collapse and sinkhole subsidence is also considered in this study. The intense weathering led to the alteration of minerals, the disruption of rock skeletons and microcrack augmentation, with subsequent gradual degradations of the properties of the rock masses surrounding the tunnel. The effects of weathering on rock mass characteristics has been addressed in the relevant references (Karpuz and Pasamehmetoglu 1997; Pasamehmetoglu et al. 1981).

In terms of geotechnical characteristics, fault zones in the tunnel area consisted of heterogeneous materials (see Fig. 7). In particular, the materials in the shear zone of fault in the Northern part, F1 and F3 were brecciated and clayey. However, some blocky

structures were also observed during tunnelling in fault zones (e.g., F2) which were formed due to discontinuities intersection zones. A series of geophysical tests (seismic reflection method, e.g., TSP) were performed to determine the geological structure of the sub-surface and to identify the location and distribution of faults and fractured zones around the tunnel (IRIMPO 2016; Sharifi et al. 2011). Figure 8 shows the results of the geophysical tests in the tunnel. The tests revealed that the rock mass in the intersection of tunnel and fault F3 was severely broken. The thickness of the encountered fault zone exceeds 20 m in this area (IRIMPO 2016; Sharifi et al. 2011). Tunnelling in this zone was extremely challenging and associated with excessive overbreak and deformation, squeezing of the surrounding rocks, and ingress of

groundwater and gouge materials into the tunnel. These eventually led to the failure of the support measures and collapse of the tunnel's crown and face. Withdrawing the fault gouge materials and broken rocks led to the progressive upward movement of the cavity and the formation of the sinkhole (Ghasemi 2008; Ghasemi et al. 2007; Sharifi et al. 2011).

4 Rock Mass Characterisation

Rock mass characterisation and the determination of the mechanical characteristics of rocks and discontinuities that contribute to the stability of an excavation are fundamental steps in rock engineering projects (Barton 2002; Singh and Goel 2011). Experimental

(A) **(B)**

(C)
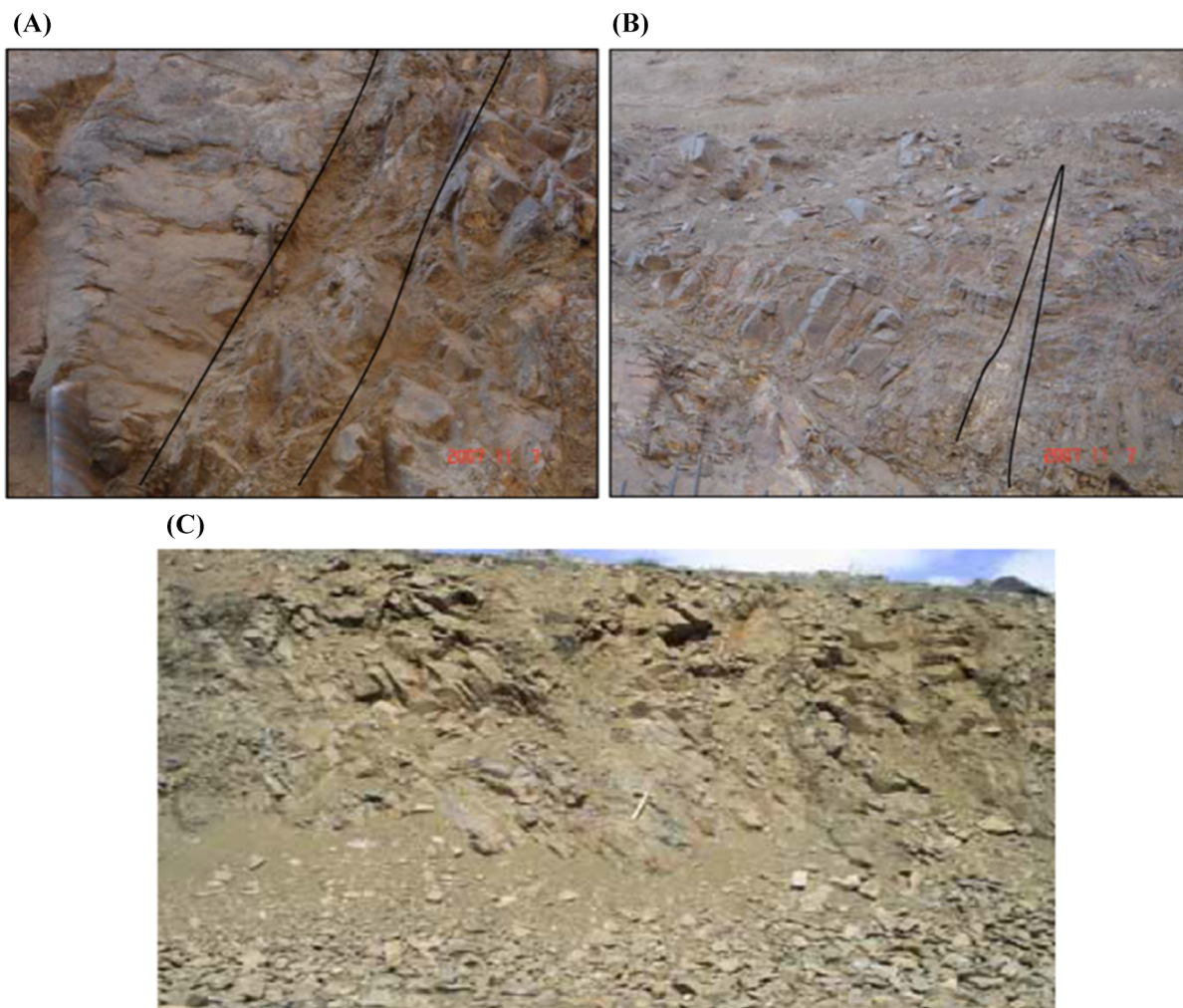
Fig. 7 Crushed hornfels-schists in the fault zones around the tunnel. Courtesy of Ghasemi et al. (2007) and Sharifi et al. (2011)
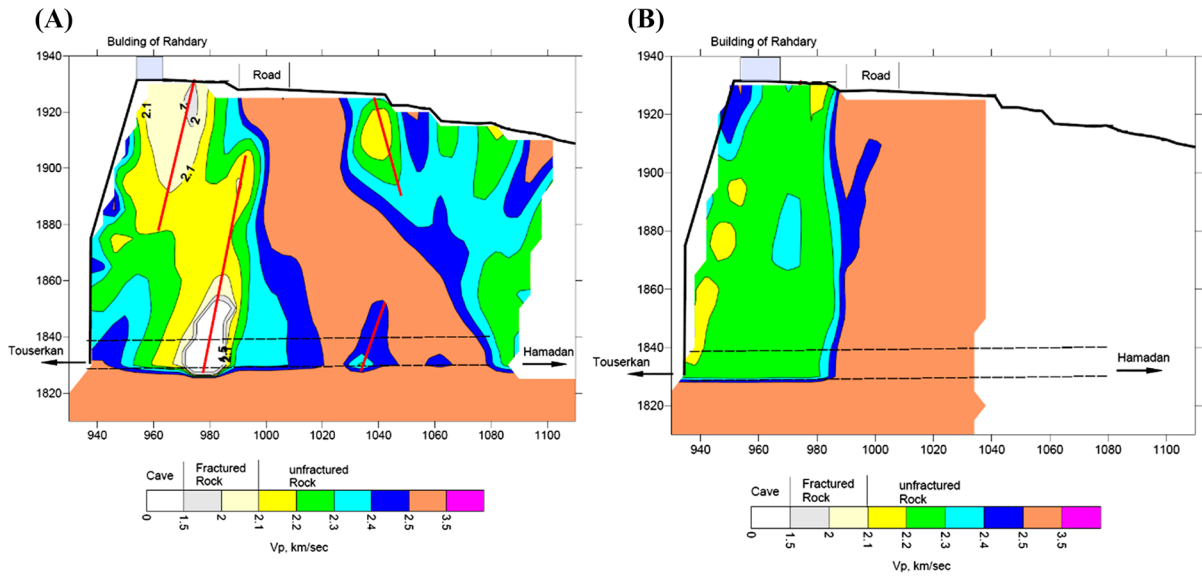
Fig. 8 The outcomes of geophysical tests (seismic wave propagation) in **a** *left* and **b** *right* walls of the Dolaei tunne

laboratory tests are normally employed for assessing the mechanical properties of intact rocks and discontinuities. However, due to scale effects, as well as the anisotropic characteristics of rocks, approximating the mechanical behaviours of rock masses is an extremely challenging task (Barton 2002). Therefore, alternative approaches, such as in situ tests or rock mass classification methods, are used to estimate the mechanical parameters of the rock mass. However, conducting in situ tests is usually difficult, expensive, and time consuming (Aydan et al. 2013; Bieniawski 1989). Thus, in situ tests are rarely used in typical rock engineering projects. In contrast, several empirical relations have been proposed for estimating the properties of a rock mass based on rock mass classification methods. Although employing such relations can facilitate the process of deriving rock mass properties, they may involve a considerable level of uncertainty in the approximation, since the generalisation of these empirical equations has led to complex relations. In other words, any uncertainty in the input parameters may lead to a huge levels of uncertainty in the output parameters (Ván and Vásárhelyi 2013). It is also notable that some of the empirical relations have been proposed for local and specific geological conditions. Therefore, they cannot be generally applied (Hoek 2007). In this section the procedure employed for deriving the rock mass characteristics in the Dolaei tunnel is briefly described.

4.1 Intact Rock Properties

The mechanical behaviours of the metamorphic rocks in the tunnel area have been very well investigated and documented in the relevant literature (Fereidooni et al. 2015; Ghasemi 2008; Ghasemi et al. 2007; Ghiasvand 1999; Khanlari et al. 2014a, b; Nazeri et al. 2004; Uromeihy and Ghiasvand 1999). According to these studies, the principal characteristics of the dominant metamorphic rocks of the region, the Young's modulus E_i (GPa), uniaxial compressive strength σ_{ci} (MPa), cohesion c_i (MPa), and the internal friction angle φ_i (°) are summarised in Table 1. Since rocks in the tunnel area are schistose and foliated, their mechanical properties are functions of β, the relative angle between the direction of the loading and the planes of schistosity (Fereidooni et al. 2015; Khanlari et al. 2014a). The values of the intact rock's properties are, therefore, given with respect to β. The rock density is also around 2600 (kg/m^3).

Table 1 Mechanical parameters of schistose rocks surrounding the tunnel

β (°)	E_i (GPa)	σ_{ci} (MPa)	c_i (MPa)	φ_i (°)
0	25	48	9.9	45
30	12	35	7.9	41
60	15	40	8.7	43
90	20	50	9.8	47

4.2 Characteristics of Joints

The strike of the Dolaei tunnel is almost in a North–South direction. Field observations indicate that there are three dominant joint sets in the tunnel area. Therefore, the excavation direction was with dip in one side and against the dip in the other side. The main joint set properties reported by Nazeri et al. (2004) and Ghasemi et al. (2007) are shown in Table 2.

4.3 Rock Mass Classification

Rock mass classification methods are widely used for the design and stability analysis of underground structures. One of the most important advantages of employing rock mass classifications is that they can provide a quantitative description of rock masses based on their qualitative characteristics (Bieniawski 1989). The Geological Strength Index (GSI) (Hoek et al. 1995) is one of the most convenient and accurate rock mass classification systems widely used for estimating a rock mass's properties. Other rock mass classification methods usually contain some parameters that do not, or rarely, contribute to the mechanical behaviours of rock masses (Marinos et al. 2005).

Moreover, some of these conventional rock mass classification methods are not reliable enough to be used in tunnelling in difficult ground conditions, such as clearly defined structural failure, squeezing and swelling, slabbing and rock burst (Marinos et al. 2005). Rock Quality Designation (RQD) (Deere et al. 1966) is also a fundamental parameter in some rock mass classifications, such as the Rock Mass Rating (RMR) (Bieniawski 1974) and Q-system (Barton et al. 1974). However, employing RQD may involve some limitations. One of the reasons is that the value of RQD depends on the coring direction. Furthermore, in poor rocks the RQD is essentially zero or meaningless (Marinos et al. 2005; Osgoui et al. 2010; Palmström 1995).

In contrast to the RMR and Q-system, the definition of GSI is independent of the RQD, and is solely based on the visual impression of the rock mass structure's fabric, particularly the rock blockiness and surface condition of the discontinuities which are indicated by joint roughness and alteration. This straightforward feature of the GSI has alleviated the characterisation of weak rocks (Marinos and Hoek 2001; Marinos et al. 2006; Osgoui et al. 2010). A significant amount of effort has also been devoted to enhancing and adjusting the GSI charts to accommodate the complex ground conditions in the classification systems. The heterogeneous flysh, severally weathered and tectonised rocks, and laminated rocks are some samples of such complex ground conditions (Hoek et al. 1998; Marinos and Hoek 2000; Marinos 2014; Marinos et al. 2005; Truzman 2009). The above-mentioned description of the modified GSI shows that it could be a reliable candidate for the characterisation and classification of the highly tectonised metamorphic rocks in the Dolaei tunnel.

This study has extracted the values of the rock mass GSI in the Dolaei tunnel based on the descriptive field observations reported in (Ghasemi 2008; Ghasemi et al. 2007; Nazeri et al. 2004; Sharifi et al. 2011) (see Fig. 9). Marinos and Hoek (2000), also used the GSI to characterise different schist and proposed the typical ranges of the GSI for strong, weak and sheared schists (See the dotted blue lines on Fig. 9). The estimated values for the GSIs of the rocks in the Dolaei tunnel are very well matched with the range proposed by Marinos and Hoek (2000). It is also noteworthy that a shift to the right-side of the diagram was performed based on the suggestion of Marinos et al. (2006), in order to incorporate the effects of intensive weathering, as well as the existence of water in the rock discontinuities. The value of $GSI = 33 - 37$ is, therefore, considered for the rock mass in the Dolaei tunnel.

As presented in Sect. 3, the Dolaei tunnel intersects with four fault zones. The rock masses around the fault

Table 2 Joint set characteristics (Ghasemi et al. 2007; Nazeri et al. 2004)

Joint set number	Dip direction (°)	Dip (°)	Cohesion (kPa)	Friction angle (°)	Spacing (m)
1	40	70	20–25	25–27	0.2
3	240	45	20–25	25–27	0.2
3	358	85	20–25	25–27	0.2

Fig. 9 The recommended GSI values for strong, weak and sheared schists (*dotted blue lines*). *Dotted red lines* also show the GSI values estimated for the rock mass and fault zones at the Dolaei tunnel. Courtesy of Marinos and Hoek (2000)
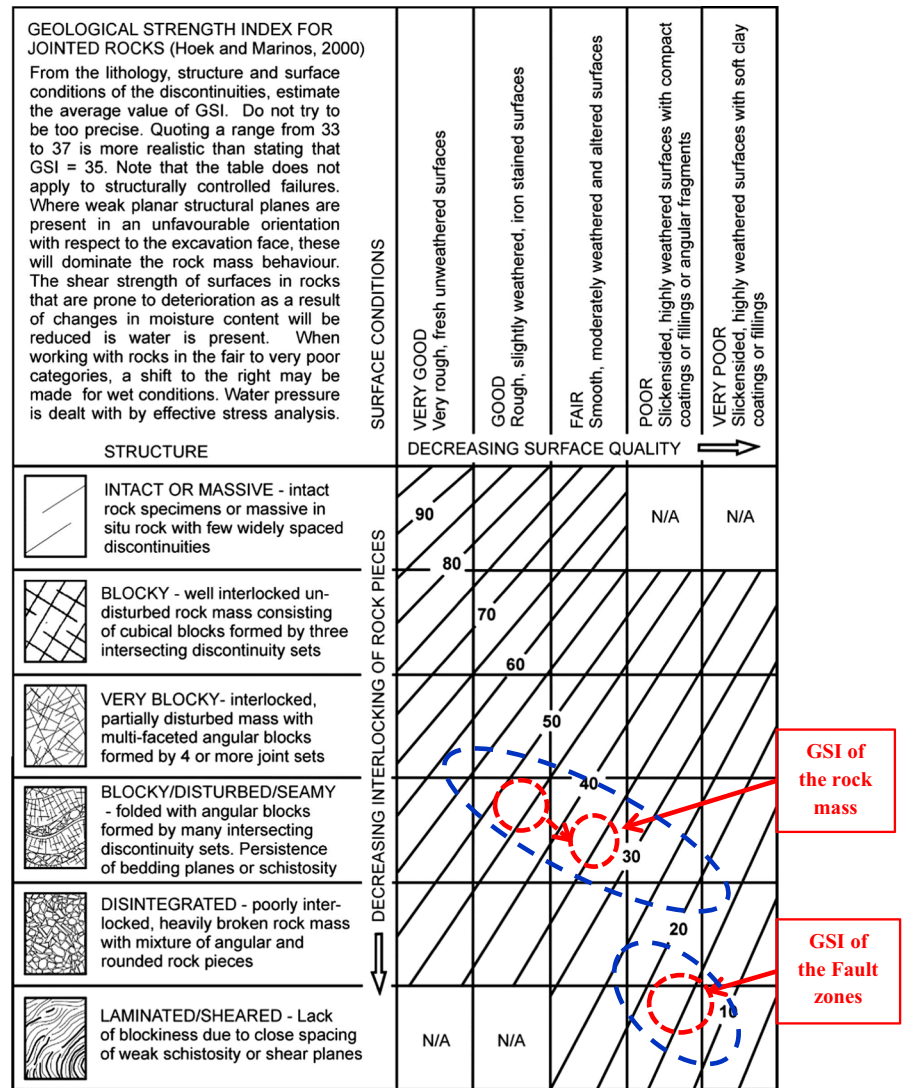
zones are extremely heterogeneous because of the tectonic stresses, the intensive weathering and the effects of the metamorphism process (see Fig. 7). The intersections of the tunnel and the fault zones are critical points for triggering a progressive collapse and sinkhole formation (Nazeri et al. 2004). In such locations, broken rocks and fault gauges can inflow into the tunnel and form upward chimneys (Hoek 2007; Marinos et al. 2006). The mechanical characteristics of the fault zones are required in order to perform the detailed analysis of the mechanisms of the sinkhole subsidence. The GSI was, therefore, used for characterising the fault shear zones (see Fig. 9). The available qualitative field descriptions of the fault

zones in the Dolaei tunnel have been used to estimate the value of the GSI (Ghasemi et al. 2007; IRIMPO 2016; Jalilvand et al. 2014; Nazeri et al. 2004; Sharifi et al. 2011). Based on this information, the GSI value of the fault zones is expected to be around 12–18.

The majority of the available and widely used relations and guidelines for estimating the primary support requirements for underground excavations are based on the RMR or Q-system. Therefore, the RMR and Q-system are also used in this study to assess the stability of the Dolaei tunnel, as well as to approximate the required support measures for stabilising the tunnel. The methodology of computing the RMR has been very well explained in the relevant references

(Bieniawski 1989). Table 3 summaries the procedure used for computing the RMR of the rock masses in the Dolaei tunnel.

The surfaces of discontinuities were mainly smooth, slickenside or slightly rough and coated with clay. Moreover, the schists and hornfelses in fault zones were heavily techtonised and weathered and had a very low quality and low core efficiency (Ghasemi et al. 2007; Sharifi et al. 2011; Yousefi Rad et al. 2013). Therefore, estimating the RMR of the fault zones based on the conventional method may involve a considerable error in the analysis. It is, however, notable that the RMR can be estimated by means of the correlating relations. This is an alternative approach for approximating the RMR in poor rocks, where the estimation of the RQD is challenging. For example, Truzman (2009) investigated the applicability of the GSI for the classification of foliated metamorphic rocks and proposed the following correlating relation between the GSI and RMR for such rocks (Truzman 2009):

$$GSI = 1.0122RMR \tag{1}$$

Equation 1 shows that the values of GSI and RMR in schistose metamorphic rocks are almost equal (Truzman 2009). Since the surrounding rock mass in the Dolaei tunnel consists of metamorphic rocks, Eq. 1 can be used for estimation of the RMR. In addition, the RMR of very weak fault zones can be approximated based on the correlating relation proposed by Osgoui and Ünal (2009):

$$RMR = 20Ln(GSI/6) \tag{2}$$

Equations 1 and 2 were used to estimate the RMR value of the surrounding rock mass and fault zones. Having in mind the average values of GSI = 35 and 15 for the rock mass and the fault zones, the RMR values for these rocks are around 35 and 18,

respectively. The computed value for the rock mass RMR based on Eq. 1 well matches with the computed value from the conventional method (see Table 3).

Furthermore, Q-value can be reckoned by (Barton et al. 1974):

$$Q = \frac{RQD}{J_n} \frac{J_r}{J_a} \frac{J_w}{SRF} \tag{3}$$

where J_n, J_r and J_w are the joint set number, joint roughness number and joint alteration number respectively. SRF is also the stress reduction factor (Barton et al. 1974). The Q-value may also be estimated by (Bieniawski 1989):

$$RMR = 9Ln(Q) + 44 \tag{4}$$

Bearing in mind Eq. 3 and considering the described characteristics of the site, RQD = 80, $J_n = 15$, $J_r = 2$, $J_a = 3.0$, $J_w = 0.5$–0.66 and $SRF = 5$, the expected Q-value of the rock mass is around 0.36. Employing Eq. 4 also results in Q-values of 0.37, and 0.056 for the rock mass and the crushed rocks in the fault zones, respectively. These values are, therefore, adopted for further analysis in this study.

4.4 Rock Mass Properties

Rock mass classification systems are commonly used to identify the mechanical characteristics of rock masses. Several empirical relations are available for predicting the mechanical parameters of rock masses based on different rock mass classification methods (Barton 2002; Bieniawski 1989; Hoek 2007). Some of the available relations can be employed for estimating rock mass parameters based on the GSI and the counterpart parameters of intact rock can be seen in the relevant references (Aydan et al. 2013; Bieniawski 1989; Hoek 2007). The Hoek–Brown failure criterion was employed in this study to compute the equivalent

Table 3 Computing the rock mass RMR

Parameter	Description	Value	Rating
σ_{ci} (MPa)	Average	35–40	4
RQD	Good	80	17
Discontinuity spacing	Close	<20 cm	8
Discontinuity condition	Slickensided wall	–	20
Groundwater condition	Flowing	–	0
Discontinuity orientations	Very unfavourable	Drive with dip and dip 45°–90°	−12
RMR$_{89}$	RMR$_{89}$ = 37		

Mohr–Coulomb parameters of the rock masses. The mechanical parameters of the rock mass at the Dolaei tunnel are shown in Table 4. These parameters are then used to perform detailed numerical analysis for assessing the mechanisms of sinkhole formation in the tunnel. In Table 4, m_i is one of the constants in the Hoek–Brown failure criterion for intact rocks, E_m, v_m, G_m, σ_{cm}, σ_{tm}, c_m, φ_m, and ψ_m are the Young's modulus, Poisson's ratio, Shear modulus, uniaxial compressive strength, tensile strength, cohesion, internal friction angle and dilation angle of the rock mass, respectively.

5 In-Situ Stresses

Prior to the excavation of underground spaces, the surrounding rocks are under an in situ stress state. The excavation induced stresses highly depend on the pre-excavation stress states. Hence, determining the initial in situ stresses is a fundamental step in the design and stability analysis of underground excavations. Over coring, hydraulic fracturing and flat jacking tests are some typical in situ tests which are usually used for determining the state of the natural stresses (Singh and Goel 2011). However, measuring the complete in situ state of stress is sometimes difficult, expensive and often a frustrating task. Due to this reason, there has been long-standing interest in the development of theories or empirical relationships that could be employed for the approximation of in situ stresses (Brown and Hoek 1978; Zang and Stephansson 2009). In practice, the in situ stress state is usually determined based on the amount of excavation overburden, the average density of the overlaying rocks and the ratio of the horizontal stress to the vertical stress, k_0. There is no general relation capable of providing a sound estimation of k_0 value for all geological conditions. There is no evidence of any sort of in situ tests in the Dolaei tunnel area to determine the pre-excavation stress state. It is, therefore, rational to accurately

estimate the in situ stresses prior to conducting the detailed numerical modellings for investigating the mechanisms of sinkhole development.

The orientation of in situ stresses is mainly controlled by the geological structures such as folds, faults and intrusions. The trend of faults can be used for estimating the in situ stress state in an area (Singh and Goel 2011). The Dolaei tunnel was excavated in heavily techtonised rocks and it encountered with some fault zones. Almost all of the faults in the tunnel area are trust faults (Behzadtabar et al. 2012; Ghasemi et al. 2007). Fault zones extended from NW of Iran to SE are among the main important tectonic structures in the country.

Thrust faults usually have mild dip with major slip along the dip direction compared to along the strike. The hanging wall is moved upward in this case. Trust faults are formed due to compressional forces. Based on the concepts of the brittle failure mechanics the vertical stress in this type of faults is the minimum principal in situ stress while the horizontal stress along the dip direction is the maximum principal stress in the area. Therefore, the order of in situ stresses in a trust fault region is $\sigma_v < \sigma_h < \sigma_H$ (Singh and Goel 2011). This general rule can be employed for estimating the in situ stress state in the Dolaei tunnel.

Sengupta (1998) also conducted several hydro-fracturing tests to measure the in situ stress state within weak rocks in the Himalayan region and proposed the following relation to approximate the values of horizontal stresses in low depth (z < 400 m) in the area:

$$\begin{aligned} \sigma_H &= 1.5 + 1.2\sigma_v \text{ MPa} \\ \sigma_h &= 1.0 + 0.5\sigma_v \text{ MPa} \end{aligned} \tag{5}$$

Iran is located in the Alpine–Himalayan collisional and seismic belt structure (Nabavi 2006). This shows that Sengupta's relations can be used for approximating the values of horizontal stresses in the Dolaei tunnel area (Nabavi 2006).

Table 4 Mechanical parameters of the rock mass and fault zones

Parameter	GSI	m_i	E_m (GPa)	v_m	G_m (MPa)	σ_{cm} (MPa)	σ_{tm} (kPa)	c_m (kPa)	φ_m (°)	ψ_m (°)
Surrounding rock	35	10	2.1	0.36	770	2.8	280	940	28	3.5
Fault zones	15	6	0.5	0.41	177	0.25	25	100	15	0

It is also notable that based on the theories of rock deformation, soft rocks are not capable of sustaining high differential stresses. Differential stresses cause time-dependent deformation and flowing over periods of geological times. These gradual deformations prevent the accumulation of differential stresses. Hence, in deformable rocks the stress state tends to be isotropic and almost equal in all three directions which is known as the lithostatic stress state (Zang and Stephansson 2009).

Equation 5 is adopted in this study to compute the in situ stress state in the rock masses around the Dolaei tunnel. However, the action of faults, the extensive weathering and deterioration due to water effects make the gauge and broken materials very deformable. Therefore, such rock materials easily flow and cannot sustain high differential stresses. This shows that a k_0 value of 1.0 may well represent the status of in situ stresses within the broken rocks in the old fault zones which have lost their activities during the time (Nazeri et al. 2004).

6 Proposing an Appropriate Excavation Methodology and the Support Measures to Prevent Sinkholes

After identifying the most dominant geological and geomechanical characteristics of the rock mass surrounding the Dolaei tunnel, the research intends to investigate a tunnel design and construction methodology in order to avoid a sinkhole formation and its consequences.

One of the fundamental steps in the design of the support measures is to estimate the rock loads as accurately as possible. Methods based on rock mass classifications and the convergence–confinement approach are widely used in practice (Hoek et al. 1995; Vlachopoulos and Diederichs 2014). However, the convergence–confinement approach is quite cumbersome for excavations with complex geometries or for excavations performed in different stages. This method is also not used for support system designs in shallow tunnels in weak rocks, where roof caving occurs (Hoek 2004). Therefore, employing rock mass classification methods is a useful alternative for the primary design of shallow excavations in weak rocks, such as the Dolaei tunnel.

In this paper, Perri's (2007) underground excavations classifications together with the design guidelines from the scientific literature (Barton et al. 1974; Bieniawski 1989), will be used to select the proper preliminary supports for the Dolaei tunnel. Empirical relations based on the rock mass classification methods are also used to estimate the support loads. Then in Sect. 7, numerical simulations will be used to optimise the support characteristics.

Taking into account the rock mass characteristics presented in the previous sections and Bieniawski's guidelines (1989), the estimation of the stand-up time of the Dolaei tunnel shows that the tunnel excavation led to an immediate collapse. Therefore, performing staged excavations and using support measures, particularly rock bolts and shotcrete, are required to ensure the tunnel's stability and to prevent a progressive tunnel collapse (Sharifzadeh et al. 2012, 2013). In particular, considering the width of the tunnel, *B = 10* m, and an average value for the RMR equal to 35, it is suggested to divide the tunnel face into two sections, top and bench. Systematic bolting, with a length of 4–5 m and spacing of 1–1.5 m, and a layer of reinforced shotcrete with a thickness of 10–15 cm are also suggested to provide the tunnel with stability. Finally, utilising light to medium sized steel ribs is also suggested for stabilising the critical locations. However, these support measures are not adequate for excavation in fault zones. Based on Bieniawski's guidelines, employing medium to heavy ribs, spaced at 0.75 m, with forepoling is suggested for such locations. Furthermore, longer rock bolts of 5–6 m spaced at 1–1.5 m, and 15–20 cm shotcrete are proposed to provide the stability of the tunnel in weak fault zones.

The support measure chart based on the Q-value is also widely used in tunnel engineering to estimate the required support systems for underground excavations. The recommendations given in the chart are related to the permanent support system. It is also notable that these comments are general in nature. This means that engineering judgement must be applied to modify the recommendations, based on the special conditions of each individual case (Barton 2002; Barton et al. 1974; NGI 2013).

Based on the Q-support guidelines for rock masses with intermediate to good quality (Q > 0.1), the level of support required for walls is usually less than what is used for crowns. The normal support diagram is

valid for the crown and springlines of excavations. Therefore, to estimate the proper support for the walls the Q-value is adjusted. For this specific case ($Q_w = 2.5Q$), if the rock mass is weak and the level of stresses are high, no adjustment is considered in the Q-value to design the supports for the tunnel walls ($Q_w = Q$). Bearing in mind the width of the Dolaei tunnel and Q = 0.37, employing 13 cm of reinforced shotcrete with 3.0–3.5 m long rock bolts are suggested to support the tunnel crown. The lateral spacing of the rock bolts should be around 1.5 m. This support pattern is adjusted for the tunnel's walls and 10 cm shotcrete with 3 m rock bolt with 1.7 m lateral spacing are suggested as the proper support systems for these parts. Moreover, employing a thicker shotcrete layer of 15–20 cm is suggested for tunnelling in the fault zones (see Fig. 10).

The presented primary investigations (see Sect. 4) indicated that the rock mass surrounding the Dolaei tunnel is poor. This shows that the New Austrian Tunneling Method (NATM) could have been a good alternative for the construction of the tunnel. NATM is usually used for constructing tunnels in weak ground. One of the fundamental concepts of the method is to reinforce the rock mass in order to use its inherent strength for support (Karakus and Fowell 2004). This method has numerous benefits for tunnel engineers, including the flexibility to adopt different excavation geometries, different sizes and shapes; the compatibility to accept different support measures in order to control ground movements; the flexibility for installing a waterproof membrane to reduce and control the inflow of groundwater; and the convenience for monitoring the deformation and stress redistributions (Karakus and Fowell 2004). In the following subsections the steps followed for the identification and design of these measures are described.

Perri's guidelines consider the responses of excavations as controlled not only by the rock mass's mechanical characteristics, but also by the magnitudes and directions of the in situ stresses, the geometry of the excavation and the surface topography (Perri 2007). Perri classified underground excavations into three groups: shallow cover, intermediate, and deep tunnels (Perri 2007). For tunnels with an intermediate overburden thickness, the geotechnical characteristics of the rock masses are the most important factors controlling the behaviours of excavations and the support loads (Perri 2007). However, for tunnels outside an intermediate depth, additional effective factors impacting on the magnitude of the support pressure, such as the magnitude of the in situ stresses for deep tunnels or the rigid kinematics equilibrium related to the proximity of the excavation to the surface for shallow tunnels, need to be taken into account (Perri 2007).

Fig. 10 Q-support chart for selecting support measures (NGI 2013). *Dotted blue* and *red lines* show the required support measures for tunnelling in fault zones and rock mass. The *dotted black line* also indicate the proper support for tunnel walls in rock mass
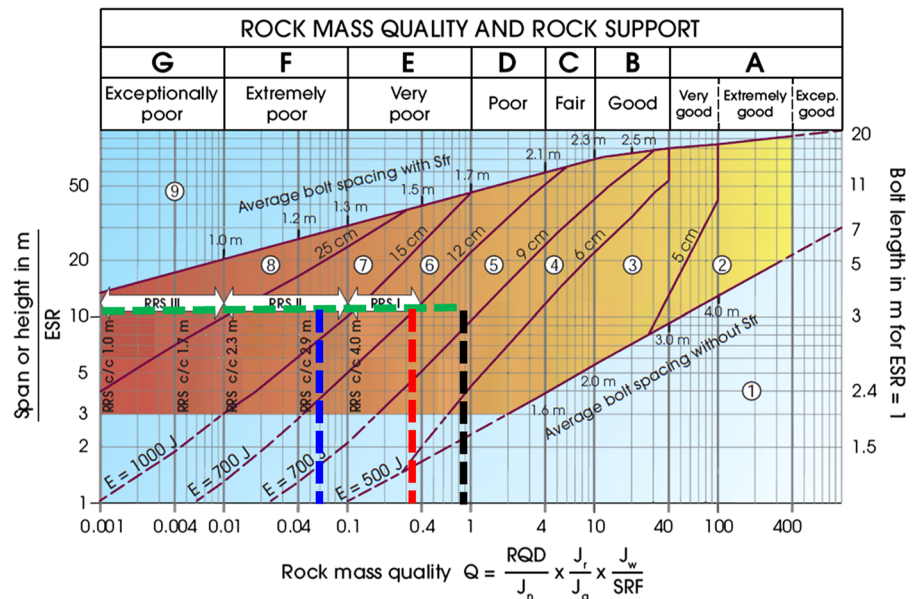
Perri ([2007]) proposed a methodology to assess the support systems of tunnels with a 10 m span, considering the limiting depths, H_i and H_s, between the three classes, the Index of Competency (*IC*) and the GSI of the rock mass. The two limiting depths, H_i and H_s, are used to classify the underground excavations as shallow, intermediate or deep excavations. They are calculated using the excavation span (*B*) and the rock mass GSI value (Perri [2007]):

$$H_i = B(50/GSI) \quad H_s = B(GSI/5) \tag{6}$$

The rock mass Index of Competency (*IC*) is obtained by (Perri [2007]):

$$IC = \sigma_{cm}/\gamma h \tag{7}$$

where h is the overburden height and σ_{cm} is the rock mass strength, which is defined by (Marinos and Hoek [2000]):

$$\sigma_{cm} = \sigma_{ci}\left(0.0034 m_i^{0.8}\right)\left(1.029 + 0.025 \exp(-0.1 m_i)\right)^{GSI} \tag{8}$$

where m_i is one of the Hoek–Brown parameters for intact rocks and σ_{ci} is the uniaxial compressive strength of intact rocks (Marinos and Hoek [2000]).

For relatively large values of *IC,* which is normally the case under moderate cover depths where the in situ stresses are usually low, the rock mass quality is the key factor controlling the behaviours of the excavation (Perri [2007]).

Considering Perri's guidelines, and recalling the geometry of the Dolaei tunnel (*B* = 10 m) and the rock mass characteristics (GSI = 35), the limiting depths of H_i and H_s are calculated to be 14.3 and 70 m, respectively. Therefore, 16 cm shotcrete and 11 rock bolts (with 6 m length and 1.25 m lateral spacing) are proposed to provide the tunnel with stability. According to Perri's guidelines, the bearing capacity of the proposed support is approximately 0.35 MPa.

Generally speaking, the support pressure of an arch shaped tunnel, which is excavated in hard rocks, partially depend on the size of the excavation, while the support pressure of an excavation in a poor rock mass highly depends on the excavation size (Bhasin and Grimstad [1996]; Bhasin et al. [2006]). Since the rock masses around the Dolaei tunnel are weak, the magnitude of the support pressure depends on the tunnel size, which can be estimated by Eqs. ([9]) or ([10])

proposed by Bhasin and Grimstad ([1996]) and Perri ([2007]), respectively:

$$P_r(MPa) = 0.04 \times B/J_r \times Q^{-\frac{1}{3}} \tag{9}$$

or

$$P_r(MPa) = \alpha(B+H)\gamma$$
$$\alpha = 1244 m_i^{-1.433} GSI^\delta \delta = 0.0004 m_i^2 - 0.0046 m_i - 1.2344 \tag{10}$$

where J_r is the joint roughness number in the Q-system and its value for the rock mass around the Dolaei tunnel is approximately 2.0. H is the height of the excavation (m), and is 8 m. Equations ([9]) and ([10]) lead to the same support pressure of 0.26–0.28 MPa in the Dolaei tunnel, viz., the proposed support is adequate for the Dolaei tunnel case.

7 Evaluation and Modification of the Primary Design Based on Numerical Simulation

Predicting the responses of excavations in a jointed and weak rock mass might be a complicated task (Hoek [2001], [2007]). As shown in the previous section, empirical approaches such as the rock mass classification methods are commonly employed to design the primary supports, such as rock bolts and shotcrete. However, these methods suffer from some limitations related to the difficulties to capture the realistic behaviours of the excavation and the supports, and therefore, they may lead to unsafe or too conservative designs (Banton et al. [2004]; Hoek [2007]; Potvin et al. [2012]). For this reason, numerical simulations are employed to check and ultimately modify the primary design. Applying numerical simulations in the design of underground constructions has numerous benefits which could considerably increase the accuracy of the analysis. For example, numerical methods are capable of considering the nonlinear behaviours of the rock mass and the support systems, the irregular geometry of excavations, and the complex initial and boundary conditions.

7.1 Selecting a Proper Numerical Method

The methods for the numerical simulation of rock masses are generally classified into continuous and

discontinuous. Stille and Palmstrom (2008) proposed a simple strategy to select a method suitable for the numerical analysis of underground structures, based on the geological characteristics of the rocks and the geometry of the excavation. According to this method, if the Continuity Factor of the ground, CF, is greater than 30, the rock mass will be classified as a continuous medium, which is more convenient to be analysed by continuous methods (see Table 5). The CF is defined as the ratio between the excavation width, B (m), and the block size, W_b (m) (Stille and Palmström 2008).

Considering the three joint sets with relatively low spacing of 0.2 m (see Table 2) and the schistosity, the rock mass surrounding the Dolaei tunnel is classified as heavily jointed. In cases where three joint sets occur, the block volume can be estimated by (Palmström 1995):

$$V_b = S_1 \times S_2 \times S_3 \times (Sin\delta_1 \times Sin\delta_2 \times Sin\delta_3) \quad (11)$$

where S_1, S_2 and S_3 are the spacing between the joint sets and δ_1, δ_2 and δ_3 are the angles between the joint sets (Palmström 1995).

Equation 11 shows that the size of the rock blocks is very small in comparison to the tunnel span. Therefore, considering Table 5, a continuous method (e.g., Finite Difference (FD) or Finite Element (FE)), could be a suitable approach for analysing the tunnel's stability. Phase2 and FLAC^{3D} are used in this section to perform the two and three-dimensional numerical modellings. The Mohr–Coulomb constitutive model is used to replicate the mechanical behaviour of both rock mass and fault shear zones. The required input parameters were previously derived and explained in Sect. 4.4 (see Table 4).

Numerical simulations are used to investigate the effects of the excavation method and the support on the stability of the Dolaei tunnel in order to prevent a progressive collapse and sinkhole formation. For this

purpose, two different strategies are employed. Firstly, the overall tunnel section is excavated in one stage. The deformations induced by the excavation will be checked based on Sakurai's failure criterion, to assess the tunnel's stability. In the second strategy, the tunnel section is divided into two sections, consisting of the top and bench, to control the tunnelling induced deformations and to reduce the sudden load transfers due to the excavation (Sharifzadeh et al. 2012, 2013; Song et al. 2013). The supports are installed after excavating each sub-section. The effects of performing staged excavation on stabilising the tunnel is analysed through the numerical modelling (see Sect. 7.4). Finally, a numerical simulation is performed to investigate the effects of tunnelling in weak fault zones. The details of the numerical simulations are described in this section.

Since the tunnel is excavated in weak rocks, it is expected to disturb a large area surrounding the tunnel. Therefore, a uniform mesh consisting of 3-node triangle elements has been selected for analysis. The overall geometry of the model and the boundary conditions are schematically shown in Fig. 11.

7.2 Mechanical Parameters of the Rock Bolts and Shotcrete

The mechanical parameters of the support measures are important items and are highly necessary for conducting a detailed numerical stability analysis. The main goal of using rock bolts is to help the surrounding rock mass to support itself (Lang and Bischoff 1982; Osgoui and Ünal 2009). The design of a rock bolt system includes selecting the type, geometry (length and diameter), capacity, and pattern (lateral spacing) of the bolts. The interactions among the bolts depends on their length and spacing, the tension, the installation time, the mechanical properties of the rock mass, the status of the in situ stresses, and the values of the

Table 5 Classifying grounds based on the CF (Stille and Palmström 2008)

Ground continuity factor	$CF = B/W_b$	Variation in block volume $D_t = 10$ m
Continuous (bulky)	>30	<0.04 m^3
Discontinuous–continuous (blocky–bulky)	15–30	0.04–0.3 m^3
Discontinuous (blocky)	6–15	0.3–5 m^3
Continuous–discontinuous (intact–blocky)	3–6	5–40 m^3
Continuous (intact)	<3	>40 m^3
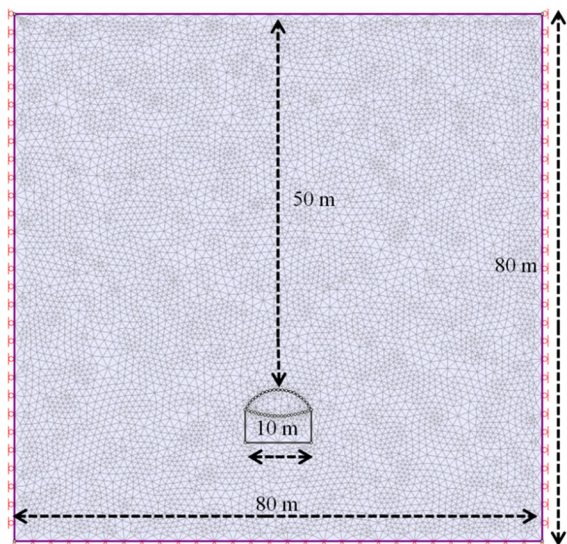
Fig. 11 A view of model geometry, boundary condition and uniform mesh

rock loads (Lang and Bischoff 1982; Osgoui and Ünal 2009). The mechanical characteristics of the proposed bolts for stabilising the Dolaei tunnel are summarised in Table 6 (Flåten 2015; Malmgren 2005; Marenče and Swoboda 1995).

Shotcrete is also an effective support, particularly for controlling excavation induced deformations in weak ground. It is widely used around the world to support both underground and surface excavations. Several factors impact on the quality of shotcrete, such as the ground conditions, the mix design (material components and properties), and the time of the installation (Hoek 2007; Malmgren 2005).

Understanding the interactions between the rock mass and the shotcrete is useful for the design of the shotcrete supports. This interaction varies, based on the loading environment and the values and directions of the applied loads (Banton et al. 2004; Stacey 2001). In the construction of underground structures in

jointed rocks, such as the Dolaei tunnel, shotcrete is used to stabilise the jointed rock mass and to prevent unravelling. Therefore, the adhesion and flexure of the shotcrete are critical parameters which must be considered in the design (Banton et al. 2004; Chryssanthakis et al. 1997; Stacey 2001). Additionally, the shotcrete can potentially infill discontinuities in the rock, with a consequent increase of the discontinuities' shear strengths and a decrease in the water flow (Chryssanthakis et al. 1997; Stacey 2001). Spraying a layer of shotcrete immediately after excavation can also provide a good barrier to protect the rock mass against weathering and deterioration (Stacey 2001).

Empirical charts, mainly based on the RMR and Q-system, are usually used to design a shotcrete support for an underground excavation (Barton and Grimstad 1995; Lowson and Bieniawski 2013). Generally speaking, the thickness of the sprayed shotcrete increases with decreasing the Q or RMR values and increases with the excavation span. Reinforcing the shotcrete with fibre or wire mesh is also recommended for cases where the levels of induced stresses are considerable or the magnitude of the deformation is significant (Barton and Grimstad 1995). The mechanical characteristics of the shotcrete considered as the most appropriate primary support for the Dolaei tunnel have been collected from the related literature (Chryssanthakis et al. 1997; Malmgren 2005; Perman et al. 2007; Pöttler 1990), and are summarised in Table 7.

7.3 Excavating the Tunnel Section in One Stage

Figure 12a, b show the contour plot of the displacements and the plastic zone around the unsupported tunnel in the case that the whole tunnel section is excavated in one stage. The results indicate that the overburden materials above the tunnel are significantly impacted by the excavation. In particular, the rock material disturbed by the excavation tends to cave

Table 6 Mechanical parameters of fully grouted rock bolts

Parameters	Value	Parameters	Value
Elastic modulus of bolts	210 GPa	Bolt yield stress	500 MPa
Cross section area of rock bolts	804 mm^2	Grout stiffness	8.0 GN/m
Peak bond strength	14 MPa	Grout exposed perimeter	141 mm
Residual bond strength	5 MPa	Density	7800 kg/m^3
Grout friction angle	50°	Bolt length	3.0–5.0 m
Residual tensile capacity	0.01 MN	Lateral spacing	1.5–2.0 m

Table 7 Mechanical parameters of shotcrete

Parameter	Value	Parameter	Value
Elastic modulus	7 GPa	Poisson's ratio	0.20
Compressive strength (28 days)	25 MPa	Shotcrete thickness	15 cm
Density	2400 kg/m^3	Material description	Isotropic

into the tunnel under its own weight, leading to a progressive upward collapse, which results in the formation of a sinkhole subsidence. It is, therefore, necessary to modify the method of excavation and to use the appropriate support systems to control the excavation induced deformations.

7.4 Multi-section Excavation

The numerical findings showed that excavating the whole tunnel section in one stage induces huge deformations and leads to tunnel instability (see Sect. 7.3). The empirical guidelines suggest that using a multi-section excavation technique with a short advance length could be a practical method to increase the tunnel's stability (Bieniawski 1989; Perri 2007; Sharifzadeh et al. 2013; Yu and Chern 2007); and

therefore, a staged excavation is suggested for the Dolaei tunnel.

The diagram proposed by Yu and Chern (2007) is used for selecting the optimum construction procedure, based on the tunnel's span and strength/stress ratio at the construction site. Based on this diagram, top heading and benching technique is suggested as the proper method for excavating the Dolaei tunnel. Therefore, the effects of performing a staged excavation is analysed through the numerical simulations in order to understand the real benefits of this technique.

The tunnel section is divided into two parts, top and bench, as shown in Fig. 13. In the first stage the top section is excavated. Then the top support systems, which consist of bolts and shotcrete, are installed. The next stage includes excavating the bench. Finally, the support systems for the walls and floor are installed. It
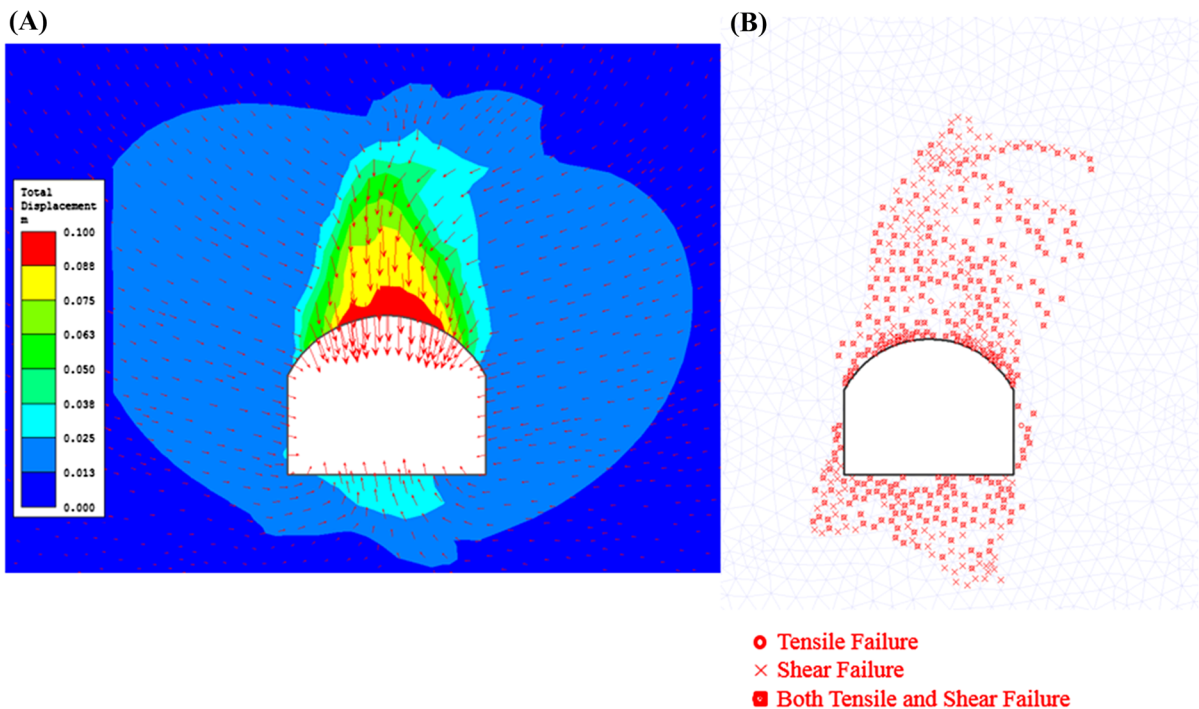
Fig. 12 a Contours and vectors of displacement around the unsupported tunnel, and **b** plastic zone (shear and tensile failure) around the unsupported tunnel, showing the area of caving

should be noted that a portion of the deformations take place during the excavation of each part, and before installing the supports. This partial deformation significantly affects the pressure applied on the support systems and plays an important role in designing the optimum supports to stabilise the excavations (Hoek 2007; Pöttler 1990). One of the benefits of the numerical simulations is in considering the effects of this partial deformation in the support design.

When a two-dimensional numerical model is employed for the design of support measures in underground excavations the effects of the three-dimensional nature of the excavation should be involved in the analysis. Hoek (2007) stated that displacement in the rock mass begins at a distance of about one half a tunnel diameter ahead of the face. These displacements gradually increase and almost reach one-third of the maximum value in the tunnel face. The displacement reaches a maximum value when the face has progressed about one and a half of tunnel diameter and the support provided by the rock

Fig. 13 Stages of excavation and supporting. **a** Before excavation, **b** excavating the top, **c** installing bolts and shotcrete in the top, **d** excavating the bench, **e** installing shotcrete in the walls and bolts in the bench and floor

core in the face is no longer effective (Hoek 2007). This rule of thumb can be used to estimate the appropriate relaxation before installing the tunnel supports. Vlachopoulos and Diederichs (2014) also investigated the applicability of the two-dimensional analysis for the design of support measures in complex conditions such as anisotropic stress field, multi-staged excavations and non-circular shaped tunnels. Outcomes of their investigations resulted in the following Sigmoid relation for approximating the Longitudinal Displacement Profile (LDP) (Vlachopoulos and Diederichs 2014):

$$u/u_{max} = 1/[1 + \exp(0.6(1 - 0.1S/R)(S/R - 5x/R - 1)] \quad (12)$$

where x is the distance from the face, R is the radius of the tunnel, and S is the distance between the face and the support.

Equations 12 may be used to estimate the amount of ground movements which occur before installing the tunnel supports. To simulate the staged excavation, the load split option available in Phase2 is used in this study. This option allows the user to split the field stress induced loads among different stages of the model. Load split can be used to gradually apply the field stress load as excavation progresses. This technique can be employed to simulate the three-dimensional effects of an advancing tunnel face, using a two-dimensional model. In other words, load split option allow to release a proportion of the stresses after excavating each part of the tunnel to take into account the effects if the 3D nature of the excavation.

A numerical simulation was performed for the case of a staged excavation based on Fig. 13. The tunnel is supported after excavating each section. Figure 14 shows the contour plots of the displacements and the extension of the plastic zone around the tunnel, respectively. This figure shows that employing a staged excavation technique and installing proper supports could be a practical method for controlling excavation induced deformations. In particular, Fig. 14b depicts the effects of conducting a staged excavation and supports on the yield zone around the tunnel. A comparison between Figs. 14b and 12 shows that applying a staged excavation and the composite supports, consisting of shotcrete and rock bolts, significantly reduces the size of the disturbed zone around the excavation. In other words, a staged
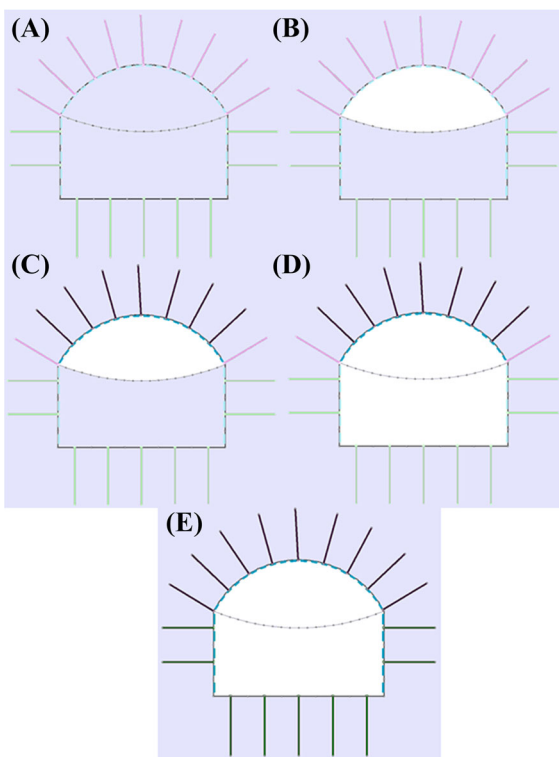
(A)   **(B)**
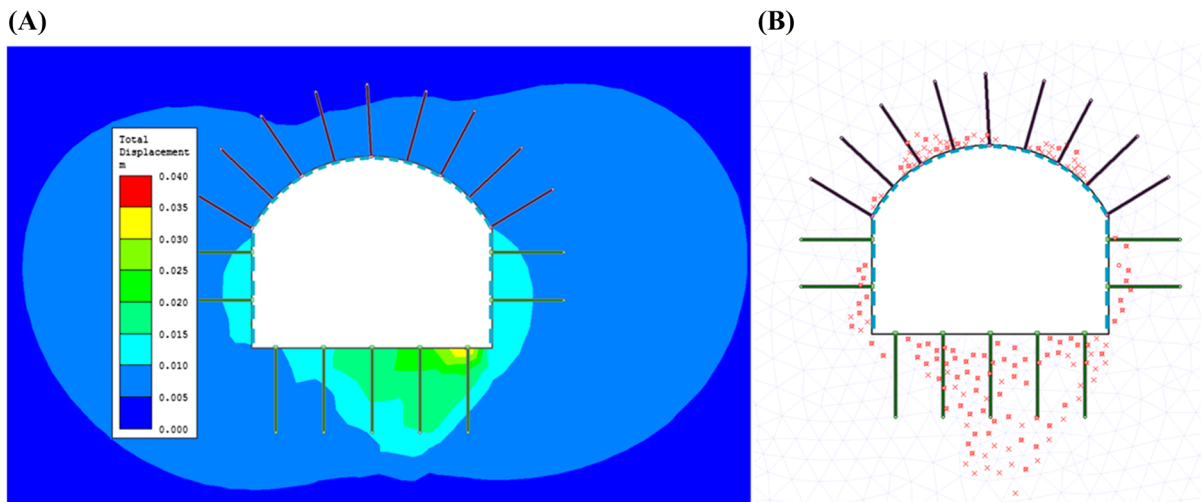
Fig. 14 **a** Contours and vectors of displacements and **b** extension of the plastic zone around the tunnel in a staged excavation

excavation technique is an effective method for stabilising excavations in relatively weak rock masses such as those surrounding the Dolaei tunnel.

The stability of the support systems is another important issue in the design of underground excavations which must be accurately assessed. Deriving explicit analytical or empirical relationships to take into account the real interactions between rock bolts, shotcrete and the rock mass is very complex (Lang and Bischoff 1982; Malmgren 2005). Therefore, a numerical simulation, which is capable of considering the nonlinear behaviours of composite supports, was utilised for analysing the reactions of the support elements and the interactions between them and the surrounding rock mass. Figure 15 shows the induced axial forces, shear forces, and the bending moments computed in the reinforced shotcrete support in the Dolaei tunnel.

The numerical results show that utilising a shotcrete layer, with a thickness of 15 cm, reinforced by 2 layers of wire mesh, with a diameter of 5 mm and a spacing of 10 × 10 cm (Φ5, @10 × 10), and 9 rock bolts (Φ32) with 3 m length and 1.5 m lateral spacing, results in a proper support system for the tunnel crown. Moreover, 10 cm shotcrete, reinforced with one layer of wire mesh are proposed for supporting the walls of the tunnel. The numerical findings also show that none of the elements of the shotcrete fail due to induced stresses and deformations. Additionally, the axial and shear forces in the rock bolts are computed and depicted in Fig. 16. This figure indicates that fully

grouted rock bolts, with a length of 3 m, a diameter of 32 mm, and a lateral spacing of 1.5 m, are appropriate for supporting the tunnel's crown and floor. Furthermore, 3 m rock bolts, Φ25, with 2 m lateral spacing are suggested for supporting the walls of the tunnel.

7.5 Excavations in Fault Zones

The Dolaei tunnel intersects with four fault zones (see Fig. 2). The weak and deformable nature of fault gouge materials and, in some cases, the presence of large volume of high pressure water trapped behind the impermeable fault materials cause several problems during tunnelling. To control and overcome such difficulties support measures must not only have sufficient capacity to sustain such huge loads, but they must be installed in appropriate sequences to prevent uncontrolled deformation of the surrounding rocks (Hoek 1999). Hence, it is necessary to analyse the capability of the proposed support system and the method of excavation in stabilising the excavation in these weak fault zones. The numerical simulations were, therefore, conducted using the mechanical parameters of the fault zones as in Table 4. The contours and vectors of the displacements around the tunnel are depicted in Fig. 17a. According to this analysis, the excavation induced displacements are significant, and a relatively large area around the tunnel is affected by the excavation in a fault zone. The numerical results indicate that the maximum surface subsidence is dramatically increased to 0.5–1.0 m.

Fig. 15 a Axial forces, **b** shear forces and **c** bending moments in reinforced shotcrete in a staged excavation
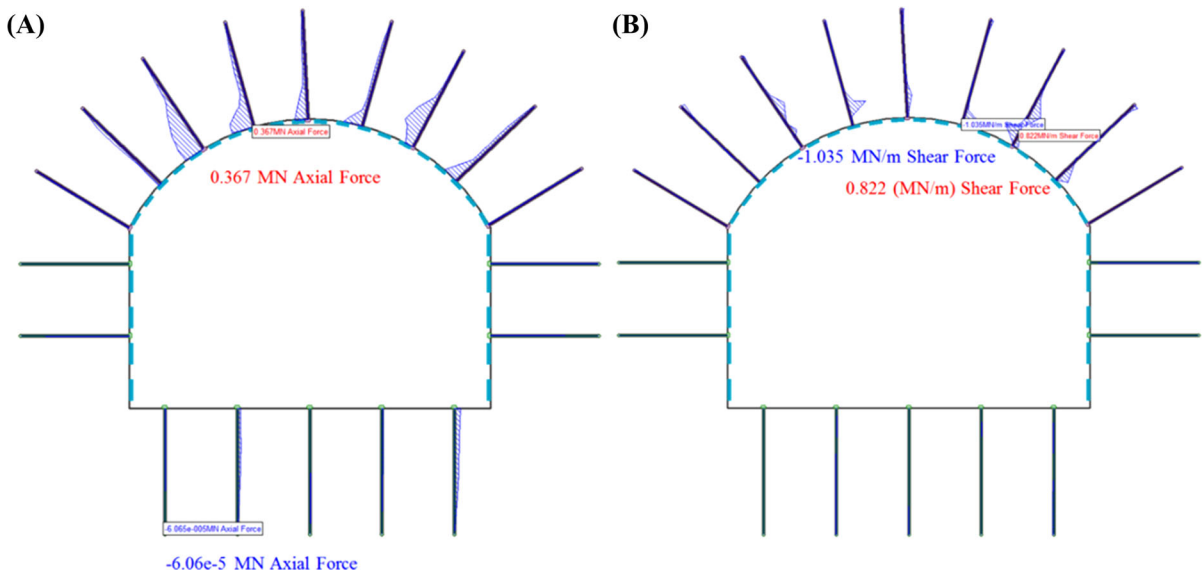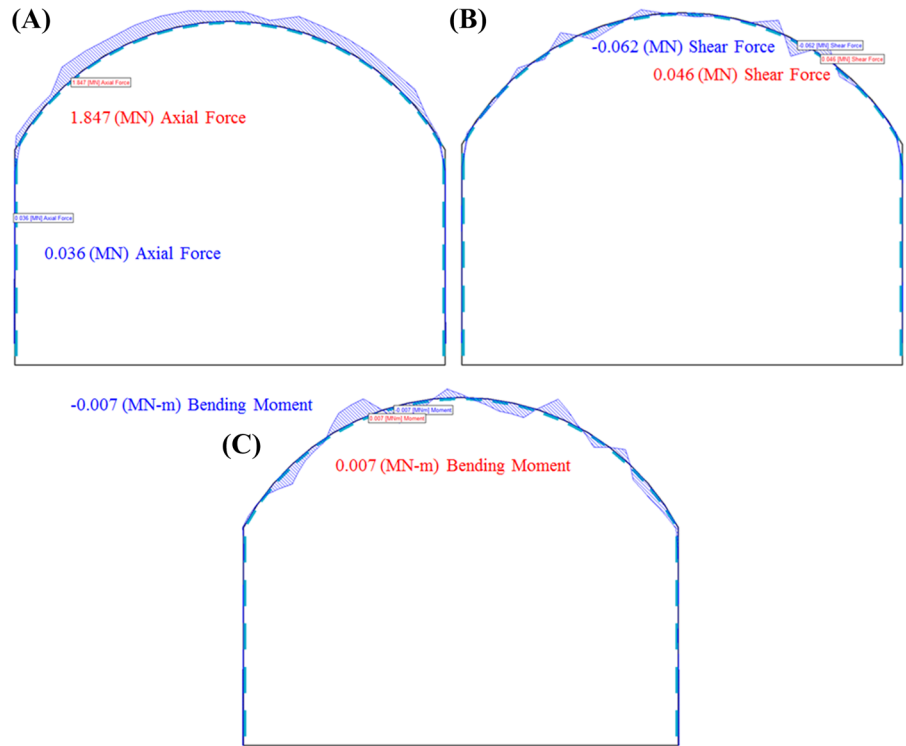
Fig. 16 a Axial and **b** shear forces in rock bolts

The extension of the plastic zone around the tunnel is also depicted in Fig. 17b. This analysis shows that the proposed support systems are not suitable for stabilising the tunnel in the fault zones. The surrounding rock masses in these regions are not strong enough to support their own weight and the loads due to the overburden materials. Therefore, the support system is extremely loaded, which eventually leads to its failure and the collapse of the overburden materials (see Fig. 17c). This also emphasises that a further analysis

(A)

(B)

(C)
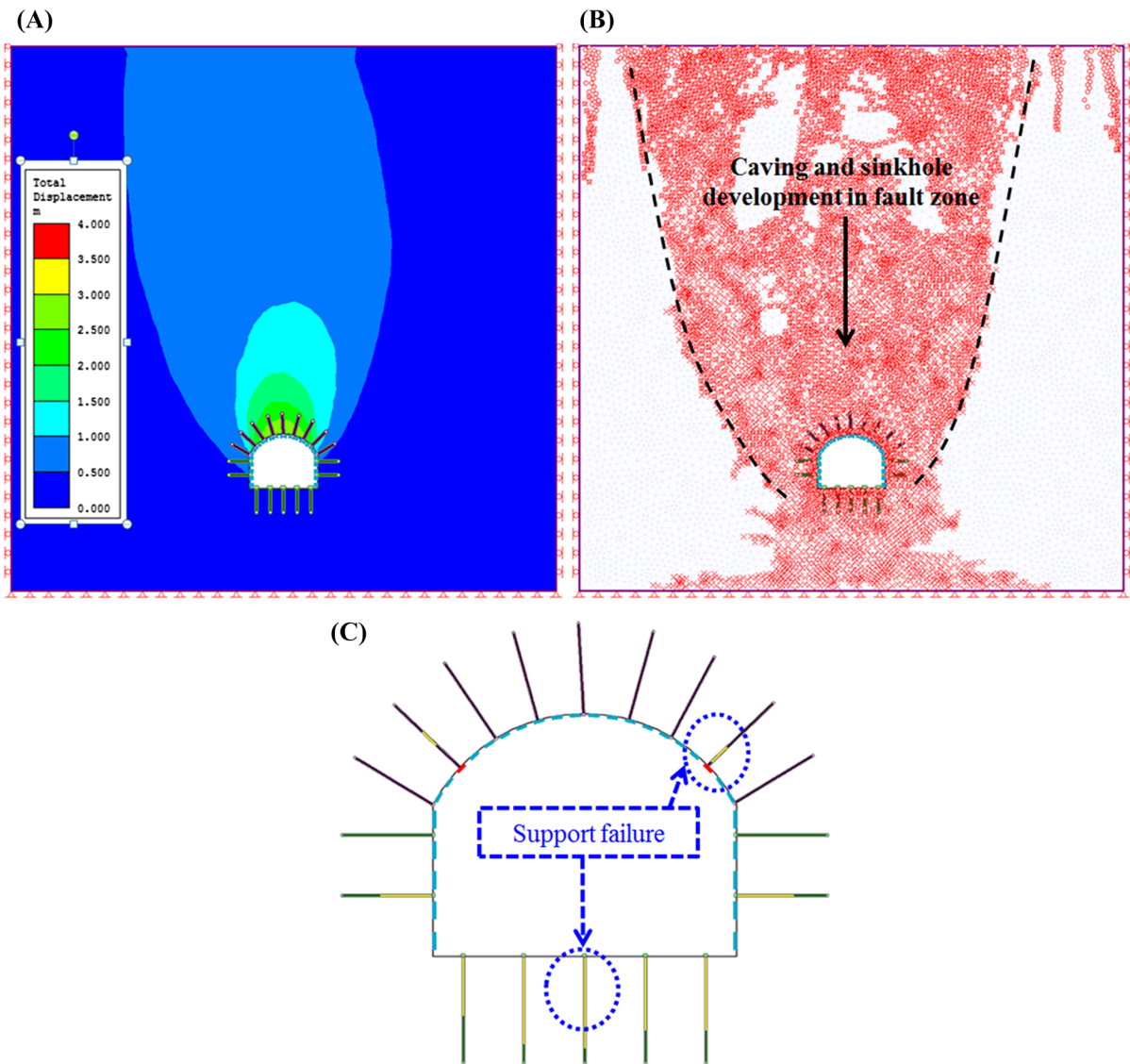
Fig. 17 *Contours* and *vectors* of displacement around the tunnel in the fault zones, **b** extension of plastic zone around the tunnel in the fault zones, **c** yield of rock bolts and reinforced shotcrete in the fault zones

is required to find an appropriate method to stabilise the excavations in the fault zones.

7.6 Pre-supporting Techniques

Finding an appropriate method to provide the global stability of the Dolaei tunnel, and to control the formation of sinkhole subsidences, is one of the main scopes of this study. In this section, numerical simulations are used to assess the effects of pre-supporting techniques for the stability analysis of the

fault zones surrounding the Dolaei tunnel. Pre-supporting methods are defined as a particular group of underground measures which are based on the systematic performing of forward treatments (e.g., forepoling, grouting, and nailing) on a tunnel face or the boundaries of excavations in successive steps. Within them, forepoling seems to be a good candidate for preventing a progressive collapse and sinkhole subsidence in the fault zones of the Dolaei tunnel (Hoek 1999, 2001; Pizzarotti 2001). Forepoling increases the rock mass's strength, decreases the ground

deformations, reduces the loads applied on the support systems, and waterproofs the rock mass (Oke et al. 2014; Pizzarotti 2001; Song et al. 2013).

Capturing the real interactions between the forepoling pipes and the surrounding rocks is very difficult. A numerical simulation might be a proper approach to investigate the role of forepoling on stabilising a tunnel in weak ground. However, performing such a sophisticated numerical analyse is very cumbersome and challenging. In this study, therefore, a simplified equivalent technique, which was proposed by Hoek (2004), is employed to reduce the complexities of the problem and to quantitatively investigate the role of forepoling in preventing the formation of sinkhole subsidence in the Dolaei tunnel (Hefny et al. 2004; Hoek 2001, 2004). Based on this technique, it is assumed that forepoling and the injection of grout into the rock mass around the umbrella pipes increases the mechanical properties of the rock mass. Therefore, to simulate the forepoling, the mechanical parameters of the area around the tunnel are increased. The area of the rock mass affected by the forepoling and grouting depends on the type of grout, the grouting pressure, the size and dimensions of the pipes, the geometry and pattern of the forepoling, and the permeability of the rock mass, which is highly controlled by the spatial geometry and the characteristics of the discontinuities. The length of the forepoling pipes is also controlled by the limitations of the drilling equipment. A length of 10–12 m is usually used in practice. The equivalent properties of the area can be estimated based on the GSI. In this study, the equivalent properties of the forepoling zone are extracted from the relevant literature (Habibi 2009; Hefny et al. 2004; Hoek 1999, 2004, 2007; Oke et al. 2014) and are presented in Table 8.

Where, E_{fr} (GPa), c_{fr} (MPa), σ_{cfr} (MPa), φ_{fr} (°) and v_{fr} are the deformation modulus, the cohesion, the compressive strength, the internal friction angle, and the Poisson's ratio of the rock mass reinforced by forepoling, respectively.

The numerical results show that a combination of a staged excavation and performing forepoling can control the ground deformations in the fault zone. The contours and vectors of the excavation induced displacements are shown in Fig. 18. By comparing Figs. 17 and 18, it is concluded that forepoling decreases the maximum ground displacements around the tunnel. The disturbed zone around the tunnel

Table 8 Equivalent mechanical properties of the zone affected by forepoling

Parameter	E_{fr} (GPa)	v_{fr}	C_{fr} (MPa)	σ_{cfr} (MPa)	φ_{fr} (°)
Value	2.5	0.25	1.0	3.0	45

decreases when forepoling is used. Since the extension of the plastic zone in the fault zone is significant (see Fig. 17), it is suggested to use longer and stronger rock bolts, i.e., 5–5.5 m in length and 32 mm in diameter.

Based on the available guidelines (Bieniawski 1989; Perri 2007) and the results of the numerical simulations, it is suggested using a composite support system consisting of 20 cm reinforced shotcrete (with 2 layers of wire mesh) and medium steel arches with 0.75–1.0 m spacing in order to retain the rock load in the fault zones. Heavy steel arches and in situ concrete were utilised in the Dolaei tunnel to ensure the stability of the tunnel (see Fig. 5). However, in the collapsed area, steel arches were heavily loaded and deformed (Ghasemi 2008).

Perhaps combining lattice girders and shotcrete could have been a solution to overcome such problems. According to the relevant references, the combination of shotcrete and lattice girders is more manageable than the combination of steel ribs and shotcrete (Baumann and Betzle 1984; Hoek et al. 2008). The former provides a very uniform and reliable support. Lattice girders are usually light, flexible and more cost effective than conventional steel arches. Additionally, the good bonding between the lattice girders and the shotcrete provides a very uniform support which can prevent the ingress of underground water (Baumann and Betzle 1984; Hoek et al. 2008). Therefore, using lattice girders, instead of steel ribs, is recommended in weak grounds. However, further analyses are required to check the performance of such a composite support, consisting of rock bolts, shotcrete and lattice girders, for preventing sinkhole formation in the Dolaei tunnel. The details of the mechanical properties and the responses of the composite support can be estimated based on the mechanical properties of the individual components and according to the methodology proposed by Hoek and co-workers (Hoek et al. 2008). It is also notable that reinforcing the floor material with rock bolts and concrete is highly suggested. This is to prevent floor

(A)   **(B)**
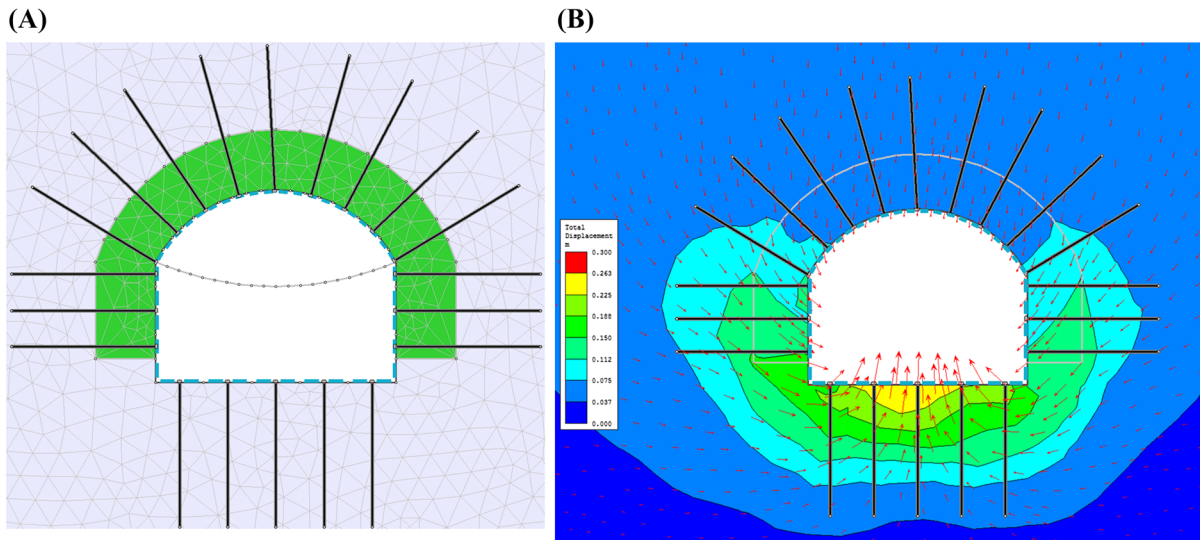
Fig. 18 **a** The geometry of the forepoling, and **b** *Contours* and *vectors* of the displacements around the tunnel in the fault zones after performing forepoling

failure and heave which significantly impact on the stability of the excavation.

7.7 The Effects of Discontinuities on the Tunnel Stability

A series of discontinuous numerical analyses were performed using the Universal Distinct Element Code (UDEC) to check the role of discontinuities in the initiation and propagation of the chimney collapses. The geometry of the numerical modelling is shown in Fig. 19. To simulate the jointed rock mass, the mechanical parameters of intact rocks and discontinues were utilised (see Tables 1, 2). However, to reduce the computational efforts, the outer domains of the numerical modelling were simulated as continuous rock masses. Therefore, the mechanical parameters of rock mass presented in Table 4 were used for this purpose. Outcomes of the numerical simulations show that due to the high level of horizontal stresses, the rock blocks are locked and the potential for block failure is not significant. It is also notable that systematic bolting and utilising reinforced shotcrete is a very efficient way to reduce the chance of block failure. Outcomes of numerical modelling also show that floor may undergo a shear failure. Therefore, employing rock bolts are suggested to stabilise the floor of the tunnel.

7.8 Three-Dimensional Numerical Simulation

A three-dimensional numerical simulation is a useful tool to assess the stability of shallow excavations (Goel et al. 2007; Hoek 2007). In this section a three-dimensional numerical simulation is illustrated to study the effects of the surface topography, the fault zone and the advancing tunnel length on the stability of the Dolaei tunnel. The Finite Difference code, FLAC^{3D}, is used for this purpose. The Mohr–Coulomb constitutive model is employed to replicate the mechanical behaviour of both the rock mass and the fault shear zone. The geometry of the model is shown in Fig. 20a.

As explained in Sect. 3.2, the intersection of the tunnel and fault F3 was one of the most critical locations. Therefore, only a part of the tunnel containing this fault shear zone is simulated. The location of the fault zone, which is almost perpendicular to the tunnel's axis, is shown in Fig. 20. Due to the symmetry of the problem, only one half of the tunnel is considered in the analysis. This is to reduce the runtime of the analysis. The advancing length in each stage of the excavation is 2 m. The supporting system consists of rock bolts, with a length of 3 m and a lateral spacing of 2 m, and reinforced shotcrete with a thickness of 0.15 m. Shell and cable structural elements are employed to numerically model the reinforced shotcrete and the rock bolts, respectively
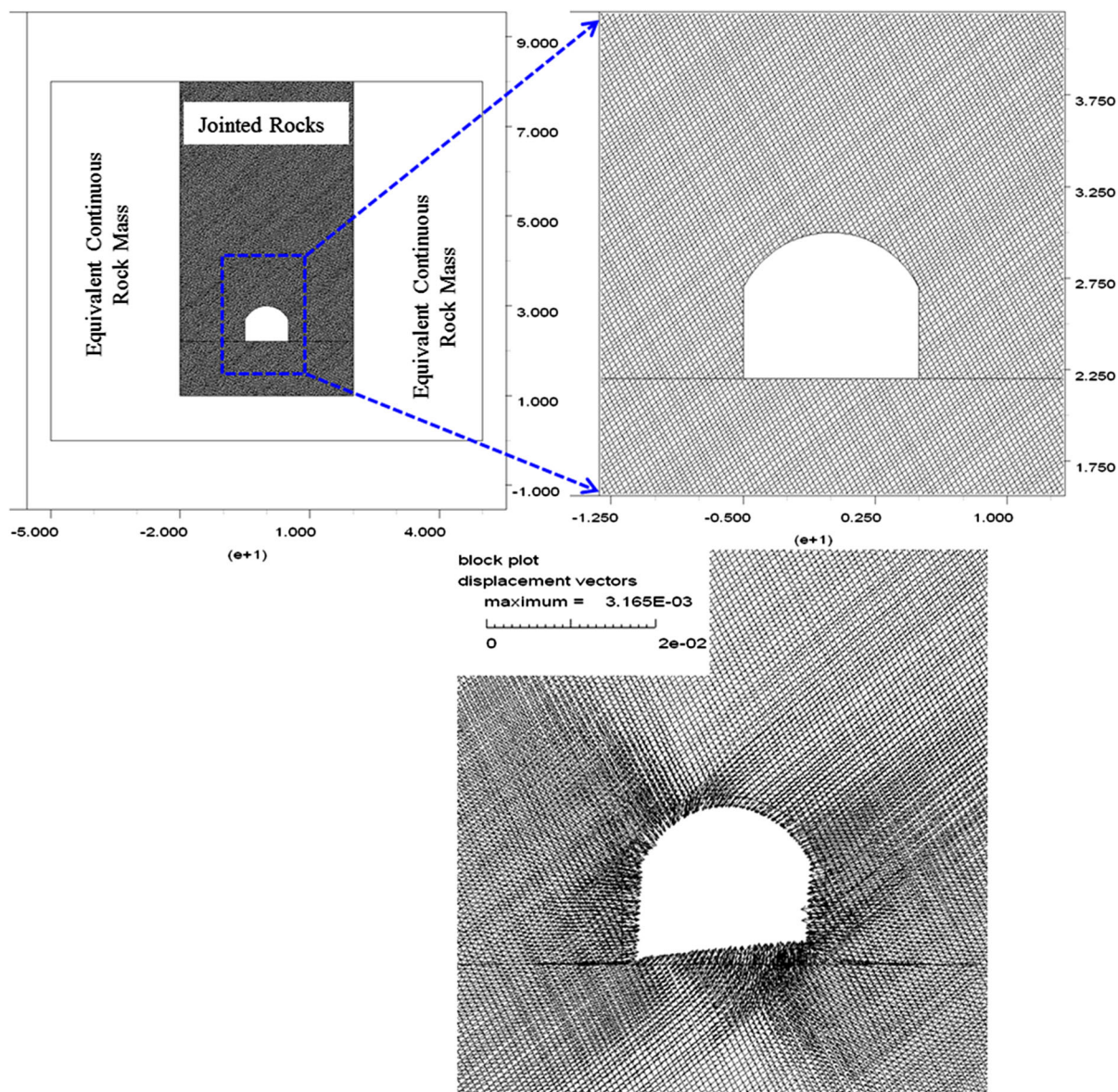
Fig. 19 The geometry of the DEM numerical modelling and the vectors of displacements around the tunnel

(see Fig. 20). Figure 20b shows the extension of the plastic zones around the tunnel as it is excavated in one stage. This figure also shows the formation of the upward progressive collapse above the tunnel. The tunnel section is then divided into top and bench sections to analyse the role of the multi-sections and the staged excavation.

At the beginning of the tunnelling only the top half is excavated. The tunnel gradually advances for ~20 m. The round of excavation is 2 m, and

after each round shotcrete and rock bolts are used to stabilise the excavation. When the top section advances for 20 m, the excavation of the bench also starts. After this stage both top and bench are excavated subsequently. This 20 m gap between the top and bench remains constant during the whole tunnelling. This is for practical applications, mainly to provide the drilling and hauling equipment with the required space to operate efficiently.
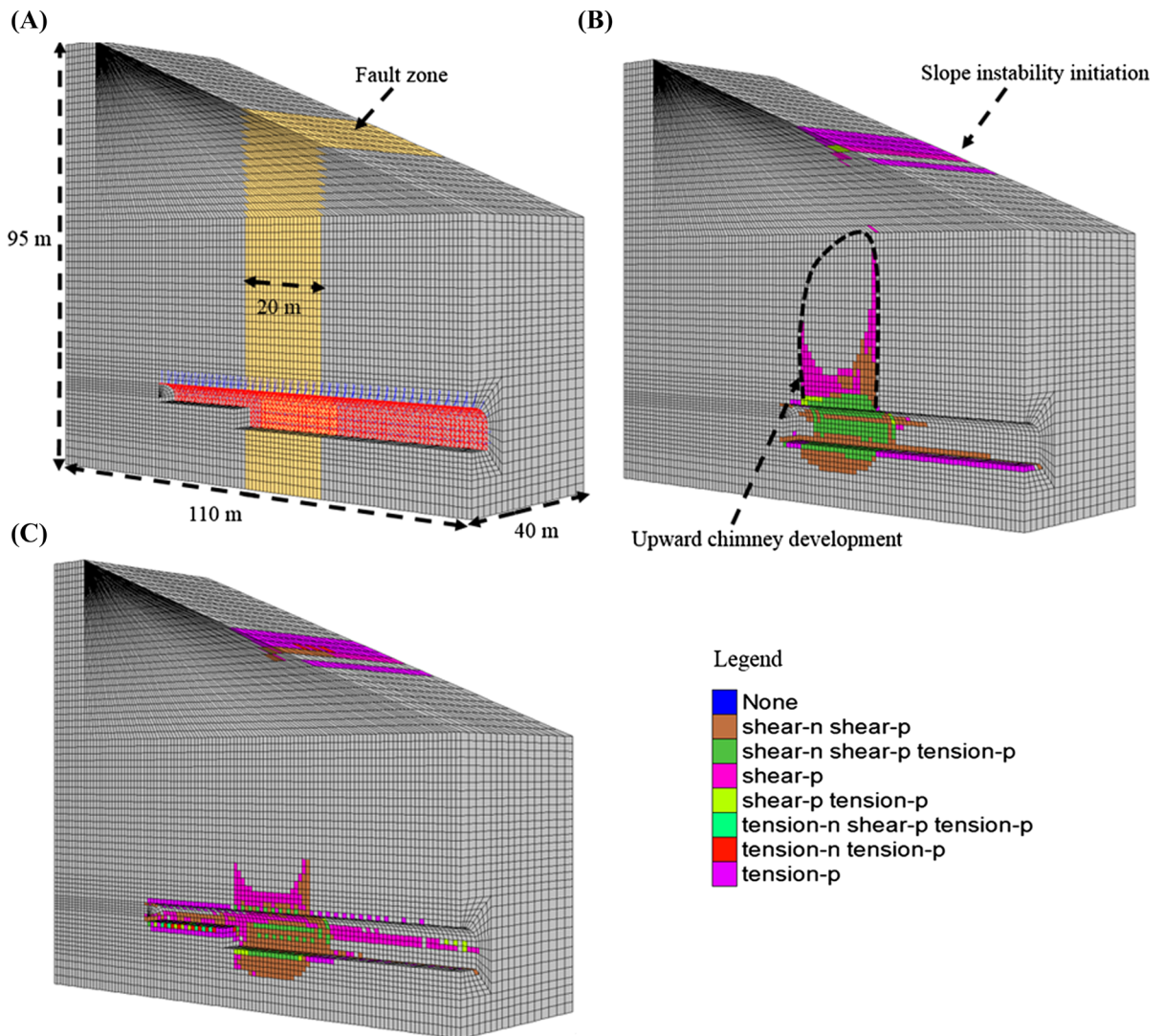
Fig. 20 Results of the 3D numerical simulation. **a** The geometry of the 3D model, **b** formation of upward sinkhole in the case that the whole tunnel section is excavated in one stage, **c** the effects of a staged excavation and supports on controlling the sinkhole formation. None: No-failure zone, Shear-n: The region failed under shear loading and the failure process is still in progress, Shear-p: The region failed under shear loading and the failure process is ceased due to the decreased amount of shear forces, Tension-n: The region failed under tensile loading and the failure process is still in progress, and Tension-p: The region failed under tensile loading and the failure process is ceased due to the decreased amount of tensile forces

The extension of the yielded elements around the tunnel is presented in Fig. 20c. According to this figure, dividing the tunnel section into two smaller parts to conduct a staged excavation and installing support systems have considerable effects on controlling the deformations of the fault zone. However, as explained in Sect. 7.6, it is common practice to consider a special design to ensure the stability of tunnels around fault zones. Pre-supporting methods,

particularly forepoling, are among the effective methods highly recommended for stabilising a tunnel's face and crown rocks in weak fault zones (Hoek 2004; Schubert 2008). Outcomes of the performed three-dimensional numerical modelling shows that employing 10–15 cm reinforced shotcrete and rock bolts with 3 m length, 25 mm diameter, and 2 m lateral spacing suffice for ensuring the stability of the whole tunnel except the fault zones. Employing longer

and stronger rock bolts, with 5 m length and 32 mm diameter, is suggested for such critical locations.

8 Discussions

Subsidence, particularly sinkhole subsidence, is an important problem, and may take place during the construction of underground excavations in weak rocks and in shallow depths. Sinkhole subsidence usually occurs due to the progressive collapse of the overburden materials. This problem does not only cause numerous difficulties for excavations, but may impact on the environment and damage the surface and sub-surface infrastructures in the proximity of the excavations.

Selecting an appropriate method of excavation and installing adequate support systems can effectively control the magnitude of the ground movements and prevent the formation of sinkholes. However, because of the complex behaviours of rock masses and the existence of several uncertainties, no method is capable of capturing the entire behaviours of the rock masses surrounding the excavations. The factors affecting the excavation's stability include the geological conditions, the mechanical behaviours of the rock mass, the in situ stress state, the method of excavation, the type of support systems and the time of their installation. Therefore, the design of underground excavations in problematic jointed rock masses is an extremely difficult task.

Rock mass classification methods are commonly used for deriving the mechanical characteristics of rock masses. Moreover, empirical methods based on the rock mass classifications are used for the primary design of the support systems. Although employing rock mass classifications methods and empirical relations have numerous benefits for the design of underground excavations and in selecting the proper supporting measures, utilising such approaches can present some limitations as the nonlinear behaviours of the rock mass and the supports is not considered in such empirical methods. The effects of the in situ stresses and the excavation method on the mechanisms of the ground movements are also overlooked. Moreover, the effects of the pre-supporting methods cannot be considered in empirical methods. Additionally, the interactions between the rocks and the support measures are not taken into account in empirical approaches.

Considering these limitations, it is suggested that empirical methods used for the preliminary studies of rock excavations and then verification of the primary design is based on numerical simulations. This strategy was used here to investigate the stability of the Dolaei tunnel and to assess the mechanisms of the sinkhole subsidence development in this tunnel. Therefore, this study proposes a combination of empirical approaches, based on the rock mass classification methods and numerical modelling, for the design and stability analysis of shallow excavations in weak rocks, such as in the Dolaei tunnel case. Since a rock mass's behaviours are extremely complex, none of the available methods is capable of capturing the overall responses of a rock mass. Therefore, comparative analyses based on the results of different approaches (e.g., empirical and numerical methods) could be a useful tool to improve designs and to minimise uncertainties.

9 Conclusions

In this study, the mechanisms of instability and sinkhole formation in the Dolaei road tunnel were investigated as a case study. The geological and geotechnical characteristics of the tunnel area were investigated in order to find the relations between the sinkhole formation and the characteristics of rock mass. The main factors triggering the collapse (e.g., foliation and schistosity, groundwater and weathering, and intersecting fault zones) were identified and briefly discussed. The mechanical characteristics of the rock mass were then extracted based on the available data in the literature. The geological studies revealed that contact metamorphic rocks (e.g., schists) are the dominant rocks of the region. This indicated that the rock masses in the Dolaei tunnel have not undergone a complete process of metamorphism in the tunnel area. Therefore, the rock mass's mechanical properties have not considerably improved.

Alternative empirical methods and guidelines, mainly based on the rock mass classification systems, were used to suggest an appropriate primary method for the excavation and for supporting the tunnel. This was to find a proper design and construction methodology in order to avoid the collapse and sinkhole formation in the Dolaei tunnel. To ensure that the proposed construction methodology and supports

could practically work, they were then examined through the numerical simulations. Both two-dimensional and three-dimensional numerical simulations were employed for investigating the stability of tunnel. The roles of the effective factors (e.g., performing a staged excavation, installing a composite support system and using forepoling) in controlling the progressive collapse and preventing the formation of sinkholes were examined through the numerical modelling.

The numerical findings revealed that excavating the entire tunnel section in one stage induces huge deformations, which lead to a progressive collapse and sinkhole formation. Therefore, a top heading and benching technique was suggested for excavating the tunnel. The outcomes of the numerical modelling also indicated that performing a staged excavation has direct effects on the support pressure. A temporary composite support, consisting of 11 rock bolts with 3 m length, 25 mm diameter and 2 m lateral spacing, and 10–15 cm shotcrete reinforced with 2 layers of wire meshes, was suggested for stabilising the tunnel. However, the numerical simulations indicated that this support strategy is not appropriate for stabilising the intersections of the tunnel and the fault zones. It was suggested using pre-reinforcing methods (e.g., forepoling) for strengthening the weak rocks ahead of the tunnel face in such critical zones. Utilising longer (5 m) and stronger (Φ32) rock bolts with 1.5 m lateral spacing is suggested to stabilise the tunnel roof, walls and floor in the fault zones. Moreover, a qualitative method was used in this study to numerically simulate the effects of forepoling. The results of the numerical study indicated that forepoling can be a very effective method for strengthening the rock mass in a tunnel crown and increases the arching effect. This phenomenon decreases the deformations due to an excavation.

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