Probabilistic and sensitivity analyses of effective geotechnical parameters on rock slope stability: a case study of an urban area in northeast Iran

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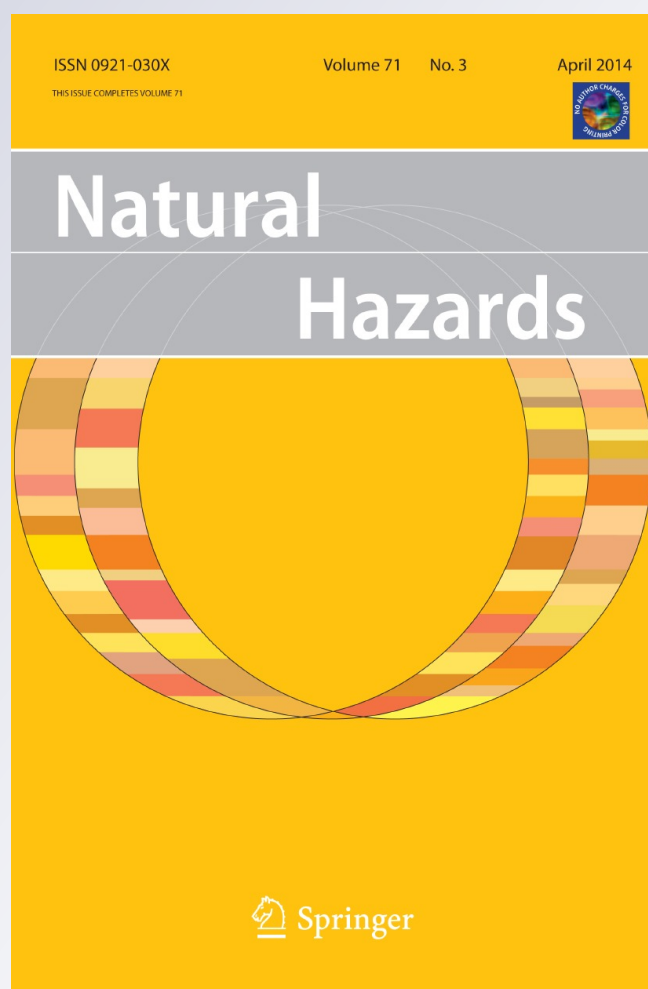
Nahid Vatanpour · Mohammad Ghafoori · Hossein Hedayati Talouki

Abstract Despite the development of cities, risk assessment of rock slope stability in urban areas seems not to be growing at the same time. Mashhad is a developed city in northeast of Iran with a population of over 2.4 million. Given the closeness of the southern part of Mashhad to the Binaloud mountain ridge, the stability of the residential complexes that are being constructed in this area is a critical issue. Based on the fundamental roles of discontinuity properties and geo-mechanical parameters of rock mass, in this study we evaluated the most influential parameters of the rock slope stability and the failure probability of the slope near the Negin residential complex built on this ridge. According to the deterministic and probabilistic analyses, the north trench that was excavated for this residential complex could potentially cause plane failure. Moreover, the relationship between effective parameters on instability and their impact on safety factors were determined by sensitivity analysis. Therefore, slope dip, pore water pressure, and joint set dip were highly influential on the safety factor. There was also a nonlinear relationship between different parameters and safety in the studied area. This study presents an approach for risk assessment of rock slope stability in urban areas.

Keywords Slide · Geo-mechanical properties · Discontinuities · Sensitivity analysis

1 Introduction

Slide is one of the most common types of natural hazards in rock slopes, which can occur in a wide range of geological conditions and slope geometry (Li et al. 2009; Singh et al. 2008). Even a minor instability might lead to slope failures and consequently

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economic loss and casualties. Movements usually happen as a result of changes in the stability factor and they are in turn resultant effects of stress distribution (Crozier 1986). Stability analysis of rock slopes is often performed either in order to design stable slopes in functional excavations (such as trenches of roads) or to evaluate the balance conditions of natural slopes. Nevertheless, it is known that slides in rock slopes should be considered as a multifactorial situation (Sharma et al. 2013). Primarily, rock slope stability was believed to be determined principally by structural discontinuities in the rock mass but not by the strength of the rock itself (Piteau 1972). Then, both slope geometry and failure direction were considered effective to generate a failure (Hudson and Harrison 1997; Wyllie and Mah 2004; Zangerl and Prager 2008). It seems that failures always occur on discontinuity directions (Hudson and Harrison 1997; Wyllie and Mah 2004). Therefore, the mechanical behavior and physical properties of the rock mass could be dominated by a number of intersecting discontinuity sets (ISRM 1978). In particular, tectonic structures, such as joints, play significant roles in both the strength properties of rock mass and stability issues (Pariseau et al. 2008; Donati and Turrini 2002; Brideau et al. 2009; Gattinoni 2009).

Geological structures and rock weathering have prominent roles in rock slope instability. The presence of tectonic structures might lead to the generation of fractures and finally failure in the rock mass (Donati and Turrini 2002; Regmi et al. 2013). Analysis of rock slopes that are susceptible to wedge failure by fracture system modeling has shown an association with equilibrium analysis (Grenon and Hadjigeorgiou 2008). The effects of tectonic forces on large slides were studied by Brideau et al. (2005, 2009). Moreover, Himalayan rock slides have been studied by probabilistic analysis (Pathak and Nilsen 2004). The effects of geomechanical properties of rock and discontinuities (Papini et al. 1998), hydrogeological conditions, and the presence of old faults on slope stability have been studied in Iran using the probabilistic expert semi-quantitative (PESQ) coding methodology (ZareNaghadehi et al. 2011).

Discontinuity properties are predominant contributing factors in rock slope stability. In addition, other effective parameters that play some roles in instability are represented by the type and the filling material properties of joints, the external forces conditions, and the weathering and groundwater level in tensional cracks (Hudson and Harrison 1997; Zhang et al. 2000; Sharma et al. 2013; Ping 1997; Regmi et al. 2013). Given that the effective parameters of the instability of a rock slope vary from one geometrical location to another (Hoek and Bray 1981; Park and West 2001; Gischig et al. 2011), the hazard assessment of rock slides on the overlapping areas of cities with mountains is an important issue for the sake of safety in building structures and infrastructures (Li et al. 2009). For instance, Alexander (1986), O'Hare and Rivas (2005) and Holub et al. (2011) studied, from statistical point of view, landslide damages to buildings and the the vulnerability of populations to slides. Indeed, differences in effective parameters of slide occurrence in diverse locations clearly demonstrate the essential role of case studies in the field of applied geoscience and might lead to better conclusions for mechanism definition and simplifying of building procedures.

In this study, given the residential extensions of the southern part of the city of Mashhad into the northern parts of the Binaloud mountainous region (Fig. 1), rock slide hazard assessment seems to be a critical issue for verifying the stability of these residential complexes. In order to reach this aim, we selected in this area one complex, the Negin residential complex, and evaluated the most influential negative/positive parameters of the slope stability of the trench of this residential complex (Fig. 2). This complex was the largest one in the study area. Moreover, we evaluated the effect of other parameters (e.g.,
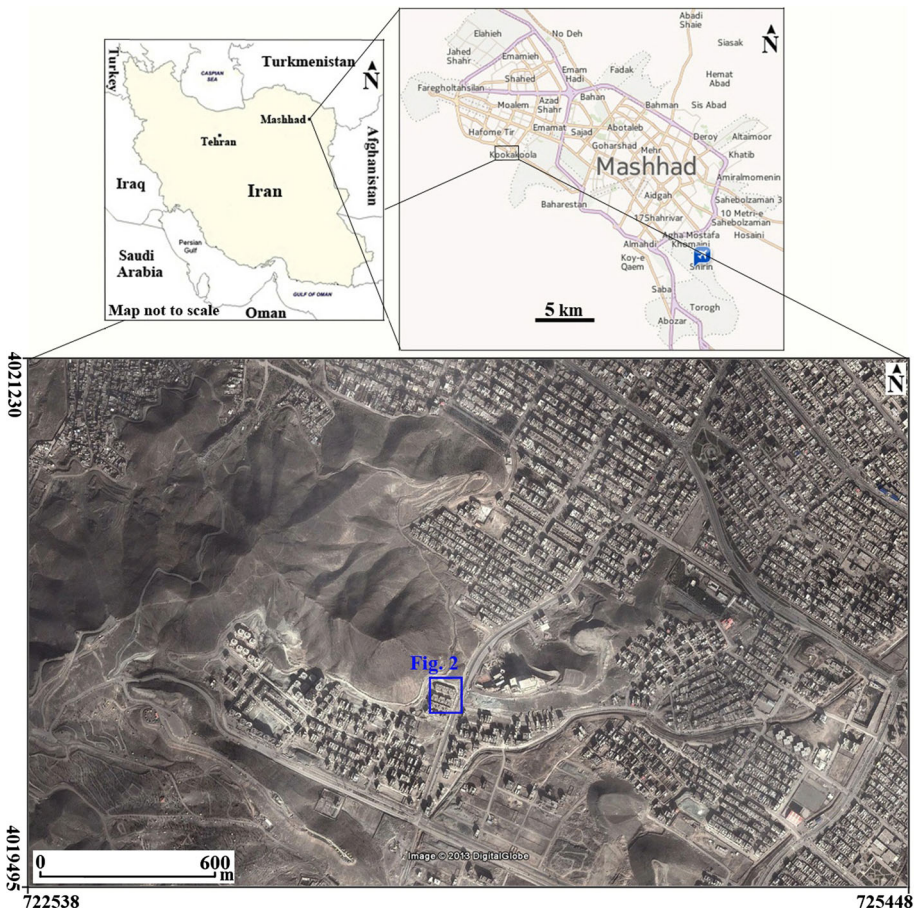
Fig. 1 The residential complex in southwest of Mashhad city in Iran
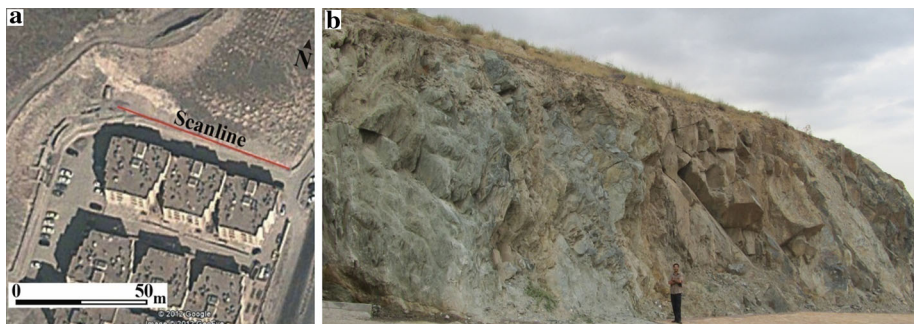

Fig. 2 Overview of the residential complex (**a**) and vertical trench before the construction of the buildings (**b**) (north view)

slope dip and height, discontinuity plane dip, and dip direction) and of some geo-mechanical properties (e.g., cohesion, friction angle, and water pressure) on the stability of this trench by probabilistic and sensitivity analysis methods.

2 The study area and region geology

Mashhad is one of the most important cities in Iran. Concerning morphological aspects, the central and northern parts of this city are located in the Mashhad plain and the southern part is in a mountainous region. In recent years, given the construction of several residential/commercial buildings, especially in the south region, even a minor slide might lead to major difficulties. Therefore, it is necessary to assess the stability of natural or excavated slopes in order to prevent any damage to areas of residential complexes.

Negin residential complex is located between the southwest and south (SW–S) part of Mashhad city in a mountainous region. The studied area has geographic coordinates, 38°18′02″N, 59°29′37″E (Fig. 1). Based on geomorphological study, this area is located in the north part of the Binaloud Mountains. Geological studies have shown that this area consists of a collection of mafic, ultramafic, and metamorphic rocks, and quaternary deposits. According to the studies on the regional rocks, there is a periodic arrangement of meta-basic and meta-sediment rocks in the trench of the Negin residential complex. The meta-basic rocks are made of meta-basalt, meta-gabbro, and meta-sediment rocks (foliated phyllite). The metamorphic rocks in the contact area and the rock masses in this region are highly weathered. Concerning the tectonic aspects in the region of interest, we can observe the influence of thrust faults and local folding. The faults in this region have a northwest–southeast trend and their direction is north–northeast to south–southwest (Afshar-Harb 1984; Darvishzadeh 2003).

3 Methodology

This study was performed according to standard geological techniques. We divided the data collection and the analysis into four steps: field surveys, laboratory tests, trench stability analysis, and sensitivity analysis.

3.1 Field surveys

The first and major step in slope stability analysis is field survey. Because discontinuities play important roles in rock slope instability (Goodman 1989; Hudson and Harrison 1997), we characterized the discontinuities systems according to the International Society of Rock Mechanic standards (ISRM 1978). First, properties of 110 discontinuities along the trench were distinguished by the scan line method, and then the parameters of strike, dip, opening, filling, roughness, spacing, and persistence were evaluated. Next, the petrology and geological structures were recognized. The trench dip and strike and groundwater conditions were also determined during the studies. Finally, many samples were collected for laboratory tests.

3.2 Laboratory tests

Based on the Mohr-Coulomb method, the shear strength of the discontinuities is one of the most important parameters on the stability of the rock slopes and can be calculated by cohesion and friction angle parameters (ZareNaghadehi et al. 2011).

The procedure was briefly as follows: to evaluate the geotechnical parameters of the rock mass, two exploratory boreholes were excavated and core samples were collected.

Cores were excavated from an area identical to the trench outcrop and were dried at 105 °C for 24 h. Cores with 54–63 mm diameter and 100–150 mm height were used. Samples were kept in plastic wrap immediately after the sampling to preserve the natural humidity. Moreover, some samples were collected by hammer from the outcrops of the rock trench—without any weathering—in order to evaluate the type and petrography parameters of the rock.

In this study, an unconfined compressive strength test was carried out according to the method suggested by ASTM D2938 (2002) using samples without any discontinuities. The test was performed under the natural humidity of the cores. Also, core samples with a single discontinuity were chosen for direct shear tests to evaluate the shear strength of joint surfaces according to the ASTM D5607 method (1995). Samples were divided into two parts: for saturated condition, some samples were put in water for 48 h, and for dry condition, samples were dried at 105 °C for 24 h. The shear strength of samples was measured by repeating the shear strength test under different normal pressures. In addition, the unit weight test was performed in both dry and saturated conditions. Finally, the strength properties of the rock samples, including the cohesion (C), friction angle (φ), discontinuity's surface, unit weight (γ), and unconfined compressive strength (σc) were determined.

3.3 Trench stability analysis

Usually, kinematic analysis seems to be a suitable method for general evaluation of the instability potential for blocks and hillside geometry (Hudson and Harrison 1997). Kinematic analysis, without the effective forces on the slide, deals with the relationship between the slope, dip strike, and discontinuity surfaces to calculate the possibility of failure occurrence (Goodman 1989; Hudson and Harrison 1997; Ping 1997; Gischig et al. 2011).

The possibility of movement in an unstable mass can be evaluated by kinematic analysis. So, in this study, we used this analysis to determine the instability mechanism. To perform the kinematic analysis, we used Wyllie and Mah (2004) method. The 'occurrence probability' of planar failure, wedge failure and toppling failure were estimated by using Dip software (v.5.072; Rocscience, Canada).

Next, the deterministic analysis of safety factor was performed based on the Wyllie and Mah (2004) method. This method employs the constant values for each parameter and the results represent the specified values for factor of safety (FS) (Ping 1997). In this study, we used the following parameters: unit weight of rock, height, dip and dip direction of slope, dip and dip direction of discontinuities, cohesion, friction angle, and underground water condition. The seismic effect was not considered for any analysis. Moreover, the deterministic, probabilisti,c and sensitivity analyses were carried out based on Mohr–Coulomb criteria using Swedge (v.4.078; Rocscience) and RocPlane software (v.2.029; Rocscience).

One of the major problems in rock slope stability analysis is the uncertainty in determining the geotechnical parameters, which is related to the nature of former slope composition materials (Park and West 2001; Coates 1981; Hoek 2007; Li et al. 2009). The orientation of discontinuities is variable even inside the joint set (Park and West 2001; Wyllie and Mah 2004). Also, along the discontinuity surface, based on joint filling and roughness type, the rock mass shear strength can be changed (Singh 2006; Hoek 2007). The rock mass strength properties and external forces conditions, such as water level and earthquake force, change over time (Hoek 2007). Consequently, choosing an acceptable value for effective parameters for stability is a major challenge in slope analysis using the deterministic method (Park and West 2001; Einstein and Baecher 1982; Hoek 2007). Thus,

in this study, to determine the probability of FS, the probabilistic approach was used to overcome the lack of deterministic methods as described in the literature (Piteau and Martin 1977; Pentz 1981; Savely 1987; Vanmarcke 1980; Grenon and Hadjigeorgiou 2008).

Probabilistic analysis is a systematic method for modeling the evaluation of uncertainties and variability of parameters. In this analysis, FS is expressed as a random variable and it can replace the probability of failure for evaluation of slope stability values. Possibility of instability is defined with FS ≤ 1 (Park and West 2001). Probabilistic analysis includes two steps. First, we have the analysis of geotechnical parameters to determine the basic statistical elements (mean and standard deviation) and the probability density function (PDF) for predicting the random characters of geotechnical parameters (Tabba 1984). Second, probabilistic analysis of instability is determined based on the results of the first step. In this paper, the Monte Carlo simulation method was the basis of our probabilistic analysis. This method uses a statistical distribution to define the parameters that control the slide stability. Thus, FS was determined by random values from statistical distributions (Grenon and Hadjigeorgiou 2008).

We determined the safety factor by a probabilistic method using the various values for each parameter because, without this method, slide analysis has a high level of uncertainty (Park and West 2001; Coates 1981; Hoek 2007; Duzgun and Bhasin 2009; Li et al. 2009). The required parameters for this analysis were groundwater condition, slope height, dip and dip direction of slope, discontinuities dip and dip direction, cohesion, and internal friction angle. During the analysis, unit weight and dip direction for slope and joint sets were considered constant, then for the others, the normal distribution was calculated based on interval changes of field study, laboratory data, and basic statistical parameters.

Given that in some discontinuities there are no filling materials, to clearly understand the effect of non-cohesion discontinuities on instability in both probabilistic and sensitivity analyses, the minimum hypothetical value of cohesion parameter was considered as zero. The probability of trench instability was investigated after inputting the basic statistical parameters by probabilistic analysis.

3.4 Sensitivity analysis

As mentioned in the previous section, the probabilistic analysis can predict the probability of slope failure by using the range of parameters and probability distribution of FS, while, in sensitivity analysis, the maximum and minimum values of each parameter have been used to determine the critical condition of a slope. So, the relationships between parameters and their effects on FS can be evaluated (Wyllie and Mah 2004).

Moreover, it is important to perform the sensitivity analysis because, with probabilistic analysis results, the connection between the parameters cannot be determined. To achieve this goal, the sensitivity analysis was performed in three steps.

First, between all the parameters such as slope height, dip for slope and joint sets, cohesion, friction angle, and groundwater condition, just one parameter was considered variable and all the others were made constant. For the variable parameters, all the basic statistical factors were defined and then FS was calculated according to these data. This procedure was carried out sequentially for all the mentioned parameters.

Second, the impact of variability of joint set dip, height, and slope dip parameters on FS for different values of cohesion was evaluated by considering the constant value of friction angle. Each time, one of them has a variable value and the other two were made constant. Similarly, in the third step, the impact of variability of joint set dip, height, and slope dip

parameters on FS for different values of friction angle was calculated by considering the constant value of cohesion. In all three steps of the sensitivity analysis, minimum and maximum values, and average and standard deviations for parameters were used from field and laboratory data. Also, during the analysis, dip direction of slope and joint sets were considered constant. The sensitivity of FS along the variations of parameters was determined and the most influential factors were defined.

4 Results and discussion

4.1 Slope geometry

Based on field studies, the trench which was 80 m long and with 181° dip direction was located on the north side of the Negin residential complex. The rock masses in the study area consisted of weathered meta-basic and meta-sediment rocks. The presence of several sets of discontinuities has created blocks with polygonal shape. The volumes of the blocks were approximately between 0.004 and 0.70 m^3.

4.2 Discontinuity analysis

To investigate the discontinuity properties in the study area, the scan line with a trend of 091° and a plunge of 00° was considered. Then, plotting the poles of 110 structural discontinuities in Dips software, the discontinuity planes data were statistically analyzed (Fig. 3). According to the analysis performed on these structural data, there were three joint sets with high dip and one joint set with moderate dip (Table 1). The slightly weathered joint set 1 (J1) has partially open joint plans (0.1–0.5 mm of aperture) and moderate persistence between 3 and 10 m. The joint spacing was between 0.6 and 1.05 m with an average spacing of 0.85 m and the common JRC was between 12 and 14. Joint set 2 (J2) was slightly weathered and almost parallel to the slope direction. The joint surface was usually open (0.5–2.5 mm) and oxide, and the persistence was between 1 and 3 m (low persistence). Also, joint spacing was between 0.15 and 0.55 m (moderate spacing) with average spacing of 0.3 m and the common JRC was between 12 and 14. Joint set 3 (J3) with slight to moderately weathered surfaces was usually open moderately wide (between 1 and 5 mm) with gouge filling. Persistence of joints was hardly noticeable. The joint spacing was between 0.4 and 0.8 m (moderate–wide spacing) and common JRC was between 10 and 12. Joint set 4 (J4) with moderately weathered joint surfaces was usually open (between 0.5 and 2.5 mm), with gouge filling and oxide. The persistence of joints was 1–3 m (low persistence) and the spacing was from 0.7 to 0.13 m (wide spacing), with average spacing of 0.95 m and a common JRC of 12–14 (Fig. 4).

4.3 Laboratory tests

According to the results of the 10 unconfined compressive strength (UCS) tests (ASTM 2002) performed, we calculated the minimum and maximum values of 9.83E2 and 1.441E3 MPa, respectively, with an average of 1.16E3 MPa. In nature, discontinuity surfaces are never clean and smooth, so the excavated rock samples and the roughness or smoothness of a joint surface can increase or reduce its shear resistance. For discontinuities surfaces, the correct measurement of the cohesion and friction angle seems to be a critical
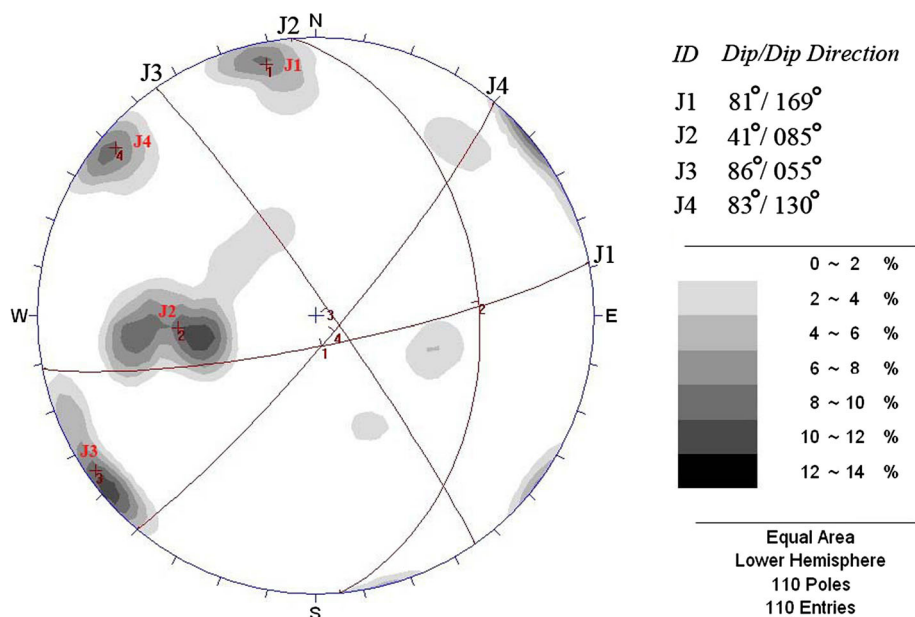
Fig. 3 Stereographic image of the region discontinuities

issue (Singh 2006; Hoek 2007). Therefore, to overcome this problem, the friction angle (φ) and cohesion (C) were determined using a direct shear strength test (ASTM 1995) on the rock sample, and we used back analysis to obtain the real friction angle and the cohesion of the discontinuity surface. Then, optimal rates of the parameters C and φ were determined in dry and saturated conditions (Table 2).

According to the results of direct shear tests on filling joints, the sample cohesion and friction angle were equal to 9.7 E–3 MPa and 32°, respectively. The maximum and minimum values that have been obtained for these parameters will be discussed in Sect. 4.4.3. Moreover, the unit weights of rocks in dry and saturated condition were 27.15 and 27.45 KN/m^3, respectively.

4.4 Trench stability analysis

4.4.1 Kinematic analysis

The first step in the analysis of failure and danger mechanisms for an unstable rock slope is the comprehensive identification of rock mass kinematic behavior (Goodman and Kieffer 2000; Goodman 1989; Wyllie and Mah 2004). Therefore, the kinematic and stereographic analyses were performed to evaluate the potential of slope instability (Wyllie and Mah 2004). To perform these analyses, the slope was considered with 86° degree dip and 181° degree dip direction, and the friction angle was presumed to be 32°.

As shown in Fig. 5a, the kinematic analysis revealed that joint set 1 has a potential of planar slide, given that its dip was less than the trench face's dip (81° < 86°); this joint set was visible at the slope surface, and the dip direction of joint set 1 differed from the dip direction of the trench face by less than 20° (181°−169° = 12°). Therefore, only this joint set might cause a plane slide. Furthermore, for joint sets 2, 3, and 4, basic stability analysis

Table 1 General characteristics of the joint system

| Joint set	J1	J2	J3	J4
Dip (deg)	81°	41°	86°	83°
Dip direction (deg)	N169°	N085°	N055°	N130°
Trace length (m)	3–10	1–3	–	1–3
Spacing (m)	0.6–1.05	0.15–0.55	0.4–0.8	0.7–1.3
Aperture (mm)	0.1–0.5	0.5–2.5	1–5	0.5–2.5
Roughness	Slickenside, stepped	Smooth, stepped	Rough, undulating	Smooth, planar
JRC	12–14	12–14	10–12	12–14
Infilling	No	No	Gouge	Gouge
Groundwater	Dry	Dry	Dry	Dry
Weathering	Slightly weathered	Slightly weathered	Slightly-moderate weathered	Moderate weathered
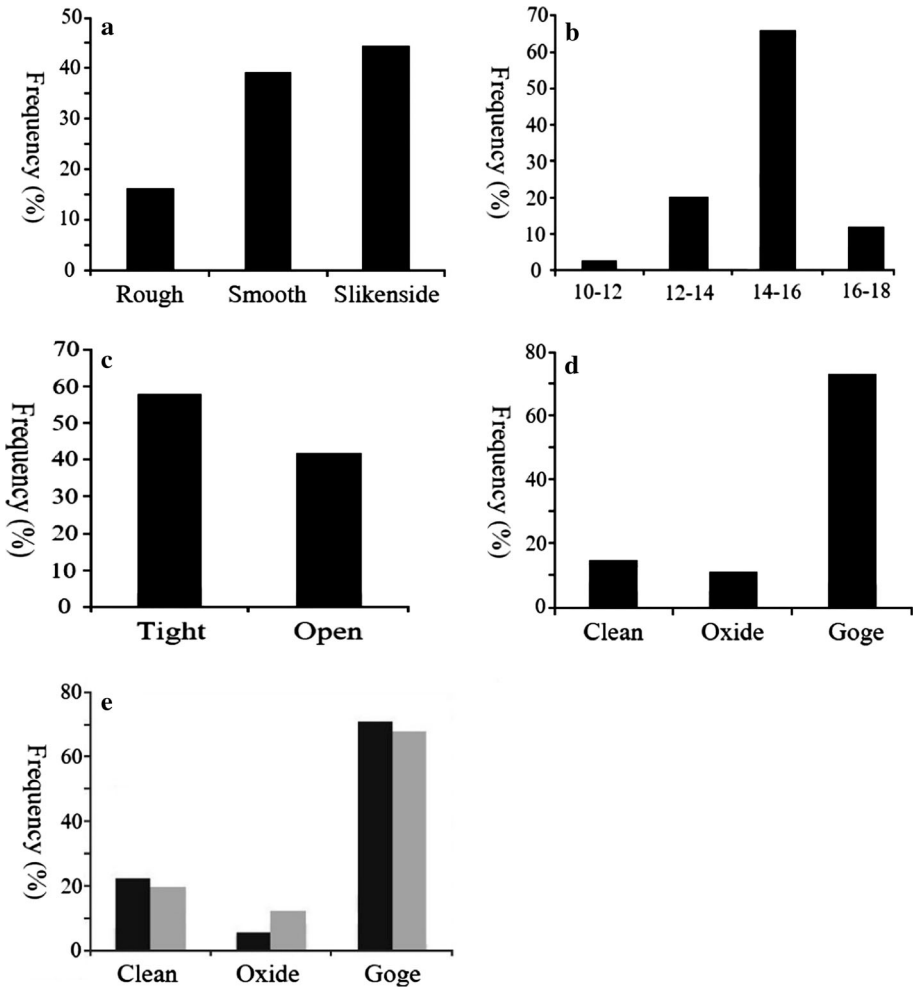
Fig. 4 Discontinuity surfaces properties, **a** the joint set roughness, **b** the joint set JRC, **c** the aperture type, **d** the filling type, **e** the relationship between the aperture and filling(*black* open, *gray* tight)

Table 2 Properties of rock discontinuities

Parameter	Condition	
	Dry	Saturation
C (Mpa)	9.7E−3	9E−3
φ (deg)	32	30
γ (KN/m^3)	27.15	27.45

showed that there was no possibility of plane failure, because the dip directions of these joint sets differed from the dip direction of the trench face by more than 20°. We also studied the possibility of wedge failure occurrence for the four joint sets. Kinematic analysis showed (Fig. 5b) that there was the possibility of wedge failure, which resulted

from intersections of joint sets 1 and 3 and joint sets 1 and 4. In joint sets 3 and 4, the intersection plunge is at a steeper angle than the trench face and there is no daylight on the face. In kinematic analysis, discontinuities were assumed to be dry, cohesion less, and continuous and rock mass to be rigid. Thus, according to this analysis, if there was the slide possibility, a more detailed analysis would be necessary (e.g., deterministic and probabilistic analysis). The deterministic and probabilistic analysis was also performed in order to illustrate the slide accuracy in joint set 1 and also the wedge failure possibility for joint sets 1 and 3 and joint sets 1 and 4.

4.4.2 The deterministic analysis

To determine the FS with fixed values of effective parameters, we used deterministic analysis, according to Wyllie and Mah (2004) protocols. The analysis was carried out based on Mohr–Coulomb criterion, taking advantages of Swedge (v.4.078; Rocscience;) and RocPlane softwares (v.2.029; Rocscience).

4.4.2.1 Swedge software analysis According to dry condition analysis, the FS values for wedge failures which resulted from joint sets 1 and 3 (wedge A) and 1 and 4 (wedge B) were near the boundary state (Table 3). Slide probability increased when half the height of the tension crack was filled with water. In mid-saturated condition, FS was less than 1. We also found that the potential of wedge failure occurrence on plane joint set 1 was more than the others because the dip direction of joint set 1 lay within the dip direction of the face and the trend of the intersection line between joint sets 1 and 3. For wedge B, the slide can happen on the intersection line of two joint sets (Fig. 6).

4.4.2.2 RocPlane software analysis According to the previous results, the potential of plane sliding was high in this trench, because the FS in dry and mid-saturated conditions was less than 1. The results are shown in Table 4 and the shape of the plane failure is shown in Fig. 7.

4.4.3 Probabilistic analysis

During the probabilistic analysis, the slope height range was considered from 10 to 20 m. The slope dip range used was between 82° and 90° (mean: 86°), and a cohesion interval was put at between 0 and 1.3 E−2 MPa (mean: 1 MPa). Also, the range of friction angle was considered from 27° to 35°.

The groundwater condition was assumed to be between dry and saturated. The ranges of the joint sets dip were equal to the field evidence: for joint set 1 it was 71°–85° (mean: 81°, SD = 6.7, df = 16); for joint set 3, 80°–88° (mean: 86°, SD = 2.13, df = 20), and for joint set 4, 76°–87° (mean: 83°, SD = 6.48, df = 14). During the analysis, the dip direction value was considered constant for both slope and discontinuities and was 181° as shown in Table 1. After inputting the data, probabilistic analyses for the wedge and plane failure were carried out using the Swedge and Rocplane software, respectively. Performing of 10,000 samplings for each parameter in the software, the instability probability was evaluated for wedge and plane failure and the results are shown in Figs. 8 and 9. According to the wedge failure analysis, the average of FS in dry and saturated conditions were 0.50 and 0.35 for wedge A, respectively (Fig. 8a, b). For wedge B, the average FS in dry conditions was regularly 0.75 and in saturated conditions 0.67 (Fig. 8c, d). Based on these
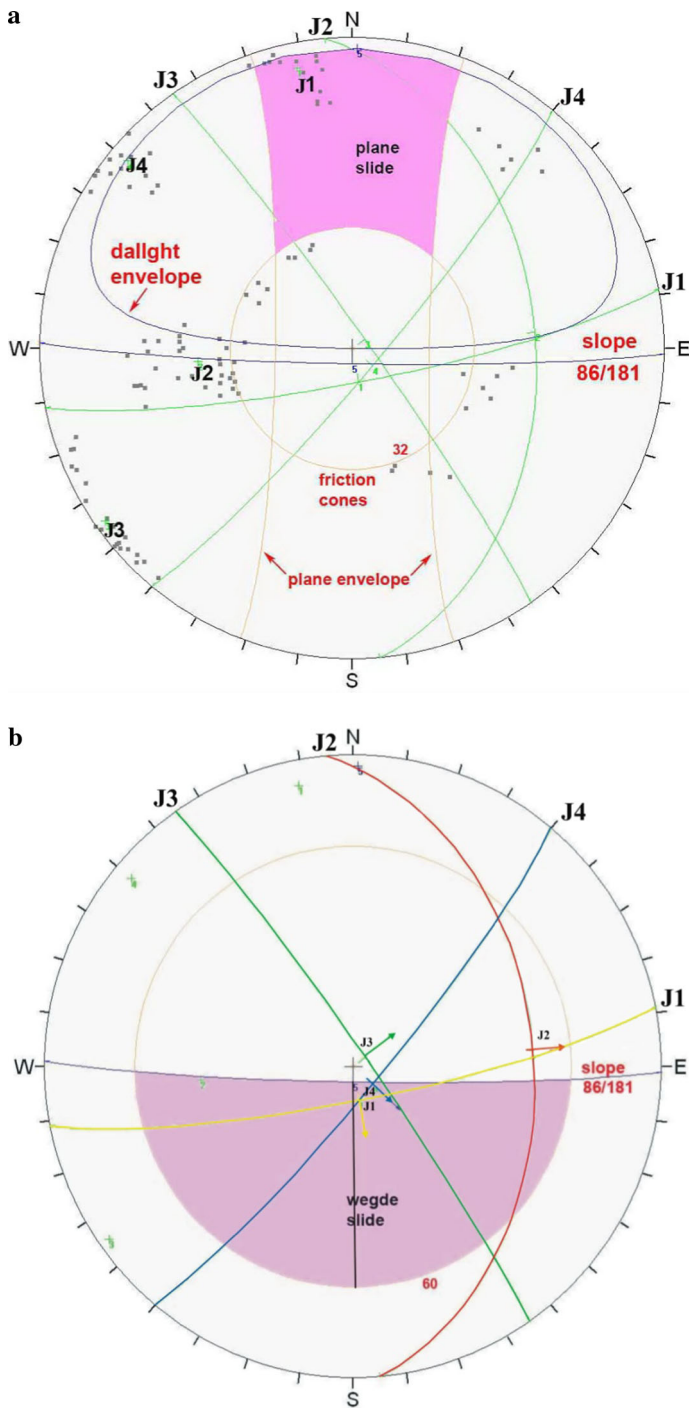
Fig. 5 Kinematic analysis of instability, **a** probability of plane slide, **b** probability of wedge slide

Table 3 Deterministic analysis for the wedge slide

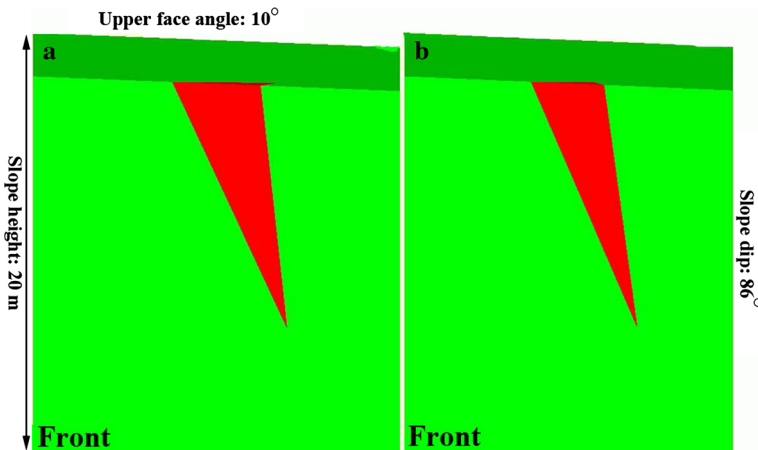| Wedge resulting from the joint sets	Fs (dry)	Fs (saturated)	Slide direction (plunge/rend)	Slide mass weight (ton)
1 and 3	1.03	0.93	On joint set 1	50.18
1 and 4	1.13	0.77	Along the intersection line of two planes 81/169	88.21

Fig. 6 Two-dimensional (2D) schematic image of wedge failure by using the Swedge software, **a** joint sets 1 and 3, **b** joint sets 1 and 4. (*Green* the slope, *red* unstable rock mass)

Table 4 Deterministic analysis for the plane slide

Plane slide surface	Fs (dry)	Fs (saturated)	Mass weight (ton)
Joint set 1	0.53	0.42	49.95 (dry) 50.54 (saturated)

results, the potential of failure for wedge A was more than wedge B. The possibility of wedge failure for dry and saturated conditions is presented in Table 5.

The analysis of the plane failure possibility showed that, in dry condition, this probability was about 90.15 % and in mid-saturated, this parameter was 100 % (Fig. 9). The average of FS in dry condition for plane failure was equal to 0.4 and this value decreased in mid-saturated condition to 0.28. So, the plane failure possibility was observed to be high in this trench and water pressure was the major contributing factor to this instability.

4.5 Sensitivity analysis

Sensitivity analysis was performed to define the impact of effective parameters on FS. According to this analysis, although the probability of the plane failure and failure of wedge A were calculated to be high, based on our filed surveys results, a plane failure was more probable. So the following sensitivity analysis was done just for plane failure.
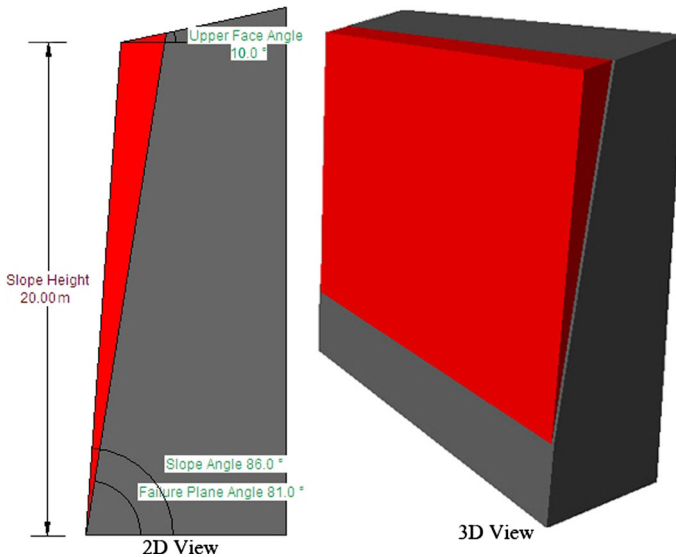
Fig. 7 The image of the unstable rock mass resulted from plane failure

Moreover, we evaluated the effects of cohesion, friction angle, and water pressure, dip of joint set 1, dip and height of slope on stability. The analysis results of the first step are presented in Fig. 10. The basic statistical parameters we used were the same as we used for the probabilistic analysis. In this analysis, the condition was considered dry and the dip direction of slope and joint sets was constant. This value was 181° for slope and for the discontinuities shown in Table 1.

Based on our results, upon reduction of slope to 84°, FS had a significant non-linear increase, so that, with a reduction of slope dip to 82°, the FS was increased to 2.11 (Fig. 10a). There was a linear correlation between the changes of FS value and the height reduction of the trench. The FS value had increased from 0.53 to 0.91 in dry condition when the slope height decreased from 20 to 10 m (Fig. 10b).

As shown in Fig. 10c, reducing the dip parameters for joint set 1, the 81° FS value significantly increased; however, for values less than 81°, just small changes occurred in FS values. Indeed, we can observe a direct correlation between the parameters and a direct relationship between cohesion and FS: whenever cohesion became zero, the FS became 0.12 (Fig. 10d).

The cohesion factor alone did not have any significant effect on FS, while increasing the cohesion factor caused a small rise in FS. Figure 10e shows that the friction angle values in the defined range did not have any considerable effect on FS. The groundwater parameter had a strong negative effect on FS (Fig. 10f). The FS value suddenly drops to zero when 27 % of the trench height was saturated with water. In step two, the variation effect of three parameters of joint set 1 (i.e. dip, slope dip, and height) was evaluated in stability by choosing 5 different values for the cohesion factor (0, 3E−3, 6E−3, 9E−3, and 1.2 E− 2 MPa). As a third part, the influence of the same parameters of step two on stability was analyzed, but this time values of friction angle were considered as variable (26°, 28°, 30°, 32°, and 34°). During these steps for three parameters of joint set 1, dip, slope dip and height, the ranges were considered identical to the probabilistic analysis (Fig. 11).
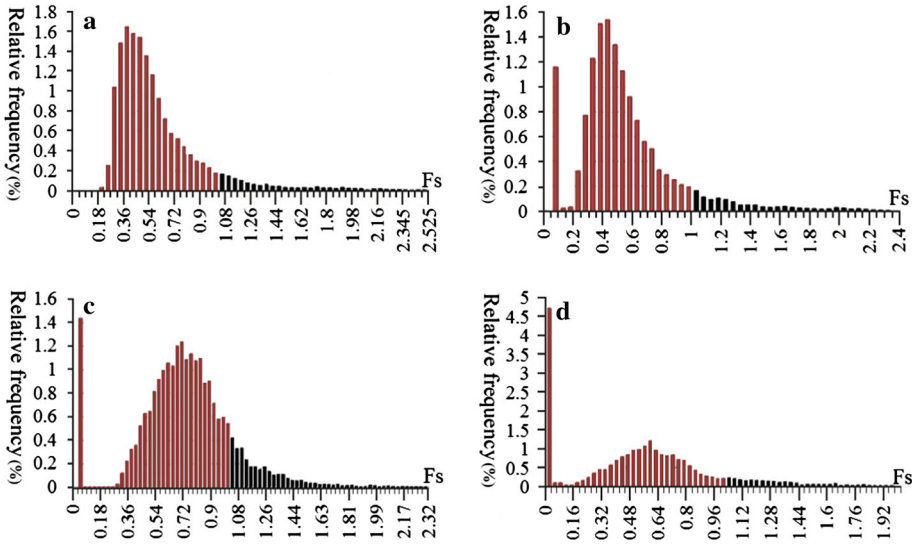
Fig. 8 Probabilistic safety factor histogram for the wedges slide resulted from **a** joint sets 1 and 3 in dry condition, **b** joint sets 1 and 3 in saturated condition, **c** joint sets 1 and 4 in dry condition, **d** joint sets 1 and 4 in saturate condition
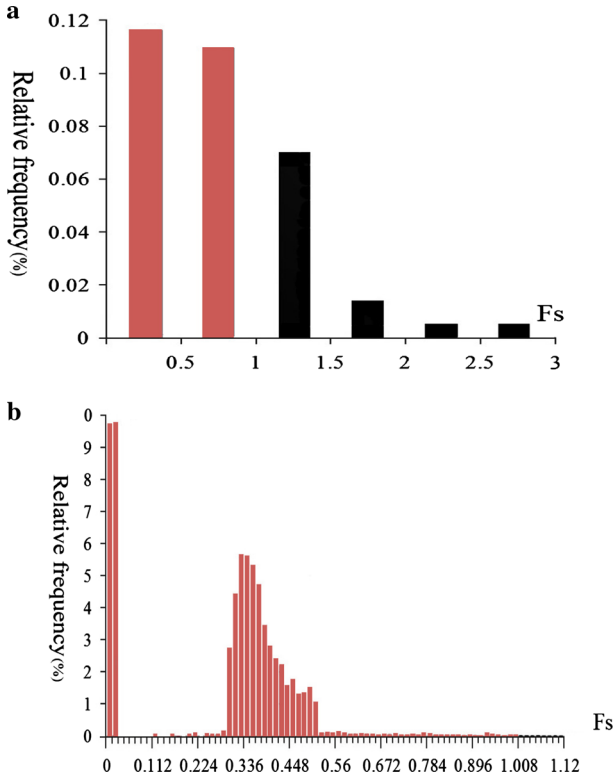

Fig. 9 Probabilistic safety factor histogram for plane slide, **a** in dry condition, **b** in saturated condition

Table 5 Probabilistic analysis for wedge slides

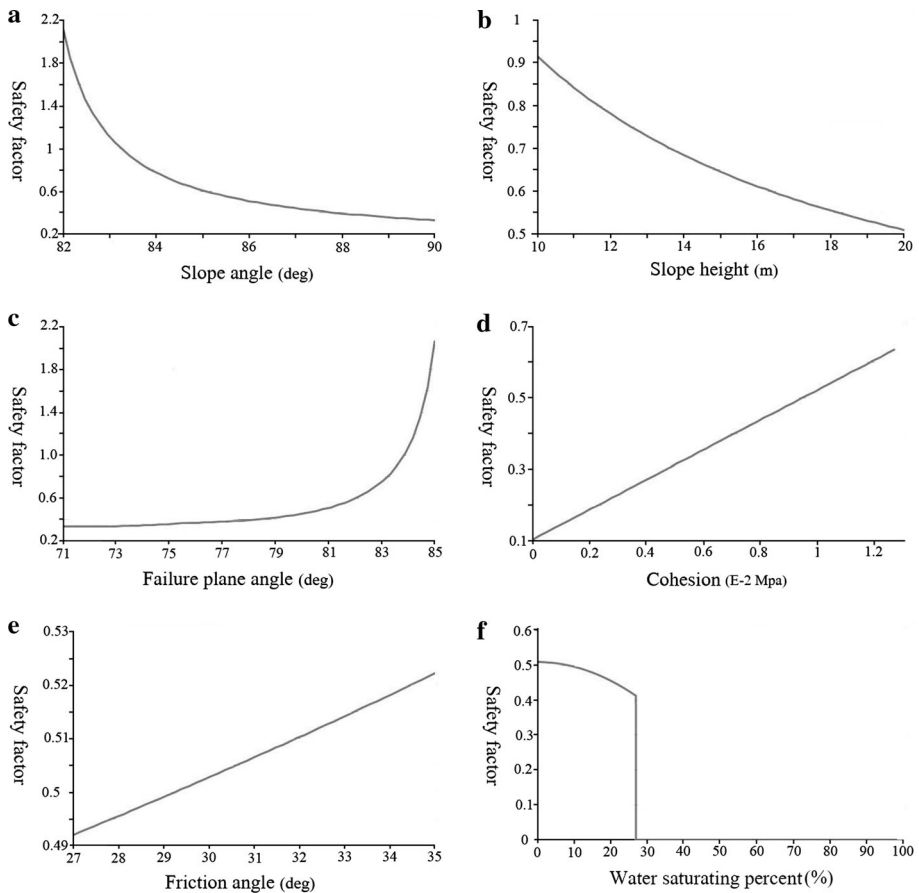| Wedge resulted from joint sets	Probability of failure in dry condition (%)	Probability of failure in saturated condition (%)
1 and 3	89.18	94.20
1 and 4	83.62	92.87

Fig. 10 Sensitivity analysis for plane slide: **a** the influence of slope dip on safety factor, **b** the influence of slope height on safety factor, **c** the influence of the joint set 1 dip on safety factor, **d** the influence of cohesion of the joint set 1 on safety factor, **e** the influence of friction angle of joint set 1 on the safety factor, **f** the influence of water pressure on the safety factor

Based on the results, variations of friction angle and slope dip were always simultaneous with each other in all the graph plots. Therefore, as shown in Figs. 11a and 10a, during the simultaneous variations of slope dip and friction angle, slope dip is the only parameter that can play an effective role on FS. The effect of changes in slope dip on FS was also evaluated with different cohesion values (i.e., 0, 0.30, 0.60, 0.90, and 1.20 E−2 MPa; Fig. 11b).
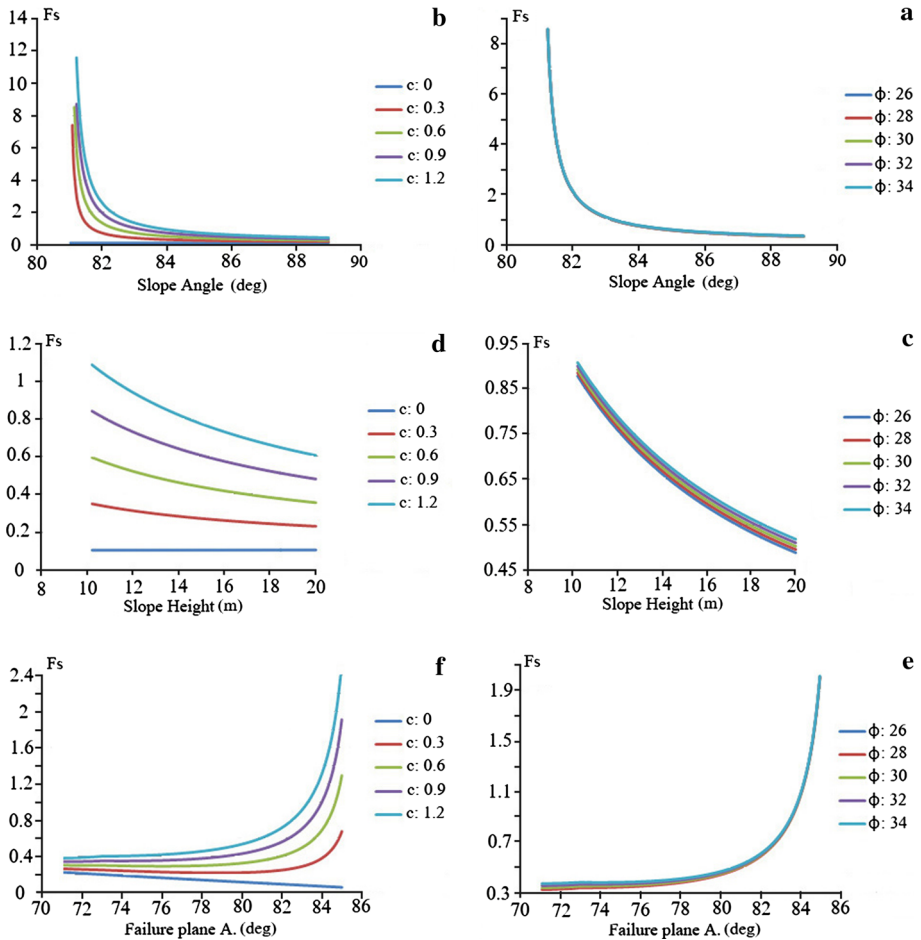
Fig. 11 Sensitivity analysis for the effects of changes in the various parameters on the safety factor: **a** sensitivity analysis for the slope height in different friction angles (Φ in °), **b** the sensitivity analysis for the slope dip in different cohesion (**c**) values (c in E−2Mpa), **c** the sensitivity analysis for the slope height in different friction angles (Φ in °), **d** the sensitivity analysis for the slope height in different cohesion values (c in E−2Mpa), **e** the sensitivity analysis for the joint set 1 dip in different friction angles (Φ in °), **f** the sensitivity analysis for the joint set 1 dip in different cohesion values (c in E−2Mpa)

Based on these results, increasing the cohesion factor, the effect of slope dip changes on FS heightened. For example, when the value of C was equal to 3E−3 MPa in slope dip 81°, FS was equal to 1, while for 1.2E−2 MPa cohesion the slope dip must be 84° for FS equal to 1.

Figure 11c shows the effects of slope height changes on FS in different friction angles (i.e., 26°, 28°, 30°, 32°, and 34°). Based on these results, the analysis procedure of the impact of changes in trench height on FS was independent from the friction angle effect, while the cohesion played a significant role. Increasing the cohesion factor, the effect of a reduction in trench height rose on FS (Fig. 11d).

We can see the effects of changes in the dip of joint set 1 on the FS in 5 different friction angles (Fig. 11e). These results showed that the variation of friction angle during the dip analysis of joint set 1 did not have a major effect on FS (Figs. 10c and 11e).

Figure 11f shows the effect of changes in the dip of joint set 1 on the FS in different cohesion values (i.e., 0, 0.30, 0.60, 0.90, and 1.20 E−2 MPa). It is observed that with cohesion of zero, as the failure plane dip increased, the FS decreased. However, when cohesion was not zero, upon the increase of the failure plane dip from 71° to 79°, there was no maked change in the FS. By increasing the failure plane dip from 79° to 85°, FS increased in a non-linear way.

Based on the results presented in Fig. 11b, d, f, the increase in the cohesion values caused a rise in FS and in the effect of the three parameters of slope dip, slope height, and joint set 1 dip on FS. Because the cohesion factor is an inherent parameter of rock mass, the presence of groundwater, slope dip, and joint set 1 dip were the most influential parameters on stability. It should be noted that the slope height has a less intense effect than the other above-mentioned parameters.

5 Conclusion

Geotechnical parameters of rock and discontinuities are considered as effective factors in rock slope stability; on the other hand, the uncertainty of these parameters and the difficulty in choosing suitable quantitative values for them are important problems in rock slope stability analysis. In this study, we evaluated the effects of different parameters on slope stability of the southern margin of Mashhad city, based on the sensitivity analysis. The results clearly show that there is a potential of wedge failure and a possibility of plane failure. Deterministic analysis revealed that, in dry condition, FS is less for wedge A than for wedge B. Also, based on the probabilistic analysis, the plane failure possibility was higher than for a wedge slide and this result also has a strong correlation with field studies. So, using the sensitivity analysis, we analyzed the most effective parameters on plane failure. Based on these results, the slope dip of 84° was recognized as a milestone given that, for values less than this, a significant non-linear increase in FS appeared and the stability of trench improved rapidly. There is an inverted relationship between slope height and FS so that half the slope height makes the FS double. The presence of groundwater can be considered as the other external parameter that increases the instability of the rock mass. The final results of the sensitivity analysis showed that the friction angle did not have any marked effect on FS but for different values of cohesion effect of slope dip, slope height, and joint set 1 dip, its effect on stability rose significantly. Thus, in conclusion, the reduction of slope dip and controlling of water leakages in this trench seem the most influential methods for improvement of the slope stability in this case study.

All in all, our study presented a useful approach for assessing the rock slope stability in an urban area. This approach of assessing the natural hazards in urban zones for residential complexes (especially those near to mountainous ridges) has brought a new attention for the scientific and industrial communities.

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