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Analytical solution of seismic stability against overturning for a rock slope with water-filled tension crack

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Abstract. Steep rock slope with water-filled tension crack will happen to overturn around the toe of the slope under seismic loading. This failure type is completely different from the common toppling failure occurring in anti-dipping layered rock mass slopes with steeply dipping discontinuities. This paper presents an analytical approach to determine the seismic factor of safety against overturning for an intact rock mass slope with water-filled tension crack considering horizontal and vertical seismic coefficients. This solution is a generalized explicit expression and is derived using the moment equilibrium approach. A numerical program based on discontinuous deformation analysis (DDA) is adopted to validate the analytical results. The parametric study is carried out to adequately investigate the effect of horizontal and vertical seismic coefficients on the overall stability against overturning for a saturated rock slope under two water pressure modes. The analytical results show that vertically upward seismic inertia force or/and second water pressure distribution mode will remarkably decrease the slope stability against overturning. Finally, several representative design charts of slopes also are presented for the practical application.

Keywords: rock slope with water-filled tension crack; overturning failure; discontinuous joint plane; analytical solution; design charts; horizontal and vertical seismic coefficients

1. Introduction

Slope failures induced by rainfall, groundwater table change or earthquake have caused a large number of casualties and heavy property loss every year (Wang and Li 2009, Huang *et al*. 2011, Yalcin 2011). This type of hazard is also frequently encountered in rocky slopes in many countries all over the world (Goodman and Kieffer 2000, Wyllie and Math 2004). In general, rock slope failures contain five types such as circular rotation and buckling failure occurring in a moderate to highly weathered soft rock, failure through plane sliding, wedge sliding or toppling along distinct joint planes or intersections of planes for hard rocks (Hoek and Bray 1977, Goodman 1989, Ling and Cheng 1997, Wyllie and Math 2004, Liu *et al*. 2009, Latha and Garaga 2010, Yang and Pan

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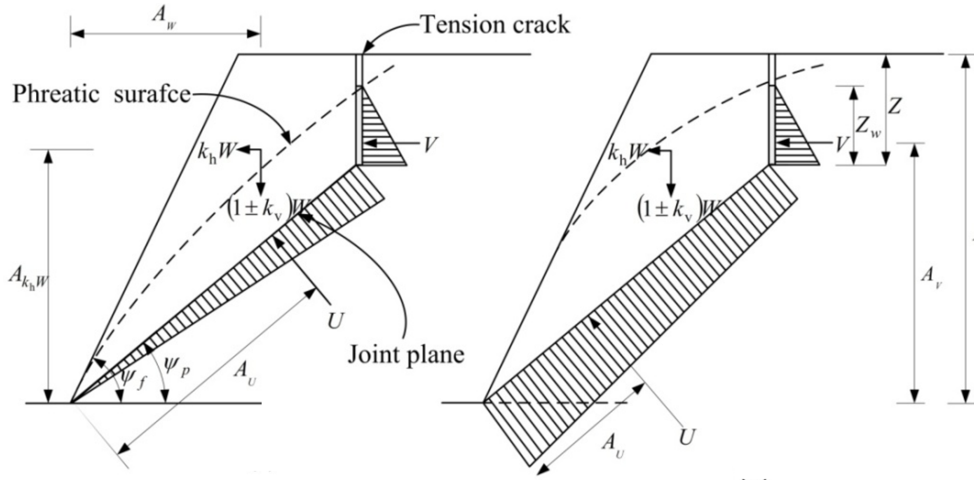
2015). In the five failure modes, plane failure as a special case of the more general wedge failure, is also frequently observed by engineers (Goodman and Kieffer 2000). In practice, various analytical approach and graphic method (design charts) were developed to evaluate the sliding stability of rock slope along a planar failure surface, in which the effects of surcharge, hydraulic distribution, earthquake and other factors on stability had already been taken into consideration (Sharma *et al*. 1995, Wyllie and Math 2004, Luo *et al*. 2010, Zhang *et al*. 2011, Wu *et al*. 2011, Zhao *et al*. 2015). The representative analytical methodology, based on two-dimensional limit equilibrium analysis, was presented by Hoek and Bray (1977) to assess the stability of rock slope along a joint plane intercepted by a tension crack, in which pore water pressures in the tension crack and along the joint plane are also considered. Later, Ling and Cheng (1997) improved the method in which horizontal and vertical seismic inertia forces were included herein to calculate the seismic factor of safety against sliding along a joint plane. If such a rock slope was considered as unstable in seismic stability analysis, some countermeasures such as anchors should be taken to enhance the stability and prevent the plane failure of slopes. Thus, based on the actual needs, a series of cases on the stability of such an anchored rock slope were considered by Shukla *et al*. (2009) and Shukla and Hossain (2011a, b), and the corresponding analytical formulations were also presented. Though the achieved expressions for planar sliding stability analysis had been widely recognized in both engineering and academic fields, the adopted theoretical assumption had its limitation. It was assumed that the forces all act through the centroid of the sliding mass, in turn, there are no moments that would tend to cause rotation of the block, and hence slope failure is induced by sliding only. In fact, this assumption may not be strictly true for the actual slopes (Hoek and Bray 1977), such as a steep and hard rock mass/block slope with a dipping discontinuous joint plane intercepted by a vertical tension crack, especially when the slope was under water pressures or/and seismic loading condition (Fig. 1). In that case, a whole overturning failure around the slope toe, different from the common toppling failure occurring in anti-dipping layered or jointed rock mass slopes with steeply dipping discontinuities (Hoek and Bray 1977), may be extremely possible to occur. And the essential difference between overturning failure and toppling failure was the role that the gravity has played in the slope stability; the former intended to maintain the stability, whereas the latter was completely opposite. In fact, some steep slopes composed of intact rock mass with few dipping discontinuous planes as shown in Fig. 1 were observed or/and speculated to slide or/and overturn during several typical earthquakes (Maugeri *et al*. 1993, Shi *et al*. 2008, Huang 2009). However, the assessment of seismic stability against overturning for such an intact rock mass/block slope with a dipping discontinuous joint plane intercepted by a tension crack filled with water is rarely reported so far.

In this paper, the analytical expressions of stability factor of safety against overturning for a rock slope with water-filled tension crack subjected to seismic inertia forces are derived, the parametric study is carried out and the typical design charts are developed for the practical use of this solution.

2. Analytical formulation

Fig. 1 shows the geometry of slope used in the present analysis. For consistency, the notations and assumptions used closely follow those presented by Hoek and Bray (1977) and Ling and Cheng (1997). A potential sliding rock block of height H and unit weight γ is considered, the slope face and joint plane are inclined at an angle (dip) ψ_f and ψ_p, respectively. The vertical tension



(a) Mode 1 of water pressure (b) Mode 2 of water pressure

Fig. 1 Forces and arms for typical rock slope under seismic loading

crack extends from the crest to depth Z behind the slope face, and intersects to the joint plane, where W is the weight of potential sliding block.

Note that the vertical tension crack is filled with water to a height Z_w, where a lateral resultant force V is defined; at $Z_w = Z$, a saturated rock slope is formed. The water flow along the joint plane can be considered into two modes: (1) water seeps along the joint plane and escapes at atmospheric pressure; (2) water is blocked at the outflow suture at the slope toe due to icing, accumulating soils and growing grass, etc. For mode 1 and mode 2, the pore water pressures along the joint plane are respectively considered to distribute linearly with a zero-value (Fig. 1(a)) or $\gamma_w (H - Z + Z_w)$ (Fig. 1(b)) at the toe of the slope, yielding a resultant force U (Fig. 1). It is assumed that the seepage force and hydrodynamic force of the pore water is negligibly small, thus not included in the analysis.`

The seismic inertia forces on the potential sliding block can be considered to be horizontal (towards the slope facing) and vertical (upwards or downwards), i.e., $k_h W$ [←+] and $k_v W$ [↓+/↑–], where k_h and k_v are the horizontal and vertical seismic coefficients respectively. Thus, the resultant of gravity and vertically seismic forces can be simplified as $(1 \pm k_v)\, W$ (Fig. 1). In particular, it is noticed that the forces $(1 \pm k_v)\, W$, $k_h W$, V and U mentioned above don't act through the centroid of the potential sliding masses, thus a whole overturning failure around the slope toe will possibly occur under high water pressure or/and seismic inertia force. Here, the corresponding arms under two water pressure modes are respectively shown in Figs. 1(a)-(b).

Referring to Fig. 1, the weight W of potential sliding rock block, the water pressure V in the tension crack and U along the joint plane (two water pressure modes) are respectively obtained as follows

$$W = \frac{1}{2}\gamma H^2[(1 - Z^{*2})\cot\psi_p - \cot\psi_f] \tag{1}$$

$$V = \frac{1}{2}\gamma_w {Z_w}^2 \tag{2}$$

$$U = \frac{1}{2}\gamma_{\mathrm{w}} Z_{\mathrm{w}} H (1 - Z^{*}) \csc\psi_{\mathrm{p}} \qquad \text{(Water pressure mode 1)} \tag{3a}$$

$$U = \frac{1}{2}\gamma_{\mathrm{w}} H^{2} (1 - Z^{*} + 2Z_{\mathrm{w}}^{*}) \cdot (1 - Z^{*}) \csc\psi_{\mathrm{p}} \qquad \text{(Water pressure mode 2)} \tag{3b}$$

In Eqs. (1)-(3), $Z^{*} = Z/H$, $Z_{\mathrm{w}}^{*} = Z_{\mathrm{w}}/H$, the other parameters are defined earlier. Based on the moment equilibrium principle, the corresponding arms for these forces such as W or $(1 \pm k_{\mathrm{v}})$ W, $k_{\mathrm{h}}W$, V and U are respectively achieved below

$$A_{W} = \frac{(1 + 2Z^{*})(1 - Z^{*})^{2} \cot^{2}\psi_{\mathrm{p}} - \cot^{2}\psi_{\mathrm{f}}}{(1 - Z^{*2}) \cot\psi_{\mathrm{p}} - \cot\psi_{\mathrm{f}}} \cdot \frac{H}{3} \tag{4}$$

$$A_{k_{\mathrm{h}}W} = \frac{(1 - Z^{*})[3 - (1 - Z^{*})^{2}] \cot\psi_{\mathrm{p}} - 2\cot\psi_{\mathrm{f}}}{(1 - Z^{*2}) \cot\psi_{\mathrm{p}} - \cot\psi_{\mathrm{f}}} \cdot \frac{H}{3} \tag{5}$$

$$A_{V} = H - Z + \frac{1}{3} Z_{\mathrm{w}} \tag{6}$$

$$A_{U} = \frac{2}{3} \cdot \frac{H - Z}{\sin\psi_{\mathrm{p}}} \qquad \text{(Water pressure mode 1)} \tag{7a}$$

$$A_{U} = \frac{1}{3} \cdot \frac{H - Z + 3Z_{\mathrm{w}}}{H - Z + 2Z_{\mathrm{w}}} \cdot \frac{H - Z}{\sin\psi_{\mathrm{p}}} \qquad \text{(Water pressure mode 2)} \tag{7b}$$

According to the definition of safety factor against overturning in geotechnical engineering, i.e., earth retaining structures (Das 2008), that is the ratio of resistant moments M_{R} to driving moments M_{D}. Combined with the obtained Eqs. (1)-(7), the seismic stability factor of safety against overturning (FS_{o}) for intact rock slope with a tension crack under two water pressure modes can be respectively expressed as follows

$$\begin{gathered} FS_{\mathrm{o}} = \frac{M_{\mathrm{R}}}{M_{\mathrm{D}}} = \frac{WA_{W} + \frac{1 + \cos\alpha}{2} \cdot k_{\mathrm{v}} \cdot WA_{W}}{k_{\mathrm{h}} WA_{k_{\mathrm{h}}W} + \frac{1 - \cos\alpha}{2} \cdot k_{\mathrm{v}} \cdot WA_{W} + UA_{U} + VA_{V}} = \\ \frac{\left(1 + \frac{1 + \cos\alpha}{2} k_{\mathrm{v}}\right)\left[T^{2}\left(1 + 2Z^{*}\right) - \cot^{2}\psi_{\mathrm{f}}\right]}{\left[2\gamma_{\mathrm{w}}^{*} Z_{\mathrm{w}}^{*} P^{2} + 3\gamma_{\mathrm{w}}^{*} Z_{\mathrm{w}}^{*2}\left(1 - Z^{*} + \frac{1}{3} Z_{\mathrm{w}}^{*}\right)\right] + \frac{1 - \cos\alpha}{2} k_{\mathrm{v}}\left[T^{2}\left(1 + 2Z^{*}\right) - \cot^{2}\psi_{\mathrm{f}}\right] + k_{\mathrm{h}}\left\{T\left[3 - \left(1 - Z^{*}\right)^{2}\right] - 2\cot\psi_{\mathrm{f}}\right\}} \end{gathered} \tag{8}$$

In which $\gamma_{\mathrm{w}}^{*} = \gamma_{\mathrm{w}}/\gamma$, α is an assumed directional angle, and it can be specified as $\alpha = 0°$ when a vertically downward seismic inertia force is applied on the potential instability mass, inducing a resistant moment against overturning; or $\alpha = 180°$ when a vertically upward seismic inertia force is applied on the rock block, a sliding moment is yielded. The parameters P and T can be respectively written as

$$P = \left(1 - Z^*\right)\csc\psi_{\mathrm{p}} \tag{9a}$$

$$T = \left(1 - Z^*\right)\cot\psi_{\mathrm{p}} \tag{9b}$$

Eq. (8) is the general analytical expression of seismic factor of safety against overturning for the rock slope under the first water pressure mode. Under the second water pressure mode a similar expression can be given as

$$FS_{\mathrm{o}} = \frac{\left(1 + \frac{1+\cos\alpha}{2}k_{\mathrm{v}}\right)\left[T^2\left(1+2Z^*\right) - \cot^2\psi_{\mathrm{f}}\right]}{\left[\gamma_{\mathrm{w}}^*\left(1 - Z^* + 3Z_{\mathrm{w}}^*\right)P^2 + 3\gamma_{\mathrm{w}}^* Z^{*2}\left(1 - \frac{2}{3}Z^*\right)\right] + \frac{1-\cos\alpha}{2}k_{\mathrm{v}}\left[T^2\left(1+2Z^*\right) - \cot^2\psi_{\mathrm{f}}\right] + k_{\mathrm{h}}\left\{T\left[3 - \left(1 - Z^*\right)^2\right] - 2\cot\psi_{\mathrm{f}}\right\}} \tag{10}$$

in which these parameters are defined earlier.

In Eqs. (8) and (10), let $Z_{\mathrm{w}} = Z$, the seismic stability factor of safety against overturning for a saturated rock slope under two water pressure modes can be readily obtained. Further, in the absence of seismic force, i.e., $k_{\mathrm{h}} = k_{\mathrm{v}} = 0$, the static stability factor against overturning for a saturated rock slope under two water pressure modes also can be achieved. It should be mentioned that when the static or pseudo-static seismic stability factor of safety against overturning for the rock slope doesn't satisfy the required design value (typical $FS_{\mathrm{o}} = 1.5$), a set of anchors or a row of piles can be installed to increase the resisting moments. Although the anchorage moment is not discussed herein, it is straightforward to incorporate into the resisting moment equilibrium equations as presented above.

3. Verification

In order to validate the analytical formula, a saturated rock slope with water-filled tension crack subjected to horizontal and vertical seismic loading is considered as a special case, in which a particular set of governing parameters in dimensionless forms are adopted as: $\psi_{\mathrm{f}} = 75°$, $\psi_{\mathrm{p}} = 50°$, $Z^* = 0.25$ or 0.3, $k_{\mathrm{h}} = 0.2$ or 0.3 and $k_{\mathrm{v}} = -0.5\ k_{\mathrm{h}}$. Typically, $H = 10$ m, $Z_{\mathrm{w}} = Z$, $\gamma = 25$ kN/m^3 and $\gamma_{\mathrm{w}} = 10$ kN/m^3. In addition, the water seeps along the joint plane and escapes at atmospheric pressure (mode 1), the strength of shear and tension at the joint plane (potential failure surface) can be assumed as $\varphi = 30°$, $c = 75$ kPa and $\sigma_{\mathrm{t}} = 0.5$ MPa, respectively.

Using Eq. (8), and let $FS_{\mathrm{o}} = 1.0$, the horizontally (or vertically) critical seismic coefficients against overturning for the rock slope under given water pressure modes and vertical seismic coefficients (or horizontal seismic coefficient) can be conveniently obtained, and the results are listed in Table 1. It can be seen by comparison of case (1) and (2) in Table 1 that the horizontally critical seismic coefficients decrease remarkably when water pressure in the tension crack and along the joint plane is included in the analysis. Especially, the absolute value of vertically critical seismic coefficient decreases rapidly with increasing the horizontal seismic coefficient from $k_{\mathrm{h}} = 0.2$ to $k_{\mathrm{h}} = 0.3$, even a critically downward seismic inertia force appears.

Table 1 Comparison of critical seismic coefficients from numerical and analytical method (FS_o = 1)

Cases: water pressure mode 1 and seismic loadings	Horizontal and vertical critical seismic coefficient (k_{hcr} and k_{vcr})			
	k_{hcr} from analytical	k_{hcr} from DDA	k_{vcr} from analytical	k_{vcr} from DDA
(1) $U = V = 0$, k_v = -0.10	0.4710	0.4772	-	-
(2) $Z_w = Z$, k_v = -0.10	0.2160	0.2214	-	-
(3) $Z_w = Z$, k_v = -0.15	0.1900	0.1953	-	-
(4) $Z_w = Z$, k_h = 0.20	-	-	-0.1310	-0.1359
(5) $Z_w = Z$, k_h = 0.30	-	-	0.0600	0.0554

To verify efficiently the analytical solutions in Table 1, a numerical program based on discontinuous deformation analysis (DDA) is employed herein. The DDA model is illustrated in Fig. 2, in which the slope is modeled as stiff material with large Young's modulus and the tension crack is considered as zero strength joint. The mechanical properties of rock and joints are adopted as defined earlier (also see Table 2), the settings on penalty and time step used in DDA simulation can be treated as 10^{11} N/m and 0.01s, respectively. The critical seismic coefficients (horizontal k_{hcr} and vertical k_{vcr}) are calculated by static stability analysis using DDA and are listed in Table 1. The results are independent of joint strength of the potential failure joint plane. Furthermore, it can be clearly shown from Table 1 that the results using DDA are slightly close to those obtained from the present analytical approach.

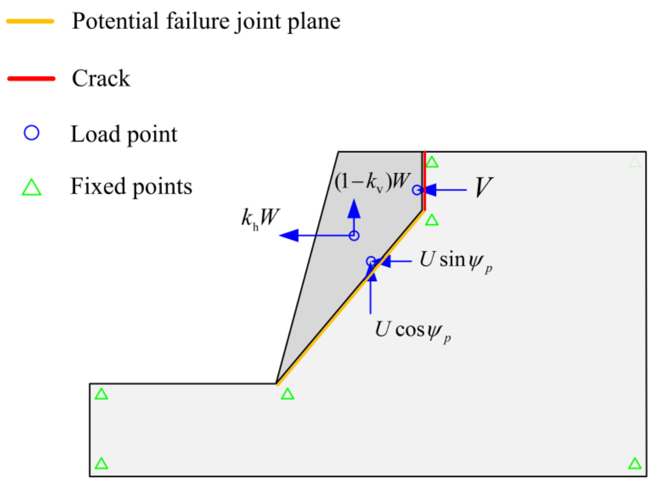


Fig. 2 DDA validation model

Table 2 Mechanical properties of rock and joints used in DDA

Rock			Tension crack	Joint plane
(1) Unit weight (kN/m^3)	25	(4) Tension (MPa)	0.0	0.5
(2) Young's modulus (MPa)	1000	(5) Cohesion (kPa)	0.0	75.0
(3) Poisson's ratio	0.2	(6) Friction angle (deg)	0.0	30

Table 3 Seismic factors of safety against overturning (FS_o) and against sliding (FS_s) by analytical approach

Case slopes (different tension crack depth, seismic inertia forces and water pressure mode 1)	Seismic factor of safety for slopes	
	Against overturning (FS_o) Eq. (10)	Against sliding (FS_s) (Ling and Cheng 1997)
(1) $Z^* = 0.25$, $U = V = 0$, $k_h = 0.3$ and $k_v = -0.15$	1.38	1.56
(2) $Z^* = 0.25$, $Z_w = Z$, $k_h = 0.3$ and $k_v = -0.15$	0.83	1.35
(3) $Z^* = 0.25$, $Z_w = Z$, $k_h = 0.2$ and $k_v = -0.1$	1.03	1.50
(4) $Z^* = 0.3$, $Z_w = Z$, $k_h = 0.2$ and $k_v = -0.1$	0.92	1.38

Further, a comparative analysis is carried out to investigate the importance of stability assessment against overturning for such a rock slope. Eq. (8) and the earlier formulation proposed by Ling and Cheng (1997) are respectively used to calculate the seismic factor of safety for the slope against overturning (FS_o) and against sliding (FS_s) , the results are listed in Table 3.

We suppose that a rock slope with water-filled tension crack is stable when the stability factors of safety against both overturning and sliding are not less than 1.5, that is, $FS_o \geq 1.5$ and $FS_s \geq 1.5$. It can be seen from Table 3 that the rock slopes in case (1) and (3) satisfy the requirement of stability against sliding, but not meet the stability against overturning. In particular, if the design factor of safety for the rock slopes under seismic loading is adjusted to 1.35, all the examined cases are basically stable against sliding; however, the stability against overturning for almost all the cases do no satisfy the design value. It evidently indicates that the assessment of stability against overturning on the basis of the stability against sliding is extraordinarily essential.

4. Parametrical study and design charts

The analytical Eqs. (8) and (10) are used to investigate the effect of any individual parameter on the seismic stability factor of safety of rock slope against overturning, and the typical design charts are also presented based on the specified case slope. Here, the fundamental geometric factor

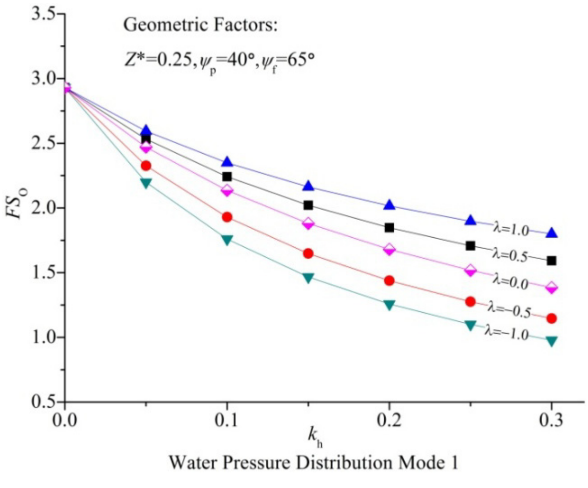


Fig. 3 Relationship between FS_o and k_h under seismic coefficient ratio λ

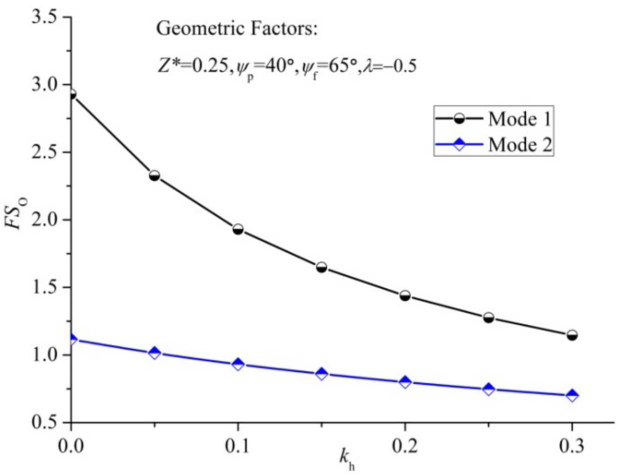


Fig. 4 Relationship between FS_o and k_h under two water pressure modes

of slope are gives as $Z^* = 0.25$, $\psi_f = 65°$ and $\psi_p = 40°$. For higher rock slopes, amplification of acceleration may be anticipated. Thus, seismic response analysis may need to be conducted before an appropriate seismic coefficient will be used. Here, it should be noted that the current pseudo-static seismic stability analysis for saturated rock slopes against overturning does not take amplification into consideration.

Fig. 3 shows that the relationship between the factors of safety against overturning (FS_o) for the rock slope and the horizontal seismic coefficients under different coefficient ratios λ (i.e., $\lambda = k_v/k_h$). It can be seen from Fig. 3 that FS_o deceases rapidly with increasing the horizontal seismic coefficient k_h. For a given k_h, FS_o deceases with the decrease of seismic coefficient ratios λ (or vertical seismic coefficient k_v) from a positive to a negative value. That is, an upward seismic inertia force remarkably lowers the seismic stability of slope against overturning.

Fig. 4 shows that for given k_h and k_v values, the FS_o obtained under water pressure mode 2 is far less than that obtained under water pressure mode 1. That is, mode 2 is a dangerous case in

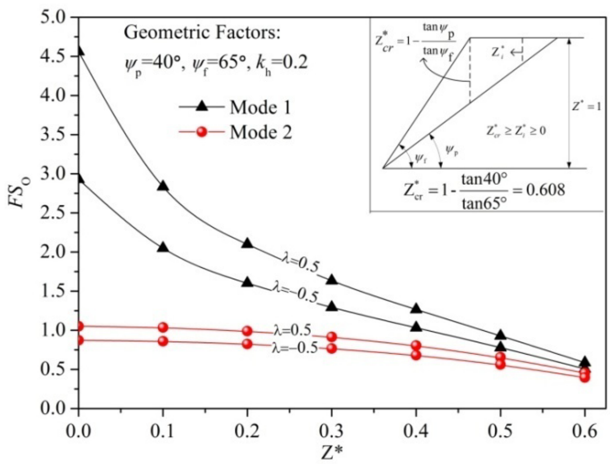


Fig. 5 Variation of FS_o with Z^* under various water pressures and seismic coefficient ratios

slope engineering; however, it should be noted that this water pressure distribution is somewhat rare in practice.

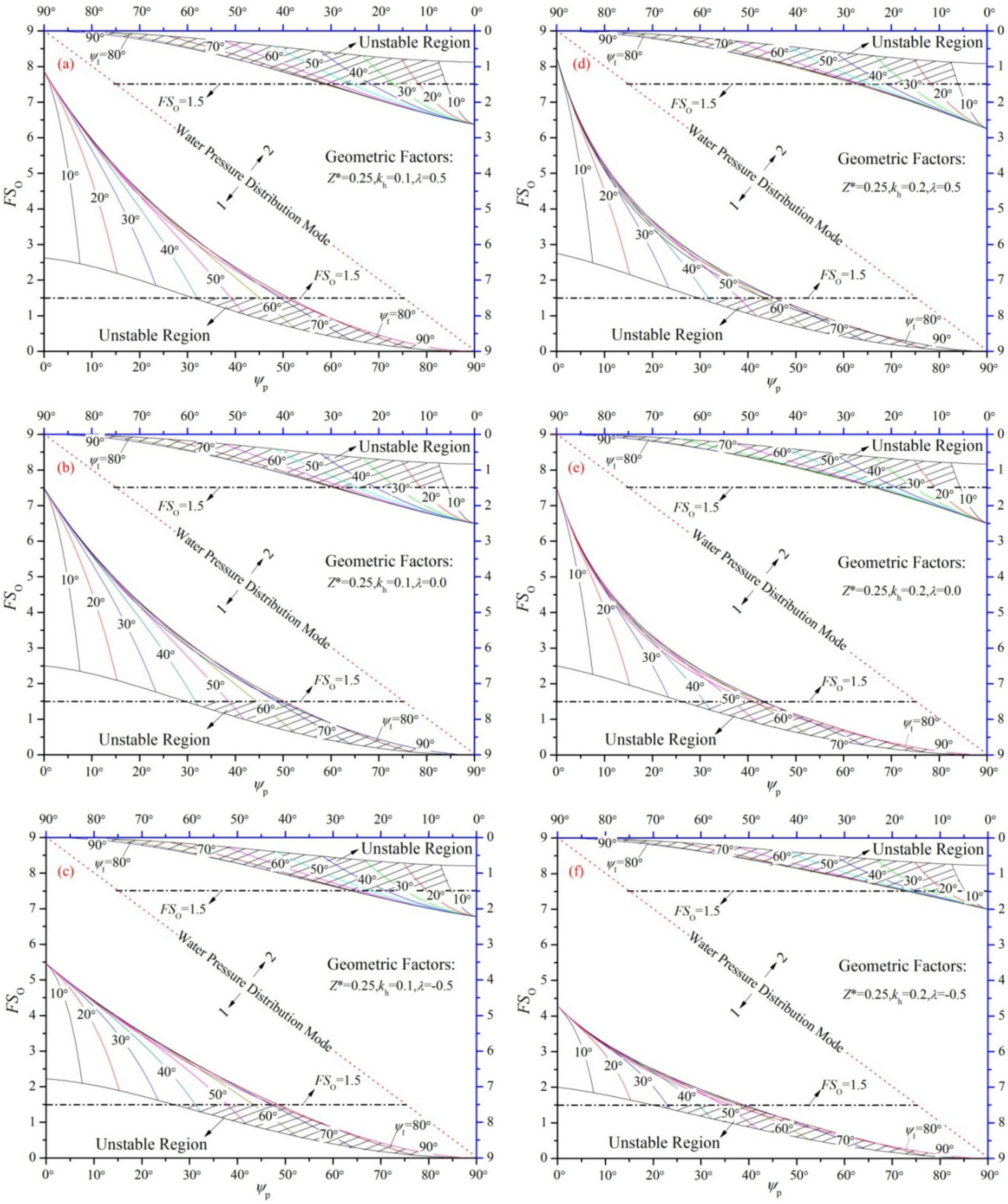


Fig. 6 Representative design chart for $Z^{*}=0.25$, $k_h=0.1$ or 0.2 and $\lambda=0$ or ± 0.5

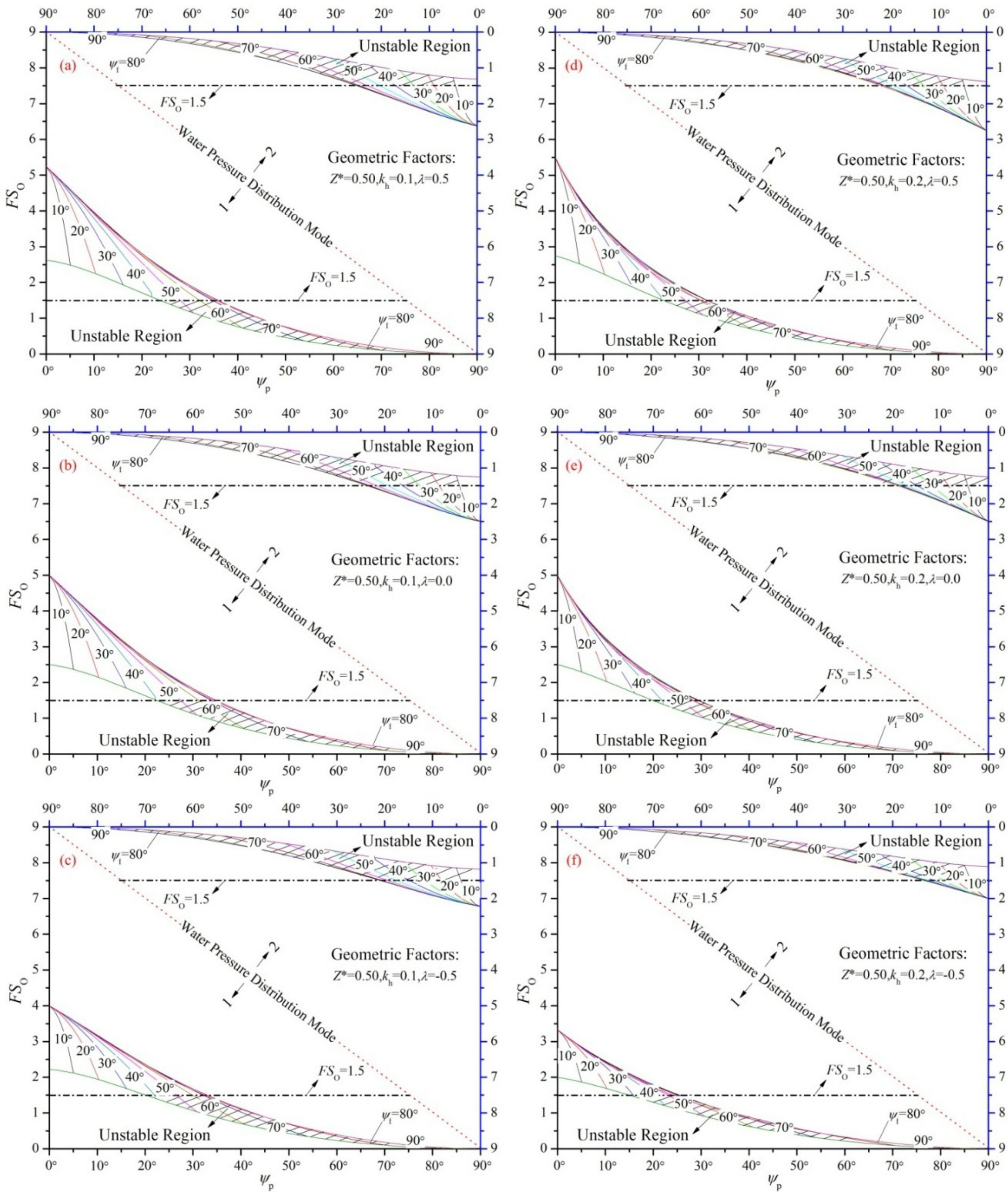


Fig. 7 Representative design chart for $Z^* = 0.50$, $k_h = 0.1$ or 0.2 and $\lambda = 0$ or ± 0.5

Fig. 5 shows that for a given horizontal seismic coefficient $k_h = 0.2$, the FS_o decreases with the increase of dimensionless tension crack depth Z^* under two water pressure modes, regardless of the directions of vertical seismic inertia forces. However, the variation of FS_o with Z^* is in general very small under water pressure mode 2 unless Z^* is greater than 0.3. It is worth mentioning that FS_o values obtained under mode 2 are rather close to 1.0 with a lower Z^* (e.g., $Z^* \leq 0.3$). In addition, it is also seen from Fig. 5 that for a given water pressure mode an upward seismic inertia force ($\lambda =$ -0.5) leads to a low FS_o by comparison of downward seismic force ($\lambda = 0.5$) no matter what the Z^* value is. When Z^* arrives at the critical value (i.e., 0.608), that is, a tension crack moves to the top (crest) of slope, at which the influence of vertical seismic inertia force on FS_o will be negligible.

To conveniently evaluate the seismic stability against overturning for a rock slope with water-filled tension crack, a series of design charts are made, in which various slope angle (e.g., 0~90° interval of 10°) and joint plane inclination (e.g, 0~90° interval of 2.5°), different tension crack depth Z^* (e.g., $Z^* = 0.25$, 0.5 and 0.75), horizontal seismic coefficients k_h (e.g., $k_h = 0.1$, 0.2 and 0.3), seismic coefficient ratio λ (e.g., $\lambda = -1.0$, -0.5, 0, 0.5 and 1.0) and water pressure modes are taken into account. Two representative design charts with the parametric ranges of practical engineering interest are shown in Fig. 6 and 7. For a given depth of water-filled tension crack, a design chart consists of six diagrams in which each represents a combination of horizontal and vertical seismic forces. From the left to the right diagrams in the same row, the horizontal seismic coefficient increases and the vertical one remains unchanged. From the upper to the bottom diagrams in the same column, the vertical seismic coefficient decreases and the horizontal one remains unchanged. Besides, one diagram can be divided into two triangle regions in which each mainly covers nine analytical curves of seismic safety factor against overturning and one critical curve which gives a low bound. The only difference of two triangle regions is that the bottom left one is for water pressure mode 1, whereas the upper right one is for water pressure mode 2. To explain how to use conveniently the design charts, a saturated rock slope with water-filled tension crack (water pressure mode 1) can be illustrated as below: geometric factor $Z^* = 0.25$, $\psi_f = 70°$ and $\psi_p = 40°$; the horizontal seismic coefficient $k_h = 0.2$ and the seismic coefficient ratio $\lambda = -0.5$, respectively. Using the bottom right diagram of Fig. 6, the factor of safety against overturning for this slope can be easily found as $FS_o = 1.4$. If the design value of FS_o is 1.5, the slope will be unstable.

Further, when the design factor of safety against overturning for the slope case is specified, i.e., $FS_o = 1.5$, an unstable region surrounded of design safety line at $FS_o = 1.5$, the curve of $\psi_f = 90°$ and the critical curve of ψ_p can be outlined and presented in the form of a grid in Figs. 6 and 7. This contributes to assessing rapidly the natural slope stability in the field or/and guiding the site excavation (critical slope angle provided) in building foundation pit and open pit of mining.

5. Conclusions

An analytical approach was developed in this study to determine the seismic stability factor of safety against overturning for a saturated rock slope with water-filled tension crack subjected to seismic loading. The analytical solutions considered two water pressure modes along the failure joint plane. Based on the parametric study and achieved design charts, some important conclusions are made as follows:

- The stability factor of safety against overturning for a saturated rock slope under the vertically upward seismic inertial force was far lower than that under the vertically downward seismic inertial force. An upward seismic inertia force should be considered for the stability analysis of rock slope against overturning in seismically active area.
- The second water pressure mode along the failure joint plane (blocked outflow suture) remarkably lowers the factor of safety against overturning for a saturated rock slope with

water-filled tension crack. This should be paid more attention under special water circumstance.

- A series of design charts for rock slope stability against overturning were presented, and an unstable region was also outlined in the form of a grid. This contributes to assessing rapidly the seismic stability of natural slopes against overturning in the field, and even guiding the reasonable site excavation (critical cut angle) in construction engineering.
- The seismic stability analysis against overturning for a rock slope should be supplemented to the conventional static or/and pseudo-static sliding stability analysis against plane failure presented by Hoek and Bray (1977) and Ling and Cheng (1997) to comprehensively evaluate the overall stability for a rock slope with water-filled tension crack subjected to seismic loading.

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