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Local soil failure before general slope failure

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Abstract Slopes are generally characterized by non-uniform stress field. Additional stresses induced by changes in boundary conditions can emphasize local differences in the stress level. As a consequence, every failure process is, at first, local, with formation of plastic zones, then general. This is the subject of this paper, which concerns clay slopes, whose failure generally implies formation of a shear zone. A special consideration is addressed to the simple and special case of infinite slope which is also featured by transition from local to general failure, even if stress conditions are uniform along the potential failure plane.

Keywords Infinite slope · Slope failure · Progressive failure · Shear zone

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List of symbols

γ	unit weight of soil
γ_{sat}	unit weight of saturated soil
γ_w	unit weight of water
ϕ_u	angle of friction in terms of total stress
ϕ'	effective angle of internal friction
c'	effective cohesion intercept
ϕ_m	mobilized angle of friction
x, y	co-ordinates parallel and normal to the slope
σ'_{xx}	effective stress parallel to the slope
σ'_{yy}	effective stress normal to the slope
σ_{yy}	total stress normal to the slope
C	ratio between effective stresses parallel and normal to the slope
τ (τ_{xy})	shear stress
τ_i	initial shear stress
τ_f	available shear strength peak available shear strength
τ_r	residual shear strength
s'	mean effective stress
u	pore pressure
E_i	resultant of stress parallel to the ground surface, above the slip surface
OCR	over-consolidation ratio
I_{BG}	generalized brittleness index [$I_{BG}=(\tau_f-\tau)/\tau_f$]
K_0	coefficient of earth pressure at rest, ratio between horizontal and vertical effective stresses

K_p	passive earth pressure coefficient
q_c	cone penetration resistance
δ	angle between the slope's direction and the failure planes
FS	safety factor against general failure
FS_{1f}	value of safety factor when local failure occurs for the first time
D	coefficient defined by Eq. (6)
D_0	D value at the beginning of the deformation process
D_{crit}	D value at the general failure
R	radius of Mohr's circle, defined by Eq. (7)
θ (θ_1, θ_2)	angle between the slope's inclination and the minor principal stress
γ_p	plastic shear strain
α	orientation of the slope against the horizontal
Y	depth from the ground surface
H	landslide thickness
H_w	depth of the water table from the ground surface

Introduction

Since the early development of modern Soil Mechanics, slope stability has been analyzed using limit equilibrium methods. The simplifying assumptions of these methods do not allow to take into account the role of pre-failure deformation on soil behaviour. However, since the Sixties several authors have suggested that slope failure may imply the development of large localized plastic shear strains well before the onset of slope rupture, eventually leading to what has been called progressive failure (Skempton 1964; Bishop, 1967; Bjerrum 1967). More recently, some well documented case histories (Burland et al. 1977; Skempton 1985), full scale experiments (Cooper et al. 1998), numerical analyses (e.g., Potts et al. 1990, 1997) and analytical models (Gilbert et al. 1996) have confirmed that slope failure is often progressive.

Local soil failure occurs when the shear strength is mobilized in a limited zone of the subsoil. In fissured or jointed clay (D'Elia et al. 1998), this mechanism may involve a pre-existing discontinuity, such as a fissure or a joint. In intact highly OC clay, local failure may lead to soil destructuration (Leroueil and Vaughan 1990), consisting in the breaking of bonds between particles or aggregates, and causes mechanisms of strain-softening and formation of shear discontinuities. These phenomena imply some deformation of the soil mass, including slippage along existing or impending discontinuities.

General slope failure occurs when kinematic conditions are such that the soil mass can experience large displacements. When the shear strength is fully mobilized everywhere along a slip surface outcropping the ground surface, slope failure develops as a slide. If a continuous weak surface or a discontinuity, such as a bedding surface, a joint or a shear pre-exists in the subsoil, general failure causes a complete mobilization of the discontinuity. In such a case, slope failure essentially depends on the state of stress and available strength along the weak surface. The mechanics of failure in intact soils is more complex, because the orientation of the developing slip surface must be consistent everywhere with that of the principal stresses. Normally, this is not assured by initial stresses whose direction could lead to local failure planes incompatible with formation of a continuous slip surface.

It is the aim of this paper to discuss mechanisms of slope failure in clay, from individual soil elements (local failure) to slope collapse (general failure). A particular consideration is paid to the case of infinite slope which also can experience transition from local to general failure, but following a special mechanical process. In particular, has been examined the evolution of the state of stress and strain prior to slope failure, showing that local failure can involve a significant lower part of the slope rather than its boundary only, as supposed by Gilbert et al. (1996).

Progressive failure along a weak surface

The mechanics of slope failure is strongly dependent on the deformation patterns in the pre-failure stage since they affect the mobilised shear strength.

The initial stress field in natural slopes is typically non-uniform because it depends on the complex geological processes that led to slope formation (Picarelli 2000). Changes in boundary conditions (surcharges, excavations, groundwater recharge, etc.) can further increase the differences in the stress levels. Induced soil deformation depends on the changes in the stress field and are consequently non-uniform too.

When a thin weak layer or a discontinuity is present in the slope, propagation of failure along it can be described following Bjerrum (1967), who deals with the case of a vertical cut excavated in an infinite intact slope. Here, it is assumed that a discontinuity (a bedding surface, a joint or a shear surface) parallel to the ground surface is located just at the toe of the cut (Fig. 1). The excavation can mobilize the shear strength on this weak plane, starting from the cut face (local failure). This occurs when the shear stress induced by excavation increases from its initial value, τ_i, to the available shear strength τ_f. The induced shear stress, τ, depends on the state of stress existing above the weak plane. The resultant, E_i, of the stresses parallel to this plane is a function of the stress history, i.e., of the OCR, as defined in the following Eq. 1. Since first failure occurs, the resulting overstress which can not be sustained

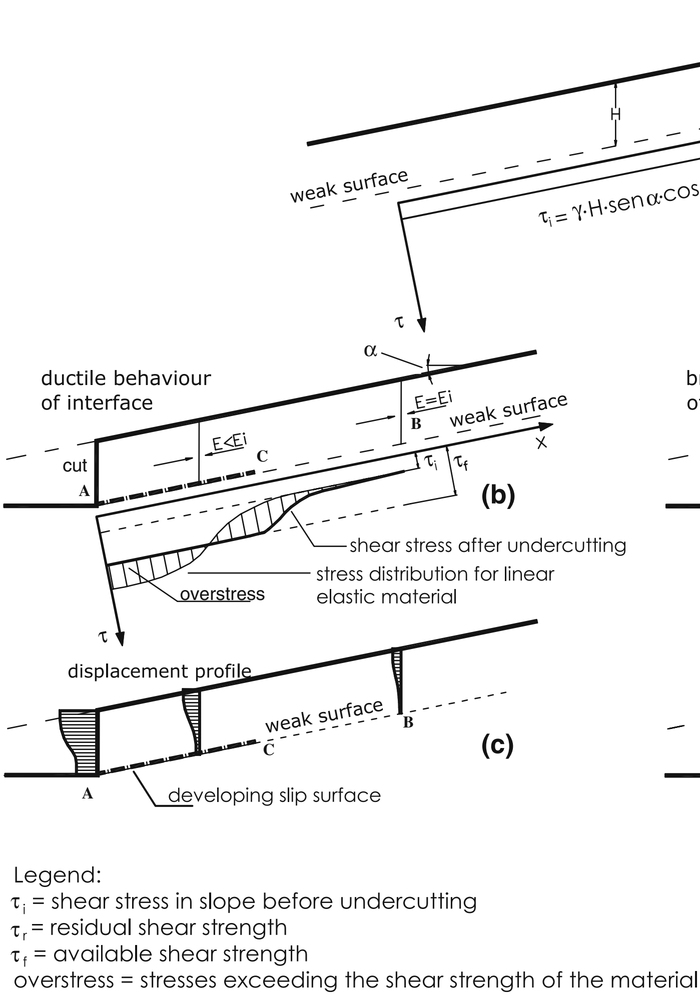


Fig. 1 Mobilized shear stress and displacement induced by a vertical cut in an infinite slope containing a discontinuity parallel to the ground surface

by clay along the discontinuity, is transferred upslope, leading to propagation of failure.

This process may be emphasised by the stress–strain behaviour of the weak plane (Fig. 1). In fact, this can have a ductile or a brittle behaviour, according to its nature (for instance, polished or rough). Along a brittle discontinuity (for example, a rough bedding surface), the length of propagation depends on the generalized Brittleness Index $I_{BG}=(\tau_f-\tau)/\tau_f$ (D'Elia et al. 1998) and on the ratio between the soil displacement at peak and the induced displacement along the discontinuity (Bjerrum 1967). The most important consequence of this process is progressive failure, which implies a decrease in shear strength along the discontinuity and further propagation of the slip surface upslope. At the end of this process, the mobilized shear strength is intermediate between the peak, τ_f, and the residual, τ_r, value.

Evidences of such phenomena in stiff clays including weak zones or surfaces are reported by Burland et al. (1977) and by Chan and Morgenstern (1987). At Saxon Pit, excavation works due to mining operations caused progressive shear displacement along a horizontal bedding plane in the Oxford Clay (Burland et al. 1977). The deformation mechanism was very similar to the one envisioned by Bjerrum, even if a global failure did not occur. A similar failure process involved a thin bentonitic layer during the excavations for the construction of the Edmonton Convention Centre. However, this did not produce any shear strength decrease due to the ductile soil behaviour (Chan and Morgenstern 1987).

Progressive deformation along a weak zone or surface may be caused also by a surcharge. However, in this case failure propagates downward, following a process similar to the one described before (Fig. 2).

As an example, Cartier and Pouget (1988) describe the deformation of a gentle slope caused by construction of an embankment at the Sallèdes site, France. Earthworks induced shear displacements along a pre-existing slip surface bounding an old landslide body.

Reactivation of old mudslide bodies in the Basento valley follow similar mechanisms (Picarelli et al. 1995)

Mechanics of failure in intact fine-grained soils from laboratory and field evidences

Experience shows that natural slopes are often constituted by high overconsolidated clay. Both structure and overconsolidation ratio may result in soil brittleness. Therefore, high stress levels are responsible for localized plastic shear strain and formation of local shear discontinuities.

The effects of localized shear strain have been described by Riedel (1929) and by Skempton (1967) who reported fundamental observations on the fabric induced in direct shear tests. Riedel noticed that rupture is preceded by formation of short and discontinuous shear fissures that are not aligned to the direction of imposed shear displacement (Stage 1, in Fig. 3). As the shear force increases, further shear fissures form with different orientations (Stage 2). According to their orientation, they are called Riedel or thrust shears: following the terminology proposed by Skempton and Petley (1967), all these fissures are referred to as minor shears, being caused by small relative displacements. Only in the final stage, a continuous shear discontinuity, the so-called principal shear (Skempton and Petley 1967), is eventually generated in the direction of the shear force (Stage 3). Therefore, a shear zone develops. This has a special fabric characterized by a number of fissures bounding intact soil elements, which sometimes are referred to as shear lenses, and a continuous slip surface. Further displacements along the slip surface lead to orientation of clay particles around the slip surface causing a decrease of the shear strength towards its residual value.

Similar results have been described by Morgenstern and Tchalenko (1967). They explain the mechanics of such a process by rotation of Mohr's failure planes during the test, and formation of fissures in the direction of failure planes until they can accommodate formation of a slip surface in the direction of shear. The same does not occur in triaxial tests, where the development of failure planes consistent with the direction of principal stresses (that remains the same during the test) is allowed by the apparatus.

Previous data suggest that the mechanism of rupture observed in direct shear tests might occur

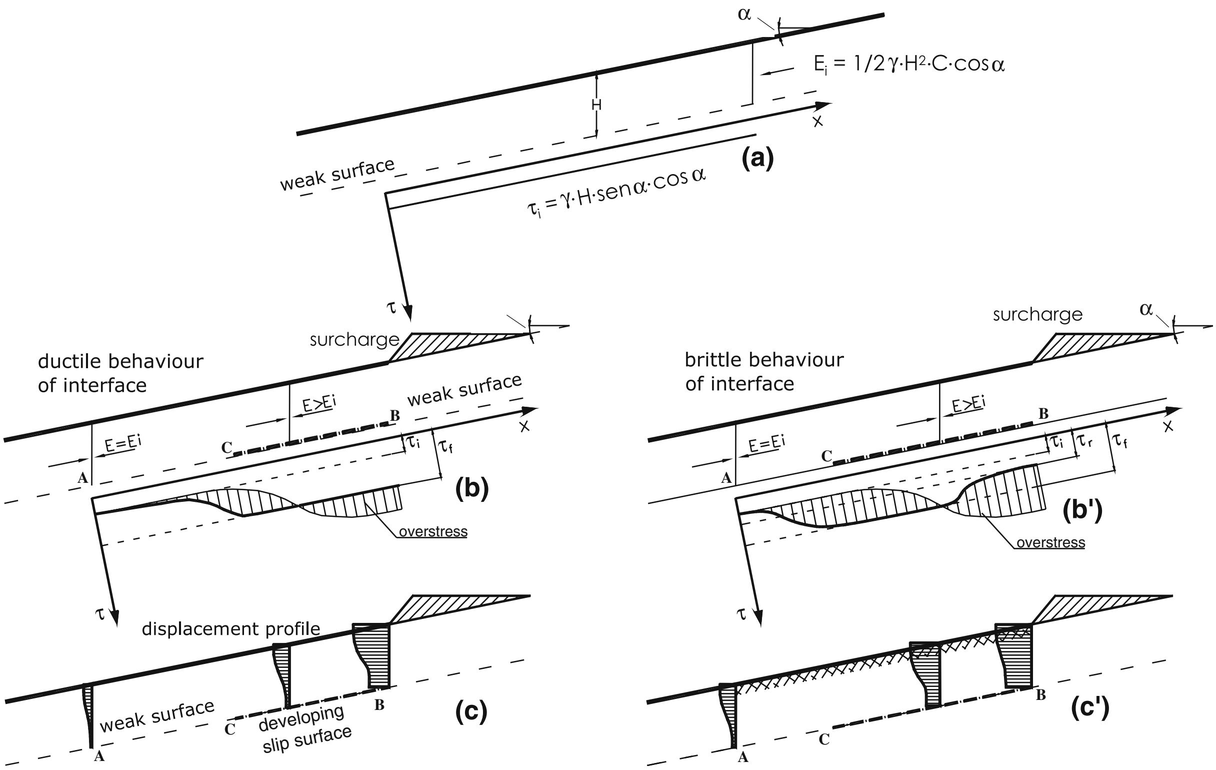


Fig. 2 Mobilized shear stress and displacement induced by a surcharge on an infinite slope containing a discontinuity parallel to the ground surface

also in the field, if failure is forced by rotation of the principal stresses. In fact, field evidences of shear-induced fabric show several similarities with the fabric of specimens subjected to direct shear tests (Picarelli et al. 1997).

Two shear zones having a different origin are shown in Fig. 4. The first one (a) is a fault investigated by Skempton (1967) at Jari, Pakistan; the second one (b) is the shear zone found at the base of the Guildford landslide, UK (Skempton and Petley 1967). Both zones have a sizable thickness and contain a set of minor shears and at least a principal shear (slip surface). The soil fabric includes shear lenses bounded by minor shears (Fig. 4). If the soil has experienced large deformations, it may be highly destructured and softened and may include lithorelicts of the parent formation (Pellegrino et al. 2004).

Further data regarding shear zones in highly overconsolidated clays or in mudstones can be found in the literature. Examples of fault zones are reported by Tchalenko (1970) and by Tani and Ueta (1998). Chandler et al. (1998) mention some shear zones induced by folding in the London clay. Matheson and Thomson (1973), Milligan (1981) and Picarelli and Urciuoli (1993) describe softened shear zones generated by valley erosion. Guerriero (1995) and Comegna (2005) report a lot of data regarding softened shear zones found at the base of mudslides in Southern Italy. Further elements concerning shear zones at the base of landslides are reported by Skempton (1985), Bertini et al. (1986), Trenter and Warren (1996) and others.

Progressive failure in intact soils

The scheme reported in Fig. 5 is referred to as the case of a cut slope. It is based on considerations by Skempton and Petley (1967) and by Morgenstern and Tchalenko (1967), suggesting a possible evolution of slope failure by localized shear strain and propagation of a slip surface in intact soils.

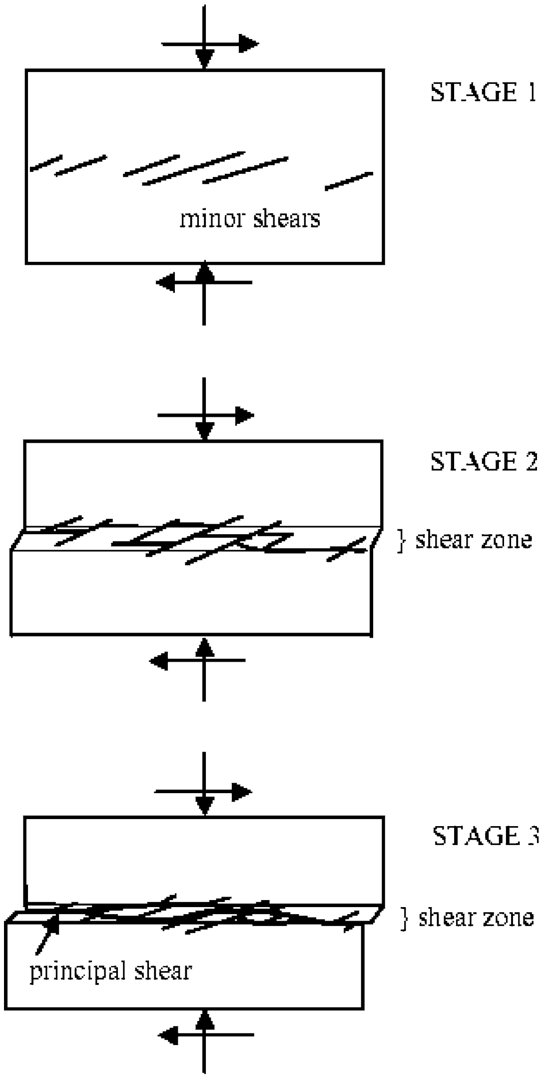


Fig. 3 Shear-induced fabric during a direct shear tests (Picarelli et al. 2000, modified after Skempton 1967)

According to the figure, local failure firstly occurs in a limited zone at the toe of the cut, where the peak strength is mobilized along directions depending on the direction of local principal stresses (Fig. 5a). This process may generate small shear fissures (minor shears) in the direction of failure planes (Fig. 5d). Such fissures are not necessarily consistent with formation of a continuous slip surface; therefore, at this stage only finite strains are allowed. A slip surface can develop only in a subsequent phase, as a consequence of further shear stress increase, causing a rotation of failure planes and formation of new

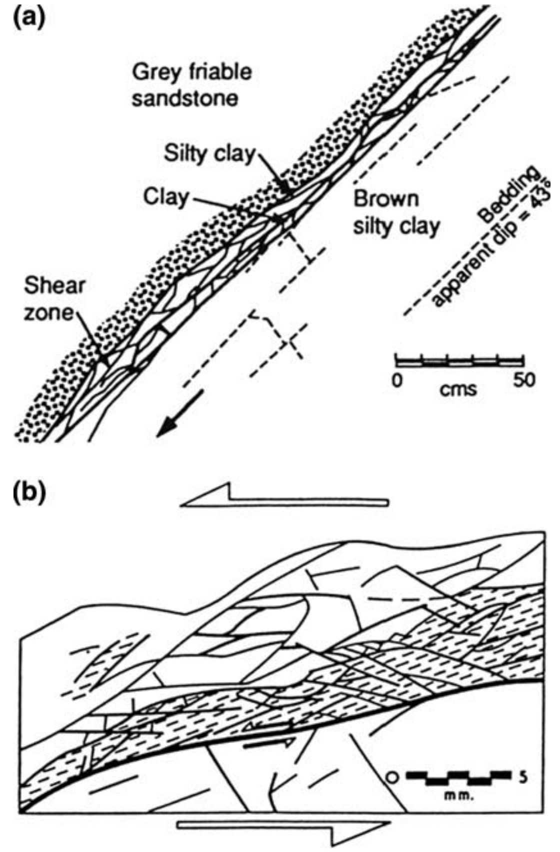


Fig. 4 Examples of shear zones: (**a**) a fault at Jari, Pakistan (from Skempton and Petley 1967); (**b**) the shear zone at the base of a landslide (from Skempton and Petley 1967)

shears, which merge into a unique principal shear (Fig. 5b and e). Further stress change can determine propagation of the shear zone and development of a slip surface. In this stage, the global safety factor of the slope remains larger than one, even if locally it is equal to one. A general slope failure will occur only when the slip surface will reach the ground surface, allowing for development of large displacements (Fig. 5c and f).

Many Authors (Skempton 1964; Bjerrum 1967) invoke progressive failure to justify the mobilization of a shear strength that is typically less than the peak. Generally progressive failure is assumed to be a drained process. Referring to soft and sensitive Scandinavian clays, Bernarder and Olofsson (1981) and Bernarder (2000) remarked that many extensive landslides occurring in such deposits have to be explained by undrained progressive failure, often developing along a surface

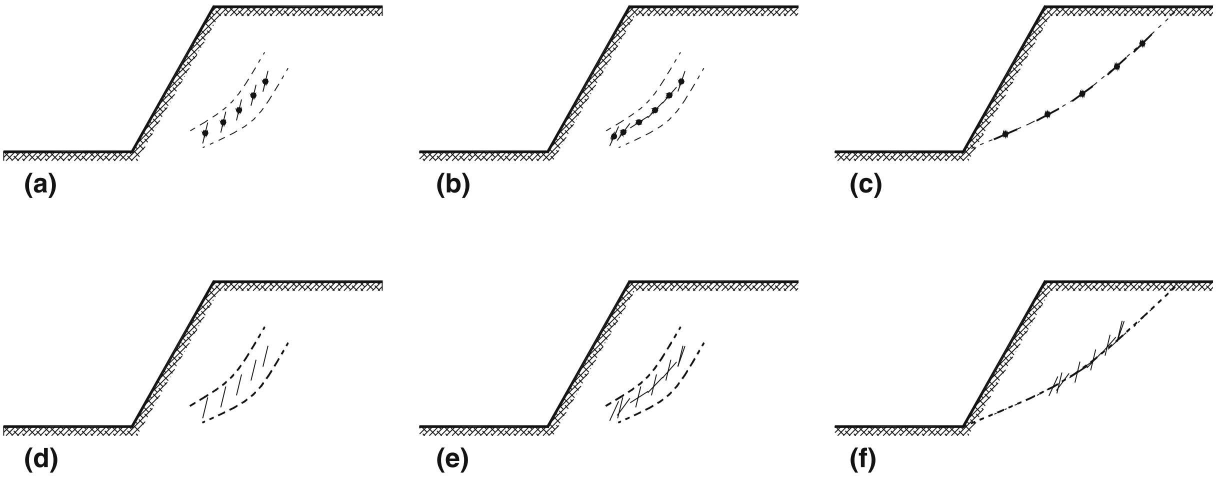


Fig. 5 A hypothetical process of slope failure induced by cutting

which is essentially parallel to the ground surface. They stress that these landslides may be triggered by different phenomena, as loading, local increase in pore pressure, lateral soil displacement due to e.g., pile driving, excavation, erosion so on. Bernarder (2000) also describes cases of "unfinished landslides" characterized by formation of long cracks on the ground surface, but no generation of general slope failure.

For the purpose of this paper, it seems appropriate to discuss at first local failure, and then development of the slip surface until general slope failure, even though these two phenomena are combined.

Development of local failure

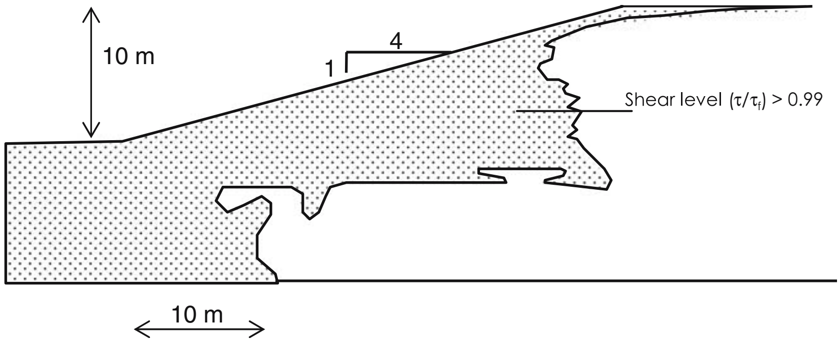
Literature reports only limited information on local slope failure (Dunlop and Duncan 1972; Lo and Lee 1973). In particular, Lo and Lee (1973) show that excavations may generate large plastic zones, and that the extension of such zones is to be related to the coefficient of earth pressure at rest K_0.

More recently, Kovacevic (1994) and Potts et al. (1997) reconsidered this topic using an elastic-plastic non-associative model in which softening behaviour is accounted for by allowing the angle of friction, ϕ', and the effective cohesion, c', to diminish with the deviator plastic strain after the soil has reached its peak strength. They analyzed the delayed failure of a 10 m high and 3:1 cut excavated in the OC London clay as pore pressure is going towards equilibrium. They show the development of general failure. In contrast, in a high 4:1 slope which remains globally stable on the long term (Fig. 6), local failure is reached in a large part of the slope, but general failure does not occur (Kovacevic, 1999, Personal communication). This is a case of generalized local failure which is not followed by slope failure; in fact, even if the local safety factor is equal or close to one, the global safety factor remains larger than one.

As previously indicated, local failure implies the development of plastic strains and shears, and possibly soil destructuration due to breaking of bonds between particles or aggregates. Weakening associated with local failure and destructuration is revealed by stiffness and strength decrease. This has been observed in several natural slopes:

– Demers et al. (1999) investigated a first-time landslide along the Maskinongé River, in Québec, in a nearly NC structured clay (OCR = 1.35). They performed a series of piezocone tests along sections perpendicular to the river and at some distance from the landslide, in areas that were stable but probably with a low safety factor. In contrast with the very homogeneous q_c profiles obtained at some distance from the slope, along the slope these profiles are irregular with rather low tip resistance, indicating local weakening of the clay. In

Fig. 6 Plastic zones induced by a gentle cut in stiff clay (From Leroueil 2001, after a personal communication from Kovacevic)



addition, weakening is not localized, but generalized to the entire clay mass close to the slope. It is worth mentioning that the failure surface observed in the nearby landslide was within the weakened zone, but not at its lower boundary (Fig. 7).

– Delisle and Leroueil (2000) and Leroueil (2001) report several other unfailed slopes in the Province of Québec where some weakening of sensitive clay was detected with the piezocone. Comparing clay samples taken in intact and weakened zones, Delisle and Leroueil (2000) conclude that in weakened zones there are more fissures and the clay is slightly less micro-structured.

Similar observations on the effects of shear on soil structure are reported by Totani et al. (1997).

All these field observations seem to confirm the results from mentioned numerical analyses.

Progressive development of a slip surface in a soil mass

Some relevant numerical analyses concerning progressive failure have been recently published by Potts et al. (1990, 1997). Potts et al. (1990) analyzed the behaviour of the Carsington embankment, UK, failed in June 1984, just prior to completion (Skempton 1985). The embankment was built above Carboniferous mudstones, which are locally covered by about 1 m of ''yellow clay'' that was left under the embankment (Fig. 8a). In the analyses, except for the mudstone foundation, strain-softening soil behaviour was assumed for all materials. The core and its upstream extension (the boot) were modelled in terms of total stress, ϕ_u=0. The behaviour of the other materials, including the yellow clay located below the shoulders of the embankment, was analyzed in terms of effective stress. The analysis shows that the major part of the core and boot reached local failure when the elevation of the embankment was at about 196 m, i.e., well before general failure, which occurred at elevation 201 m.

The FEM analysis well reproduces the progressive development of the actual failure surface and provides reasonable agreement with observations made by instrumentation (Skempton 1985). The analysis shows the development of a downward failure starting from the core and the boot and progressing in the yellow clay layer. Figure 8a and b show the shear stress within the 'yellow clay' for embankment elevations of 195.0 m (A), 200.0 m (B) and 200.75 m (C), together with the peak and residual shear strengths of the yellow clay when the embankment reaches elevation 200.75 m. Referring to the upstream side (Fig. 8b), it can be seen that: (a) when the crest is at elevation 195 m, the shear stress is far less from the peak; (b) when the crest is at elevation 200.0 m, it has already reached the peak, starting to decrease over a distance of about 15 m; (c) at an elevation of 200.75 m, it has reached the peak over a length of 40 m towards the upstream toe of the embankment. Potts et al. (1990) also showed that, at the final configuration, the safety factor calculated with limit equilibrium analysis using peak strength was about 1.2. Referring to the downstream side, Fig. 8c shows that the shear stresses in the yellow clay remains smaller than the peak strength. This analysis clearly demonstrates the role of progressive failure in the collapse of the Carsington Embankment.

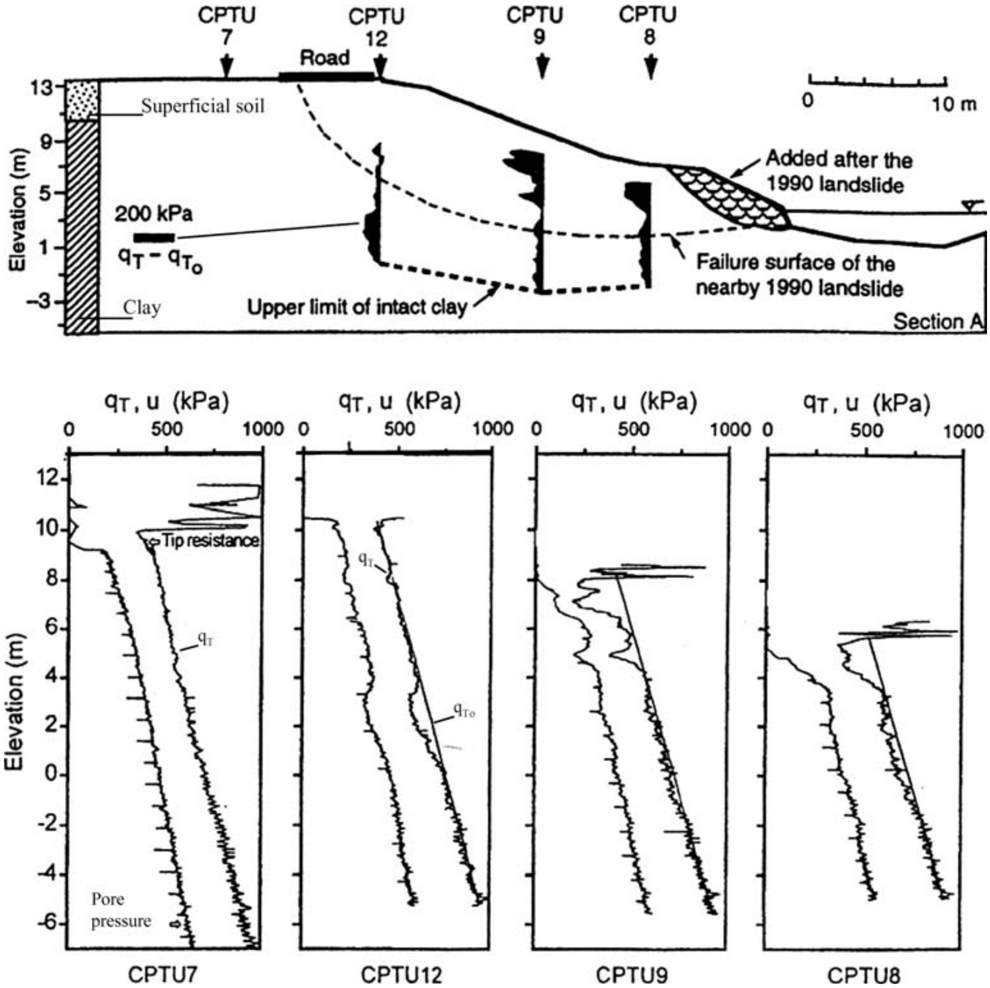


Fig. 7 Results of CPTU tests in the Maskinongé site (after Demers et al. 1999)

As shown before, further discussions concerning progressive failure induced by excavation are reported by Potts et al. (1997) who relate the development of general slope failure to pore pressure equalization.

Another evidence of progressive development of a failure surface is provided by Cooper et al. (1998) who brought to failure a well instrumented experimental cut at Selborne. UK, by increasing pore pressures through some wells built within the slope. The Selborne cut is 9 m high, 2:1 slope in overconsolidated Gault clay. Failure occurred on 16 July 1989 (day 196 in the time scale used by Cooper et al. 1998). Results as well as observations reported by Cooper and co-workers, give interesting insights in the development of progressive failure which has been summarized in Fig. 9 (Leroueil, 2001):

– All the inclinometers show progressive deformation of the soil mass with occurrence of local shearing at some time.
– Shortly after excavation, localized shearing appears at the toe of the cut, whereas the overall factor of safety is larger than 1.26. The slip surface on day 171 is reported in Figure 9a.
– At the time of reading C (days 88–96), the profile in I.06 is regular whereas localization is observed at the depth of about 2 m in I.04 and I.08. Figure 9b shows the presence of a slip surface in the higher and lower parts of the slope.

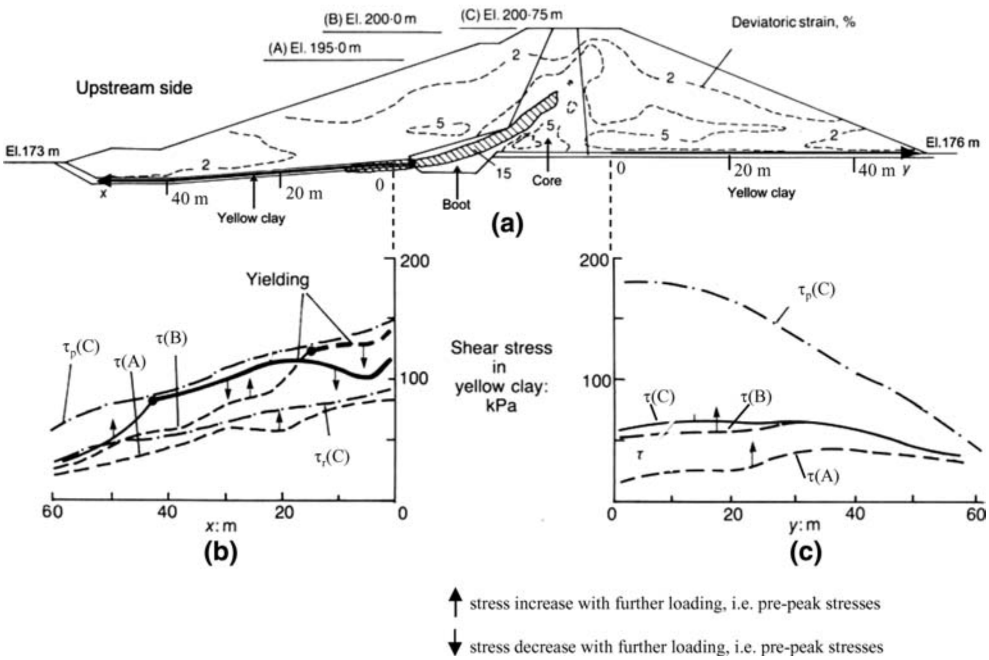


Fig. 8 Mechanics of the Carsington embankment failure as obtained by numerical analyses (after Potts et al. 1990): (**a**) deviator strains in the embankment; (**b**) shear stress along the slip surface upstream; (**c**) shear stress along the contact with foundation soils downstream

- At the time of reading D (day 175), localization is discernible in all the inclinometers, except inclinometer I.05 (Fig. 9c). According to Cooper et al. (1998), at that time the global factor of safety, calculated by a limit equilibrium method, was close to 1.04.
- The last reading (reading E on day 186) shows localization along the entire slip surface (Fig. 9d); in fact, general failure occurred 10 days later. Another slip surface is developing below the first one (Fig. 9d).

Mechanics of failure in infinite slope

In previous sections, progressive failure has been associated with the existence of a non-uniform stress field in the slope. Here it will be shown that slope failure can be preceded by local soil failure also starting from uniform stress levels along the critical surface.

From local soil failure to general slope failure

In a previous paper Urciuoli (2002) analyzed the mechanics of rupture of an infinite slope constituted by a ductile saturated cohesionless soil layer resting on a bedrock. The soil is linear elastic-perfectly plastic, with associative flow rule; its shear strength is expressed by the classic Mohr–Coulomb failure criterion. The groundwater table is parallel to the ground surface. The effective normal stress parallel to the ground surface, σ'_{xx}, is expressed as a function of the effective stress σ'_{yy} normal to the ground surface (Fig. 10), by introducing the effective stress ratio C, equivalent to the coefficient of earth pressure at the rest, K_0, thus depending on the geologic history of the slope:

$$C = \frac{\sigma'_{xx}}{\sigma'_{yy}} \qquad (1)$$

Failure is induced by groundwater rising; the stress field along planes parallel to the ground surface is uniform during the entire process. According to previous hypotheses, the direction of infinite plastic strains must coincide with the Mohr's failure plane and general failure, defined as plastic collapse, is attained at limit equilibrium conditions.

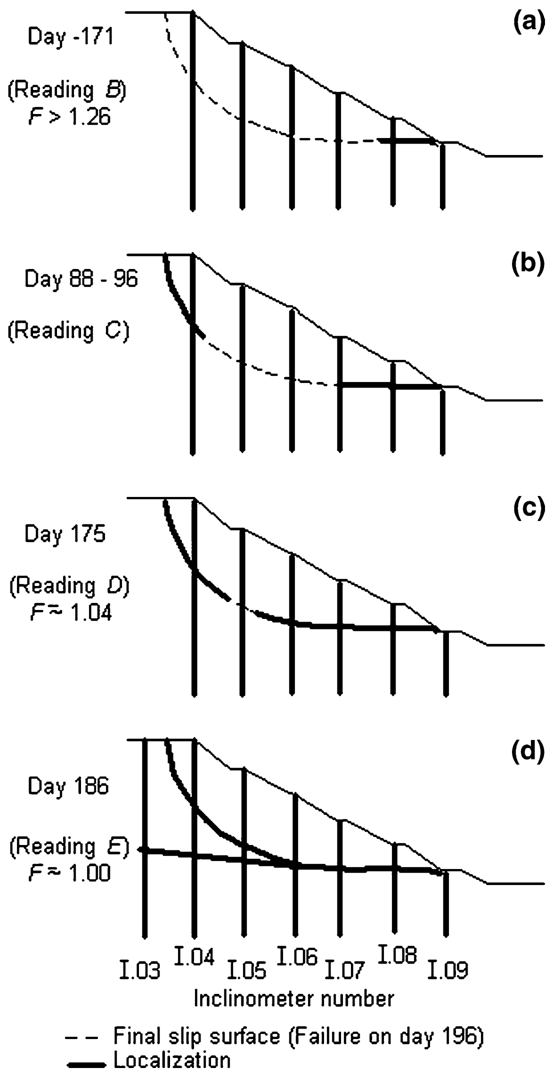


Fig. 9 Development of the slip surface in the Selborne slope (from Leroueil 2001)

Here an improvement to the model initially proposed by Urciuoli (2002), is presented. In fact, it is assumed that the soil has a cohesion, c', and both a ductile and a brittle post-peak behaviours are considered. In the latter case two stress–strain relationships are examined: in the first one, after peak, ϕ' gradually decreases as a function of accumulated plastic shear strain; in the second one, at the onset of rupture, c' instantaneously drops to zero.

Some relevant aspects of stress evolution, which are numerically described in the following section, are concisely shown in Fig. 11. During the stage of groundwater table rising, the state of

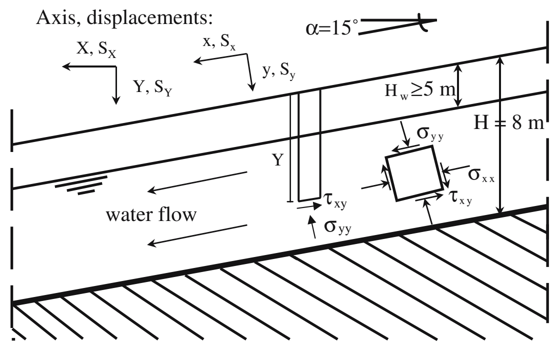


Fig. 10 Features of the infinite slope considered in the considered examples (from Picarelli et al. 2000)

stress in the soil layer can be obtained through the Mohr's circle. In the proposed example, the initial stress σ'_{xx} is smaller than σ'_{yy}. Furthermore, the safety factor FS against general slope failure is defined as the ratio between the tangent of the friction angle and the tangent of the angle of mobilized strength, ϕ_{m}, $\tan\phi_{\mathrm{m}}$ being defined in the Eq. 4 where the shear stress, τ_{xy}, and the effective normal stress, σ'_{yy}, are calculated at the base of the soil layer.

As pore pressure increases, FS decreases. For a critical pore pressure value, soil failure occurs in all points along the base of the soil layer (Fig. 11b), but failure planes, as predicted by the Mohr's theory, are not necessarily parallel to the slope. Therefore, large displacements cannot occur, since they are not kinematically admissible. Further pore pressure increases cause a decrease in σ'_{yy} and a rotation of the principal stresses at the base of the soil layer, and consequently, of failure planes (Fig. 11c). In this way, the angle, δ, between the slope's direction (i.e., the top of the bedrock) and the failure planes becomes smaller and smaller, while ϕ_{m} increases.

As pore pressure increases, local soil failure propagates vertically from the base of the soil–bedrock interface towards the ground surface, the angle between failure planes and slope direction being larger than at the base of the layer. In this way, a thicker and thicker plastic zone forms at the base of the layer. Looking at the real behaviour of natural soils, we can imagine that the described process of soil failure implies formation of minor shears consistent with the direction of the Mohr's failure planes (Skempton 1967). Once the failure planes along the soil–bedrock interface become parallel to the slope, ϕ_m becomes equal to ϕ' and general slope failure occurs (Fig. 11d).

The thickness of the plastic shear zone at slope failure depends on the initial stress field, thus on the coefficient of earth pressure at rest: the larger it is, the thinner is the plastic shear zone (Urciuoli 2002).

Analysis of the stress field

The total normal and shear stresses, σ_{yy} and τ_{xy}, along planes parallel to the ground surface can be obtained from the equilibrium in the vertical direction (Fig. 10):

$$\begin{aligned}\sigma_{yy} &= [\gamma \cdot H_w + \gamma_{sat} \cdot (Y - H_w)] \cdot \cos^2 \alpha \\ \tau_{xy} &= [\gamma \cdot H_w + \gamma_{sat} \cdot (Y - H_w)] \cdot \sin \alpha \cdot \cos \alpha\end{aligned} \tag{2}$$

where α is the angle of slope, γ and γ_{sat} are the unit weight of soil, above and below the water table.

If seepage is parallel to the ground surface, the pore pressure is

$$u = (Y - H_w) \cdot \gamma_w \cdot \cos^2 \alpha. \tag{3a}$$

Therefore the effective stress normal to the slope can be obtained from Eqs. (2) and (3a):

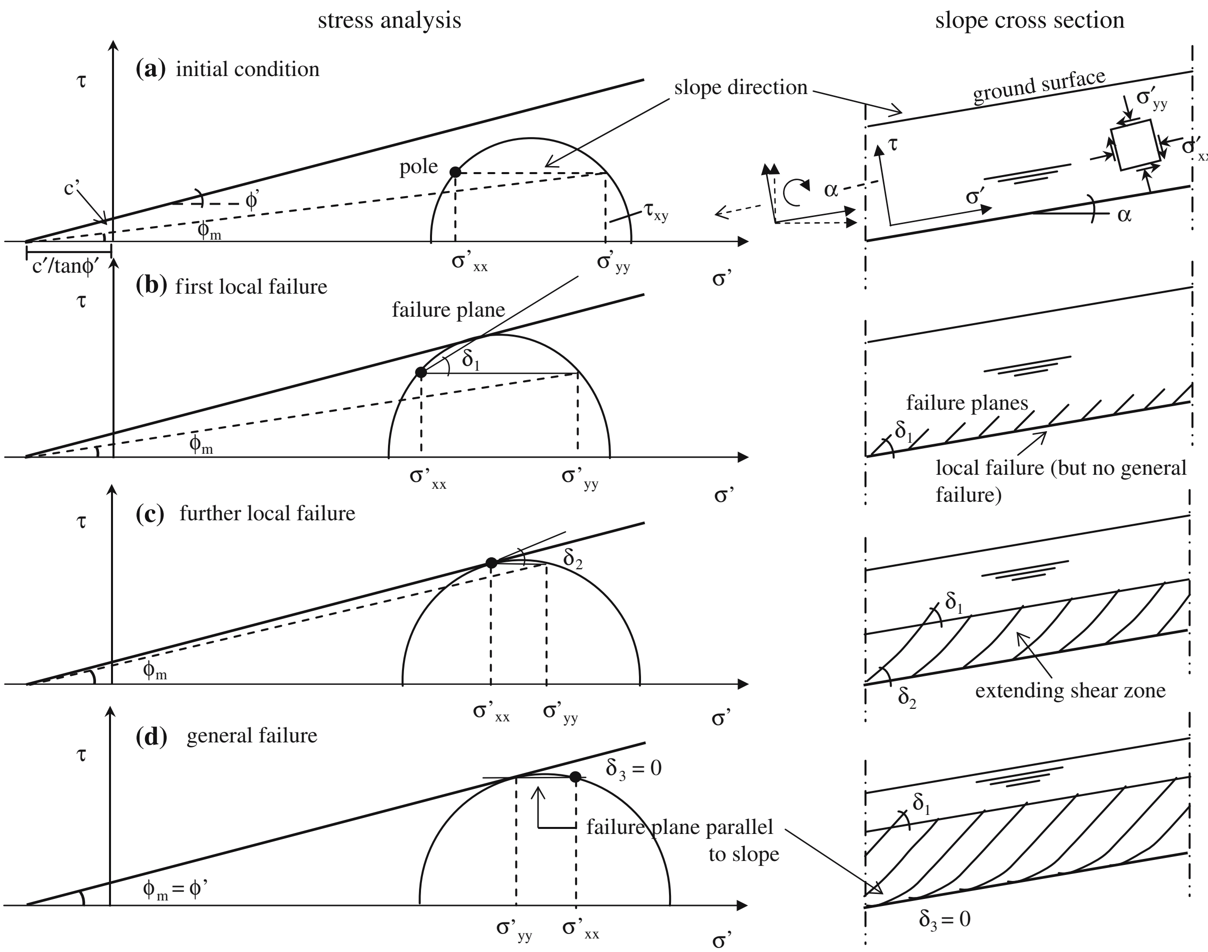


Fig. 11 Stress changes and formation of the shear zone in an infinite slope subjected to rising of the groundwater table. Note that in the definition of the state of stress, the σ' axis of the stress plane has to be associated to the ground surface direction

$$\sigma'_{yy} = \sigma_{yy} - u. \tag{3b}$$

The mobilized strength, as defined before, can be easily calculated from:

$$\tan\phi_{\mathrm{m}} = \frac{\tau_{xy}}{\frac{c'}{\tan\phi'} + \sigma'_{yy}}. \tag{4}$$

The mobilized strength clearly increases as the groundwater level increases as a consequence of σ'_{yy} decrease.

The safety factor against general failure, FS, at the depth Y, can be expressed as a function of ϕ_{m}:

$$\mathrm{FS} = \frac{\tau_{\mathrm{f}}}{\tau_{xy}} = \frac{c' + \sigma'_{yy}\cdot\tan\phi'}{\tau_{xy}} = \frac{\frac{c'}{\tan\phi'} + \sigma'_{yy}}{\tau_{xy}}\cdot\tan\phi' = \frac{\tan\phi'}{\tan\phi_{\mathrm{m}}} \tag{5}$$

Even when FS is larger than 1, it can be useful to check if soil failure is attained at the base of the soil layer. To this aim, the effective normal stress σ'_{xx} has to be calculated as a function of σ'_{yy}, through the effective stress ratio C.

To simplify the formal aspect of relationships reported in the following, a coefficient D can be defined, as:

$$D = \frac{\sigma'_{xx} + \frac{c'}{\tan\phi'}}{\sigma'_{yy} + \frac{c'}{\tan\phi'}} = \frac{C + \frac{c'}{\sigma'_{yy}\cdot\tan\phi'}}{1 + \frac{c'}{\sigma'_{yy}\cdot\tan\phi'}}. \tag{6}$$

Local soil failure occurs when the Mohr circle, expressed in terms of effective stresses, is tangent to the failure envelope. This condition is expressed by the following equation (Fig. 12):

$$R = \left(\frac{c'}{\tan\phi'} + \frac{\sigma'_{xx} + \sigma'_{yy}}{2}\right)\cdot\sin\phi' = 0.5\cdot\left(\frac{c'}{\tan\phi'} + \sigma'_{yy}\right)\cdot\sin\phi'\cdot(1+D). \tag{7}$$

Referring to the following simple equations,

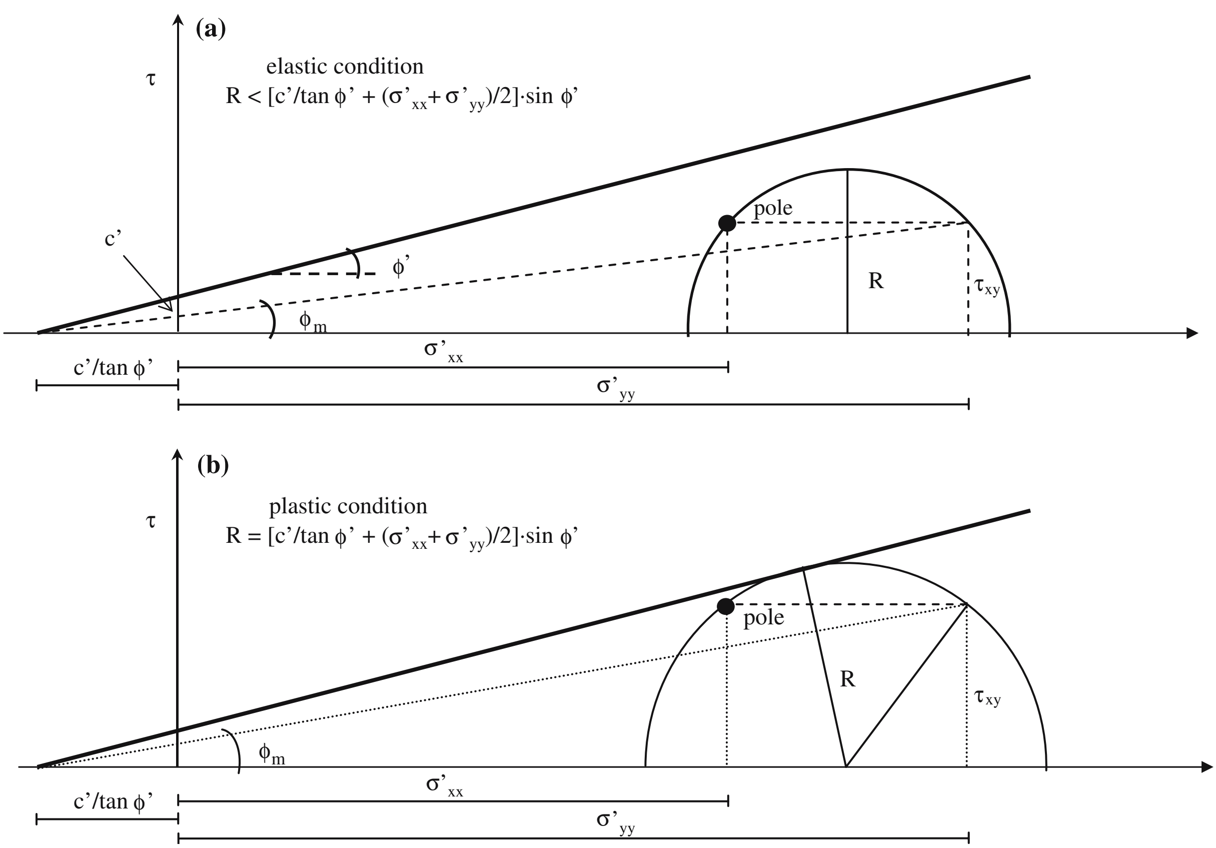


Fig. 12 State of stress at the base of the soil layer in elastic (**a**) and plastic (**b**) conditions

$$\frac{\sigma'_{yy} - \sigma'_{xx}}{2} = 0.5 \cdot \left(\frac{c'}{\tan\phi'} + \sigma'_{yy}\right) \cdot (1 - D) \tag{8a}$$

$$\tau_{xy} = \sqrt{R^2 - \left(\frac{\sigma'_{yy} - \sigma'_{xx}}{2}\right)^2} \tag{8b}$$

it is possible to express Eq. (4), by substituting Eqs. (7) and (8a) in Eq. (8b):

$$\tan\phi_{\mathrm{m}} = 0.5 \cdot \sqrt{\left[(1+D)^2 \cdot \sin^2\phi' - (1-D)^2\right]}. \tag{9}$$

Therefore, the mobilized friction angle can be expressed as a function of the friction angle itself and the coefficient D, which is related to the initial stress state in the slope.

Finally, using Eq. (5), the factor of safety against general failure when local failure occurs for the first time, $\mathrm{FS}_{\mathrm{lf}}$, may be expressed as a function of D, as:

$$\mathrm{FS}_{\mathrm{lf}} = \frac{\tan\phi'}{\tan\phi_{\mathrm{m}}} = \frac{\tan\phi'}{0.5 \cdot \sqrt{\left[(1+D)^2 \cdot \sin^2\phi' - (1-D)^2\right]}}. \tag{10}$$

The role of the geological history on $\mathrm{FS}_{\mathrm{1f}}$ can be investigated through Eq. (10) by assuming different values of C, thus of D. Equation 10 is plotted in terms of $\mathrm{FS}_{\mathrm{1f}}$ versus C in Fig. 13. There, thick layers are characterized by values of $\frac{c'}{\sigma'_{yy} \cdot \tan\phi'} \leq 7\%$ on the basis of plausible values of c', σ'_{yy} and ϕ' (c' =20 kPa, $\sigma'_{yy} \geq 500$ kPa, ϕ'=30°); for thin layers the maximum value of such a ratio is $\geq 70\%$ (c'=20 kPa, $\sigma'_{yy} \leq 50$ kPa, ϕ'=30°).

For thin layers there is a significant influence of the cohesion on the overall safety factor at first soil failure. Such an influence is stronger for smaller values of C. However, at small depths a high value of C is normally expected: for C larger

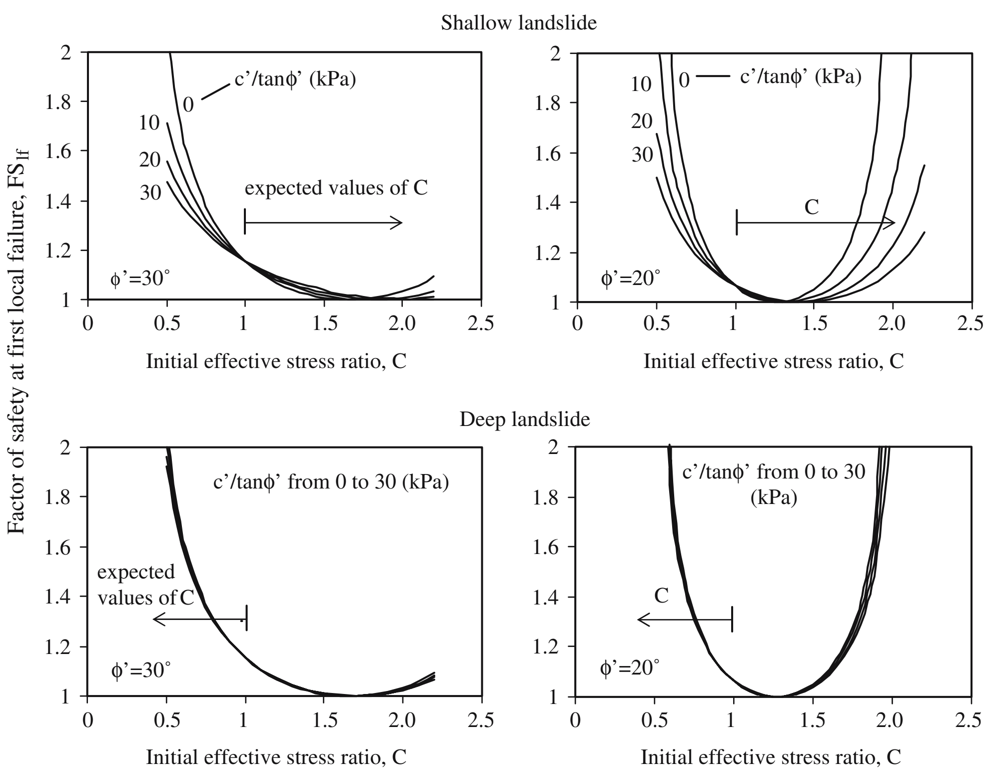


Fig. 13 Dependence of the overall safety factor of an infinite slope at first soil failure as a function of the initial state of stress and soil strength parameters

than 1, regardless of cohesion, first soil failure occurs for an overall safety factor only slightly larger than 1, thus just before general slope failure. In such a case, general slope failure is abrupt, because it is not preceded by significant displacements.

For thick layers, the influence of the cohesion is negligible. Since in this case smaller values of C are expected, first failure and significant displacements may occur well before general slope failure. In particular, where there is local failure, even if it is associated to a high overall safety factor, a small increase in stress level may involve significant soil displacements associated with induced plastic strains. As a matter of fact, it is well known that slope movements involving thick soil masses, sometimes classified as "deep gravitational movements", are generally characterized by slow displacements without abrupt slope failure.

The mechanics of general slope failure

Imposing $\phi_m=\phi'$ in Eq. (9), the value of (D_{crit} for which general slope failure occurs may be easily obtained:

$$D_{crit} = \frac{1+\sin^2\phi'}{1-\sin^2\phi'}. \tag{11}$$

Equation 11 shows that the stress field at the onset of general failure is unique, regardless of the initial value of C.

As anticipated before, during groundwater rising and formation of the plastic shear zone, principal stresses continuously rotate. Urciuoli (2002) showed that the angle ϑ, between the slope's inclination at soil bedrock interface and the minor principal stress, is a function of ϕ_m and may be obtained from the following equations:

$$\begin{aligned} \vartheta_1 &= 0.5\cdot(\psi+\phi_m) \quad \text{when } D_0<D_{crit} \\ \vartheta_2 &= 0.5\cdot(\pi-\psi+\phi_m) \quad \text{when } D_0<D_{crit} \end{aligned} \tag{12}$$

where $\psi = \arcsin\frac{\sin\phi_m}{\sin\phi'}$.

The first equation holds if the value of D at the beginning of the process (D_0) is smaller than D_{crit} and leads to $0 \le \vartheta_1 \le (\pi/4+\phi'/2)$; the second one holds for $D_0 \ge D_{crit}$ and leads to $(\pi/4+\phi'/2) \le \vartheta_2 \le \pi/2$. According to the Mohr's theory, the angle between the failure plane and the direction of the minor principal stress is equal to $\pi/4+\phi'/2$; therefore, the angle δ between the failure plane and the slope's inclination at soil bedrock interface (Fig. 14) is:

$$|\delta| = |\theta-(\pi/4+\phi'/2)|. \tag{13}$$

When $\phi_m=\phi'$, Eqs. 12 and 13 give $\vartheta=\pi/4+\phi'/2$ and $\delta=0$, respectively. Hence, one of the failure planes coincides with the slope direction provoking general slope failure.

Figure 15b shows the stress path at the base of an infinite slope having the geometrical and mechanical features reported, respectively, in Figs. 10 and 15a, as a consequence of pore pressure increase: the soil is cohesionless and is assumed to have a ductile behaviour. Since first local failure, a significant increase of the mean effective stress, s', occurs along the Coulomb line, because of the dilative behaviour of the soil and of restrained lateral strains.

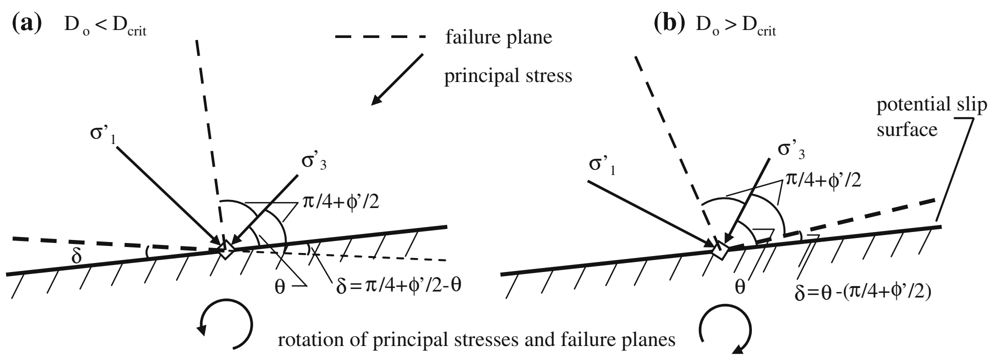


Fig. 14 Principal stresses and failure planes at the base of the soil layer as a function of D

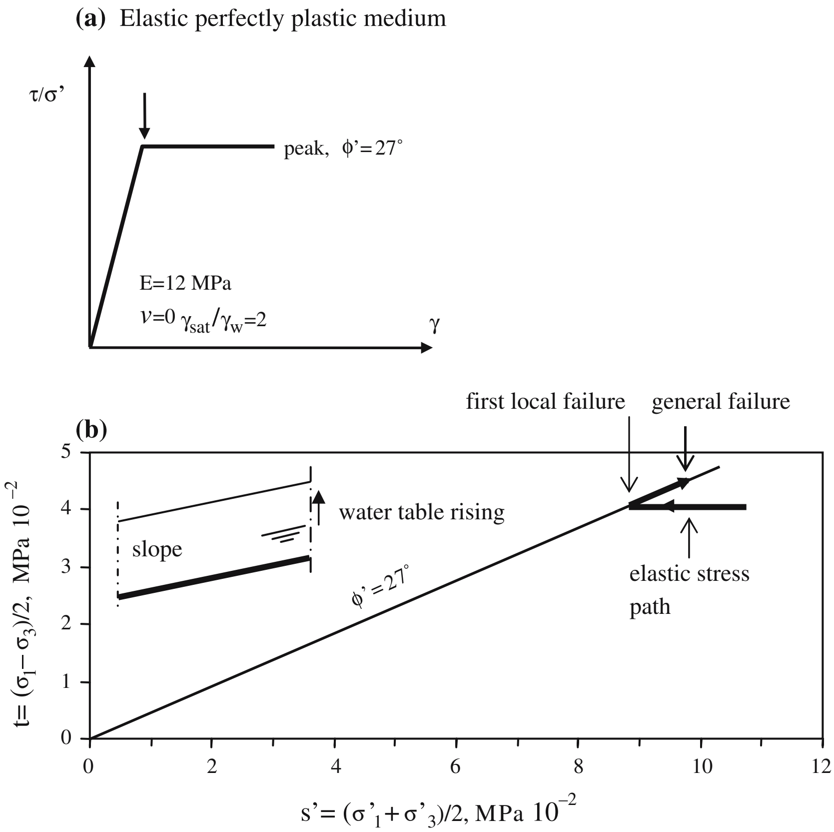


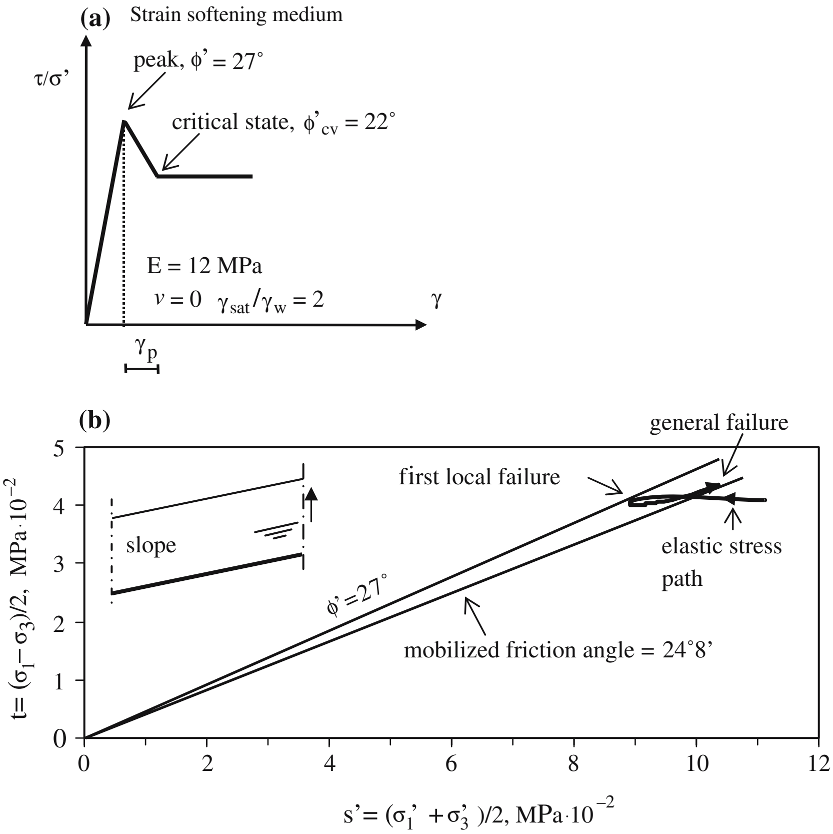
Fig. 15 Stress path until general slope failure for a ductile cohesionless soil in the scheme of Fig. 10 (from Picarelli et al., 2000)

The same case has been analyzed for a brittle soil (Fig. 16) characterized by a decrease of the friction angle from the peak (27°) to the critical state (22°), as a function of the plastic shear strain (γ_p). As a consequence of ground water table rising, slope failure occurs only after a significant rotation of the principal stresses. During this process, elastic strains are induced until mobilization of the peak strength, which implies local soil failure, but not general failure. Further pore pressure increase generates larger plastic strains with reduction of the operative shear strength until general slope failure, that can occur only when the Mohr's failure plane becomes parallel to soil bedrock interface. This condition occurs for a mobilized strength which is intermediate between the peak and the critical state: in the analyzed case (Fig. 16a), the mobilized friction angle is slightly greater than 24°.

The last example concerns a cohesive brittle soil, characterized by ϕ'=20° and $c'/\tan(\phi') = 20$ kPa: in thus case cohesion abruptly disappears as soil failure occurs. This provokes an increase of ϕ_m (see Eq. 4), even if pore pressure does not change after local failure. Principal stresses and failure planes rotate as ϕ_m increases, until general slope failure occurs. Figure 17b shows the influence of the initial effective stress field on the safety factor at first local failure. For a critical value of C (about 1.4) first local failure coincides with general slope failure. Figure 17c deals with the case C = 0.5. First failure occurs for FS = 1.42, when the failure planes are inclined of 22°6′ to the soil bedrock interface (Fig. 17b). The decrease of cohesion to 0 leads to general slope failure, since the failure planes rotate until they become parallel to soil–bedrock interface (δ=0°). In this case, formation of failure planes in the direction of the slope is a consequence of a decrease in cohesion.

As it is well known, the soil strength mobilized in first-failure landslides in brittle materials is intermediate between the peak and the critical strength (Skempton 1970; Chandler 1984). This is generally associated with softening,

Fig. 16 Stress path until general slope failure for a brittle cohesionless soil in the scheme of Fig. 10 (from Picarelli et al., 2000)



progressive failure and other phenomena (Skempton 1964, 1970; Bishop 1967). The described mechanical process may give another possible explanation of this result. In fact, since at the peak the failure planes are not parallel to the ground surface, general failure cannot occur and further plastic strains are required to bring slope to failure. These strains can provoke a decrease in the shear strength towards a value that is less than the peak and to slope failure (Urciuoli and Picarelli 2004).

Summary and conclusions

Slope failure is often localized being associated with formation of a shear zone. Since the initial state of stress is generally non-uniform, the described process is progressive being characterised by propagation of local soil failure in the subsoil. As a consequence, in brittle soils the shear strength mobilised at slope failure is intermediate between the peak and the residual value. However, local failure does not necessarily imply the attainment of a general slope failure. In fact, natural slopes may contain internal plastic zones which do not progress in nearby zones.

The analysis of the simple case of infinite slope subjected to a uniform rising of the groundwater table, gives interesting information suggesting new ideas for the interpretation of the mechanics of slides. In such a case, the state of stress is uniform along planes parallel to the ground surface, remaining uniform during rising of the groundwater table even though effective stress changes. Despite the uniformity of the state of stress, local soil failure may occur before general slope failure, if the inclination of the failure plane does not coincide with that of the slope. Further rising of the groundwater table is then required to cause slope collapse which occurs as a consequence of rotation of principal stresses until these can accommodate formation of a failure plane parallel to the ground surface. During this pro-

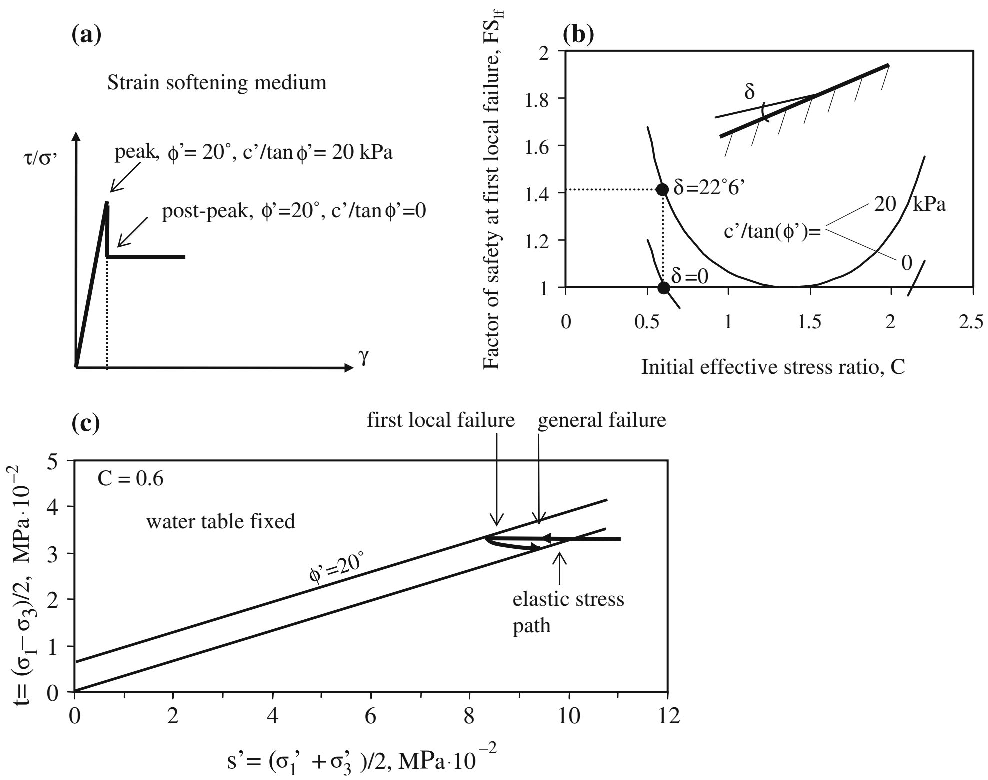


Fig. 17 Safety factor at first failure of a cohesive brittle soil (**a**) against the initial effective stress ratio, C (**b**), and stress path until general slope failure for C = 0.6 (**c**)

cess, a plastic shear zone forms at the base of the slope. Both the thickness of the plastic shear zone and displacements prior to slope failure depend on the shear strength parameters and on the initial state of stress.

In the case of brittle soil, rotation of the principal stresses brings about a decrease of the shear strength beyond the peak (strain-softening), but following a process which is different from progressive failure.

The mechanics of failure of natural slopes is much more complex than for the described model, because of the non-uniformity of the soil nature and properties, of the stress field and pore pressures, and of the induced state of stress. In general, it causes either rotation of principal stresses and formation of a shear zone, as in the described case, or propagation of the shear zone in the slope, as in the mechanism of progressive failure.

The scheme adopted in this paper could appear too idealized, due to simplified hypotheses previously introduced. Nevertheless, it quite satisfactorily simulates the mechanics of translational slides and can be applied to those situations where the curvature of the slip surface is negligible.

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