Slope stability

- Causes of instability
- Mechanics of slopes
- Analysis of translational slip
- Analysis of rotational slip
- Site investigation
- Remedial measures

Soil or rock masses with sloping surfaces, either natural or constructed, are subject to forces associated with gravity and seepage which cause instability. Resistance to failure is derived mainly from a combination of slope geometry and the shear strength of the soil or rock itself.

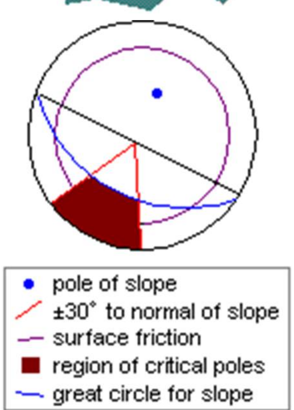
The different types of instability can be characterised by spatial considerations, particle size and speed of movement. One of the simplest methods of classification is that proposed by Varnes in 1978:

I. Falls
II. Topples
III. Slides rotational and translational
IV. Lateral spreads
V. Flows in Bedrock and in Soils
VI. Complex

Falls
In which the mass in motion travels most of the distance through the air. Falls include: **free fall**, movement by **leaps and bounds**, and **rolling** of fragments of bedrock or soil.

Topples
Toppling occurs as movement due to forces that cause an over-turning moment about a pivot point below the centre of gravity of the unit. If unchecked it will result in a fall or slide.

The potential for toppling can be identified using the graphical construction on a stereonet. The stereonet allows the spatial distribution of discontinuities to be presented alongside the slope surface. On a stereoplot toppling is indicated by a concentration of poles "in front" of the slope's great circle and within ± 30° of the direction of true dip.



Lateral Spreads
Lateral spreads are disturbed lateral extension movements in a fractured mass. Two subgroups are identified:

A. Where the spread is without a well-defined controlling basal shear surface or zone of plastic flow.
B. In which extension of rock or soil results from liquefaction or plastic flow of subjacent material.

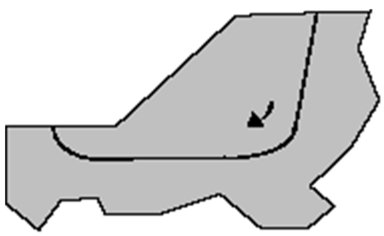
Flows
Two subtypes are identified:

A. In Bedrock
Where flows include spatially continuous deformation and superficial as well as deep creep. They also involve extremely slow deep creep and extremely slow and generally non-accelerating differential movements among relatively intact units. Movements may:

- be along shear surfaces that are apparently not connected,
- result in folding, bending or bulging, or
- roughly simulate those of viscous fluids in distribution of velocities.

B. In Soils
In which the movement within the displaced mass is such that the form taken by moving material, or the apparent distribution of velocities and displacements, resemble those of viscous fluids. The slip surfaces within the moving material are usually not visible or are short-lived. The boundary between the moving mass and material may be a sharp surface or differential movement or a zone of distributed shear. Movement ranges from extremely rapid to extremely slow.

Complex
Complex movement is by a combination of one or more of the five other principal types of movement described by Varnes' Classification. Many landslides are complex, although one type of movement generally dominates over the others at certain areas within a slide or at a particular time.



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back to Slope stability

Causes of instability

- Influence of factors on the factor of safety
- Classification of causes of instability
- Further reading

Starting from the general definition for the factor of safety,
$F_s = \tau_f / \tau$ = (shear stress at failure) / (shear stress)

Terzaghi divided landslide causes into external causes which result in an increase in shearing stress and internal causes which result in a decrease of the shearing resistance.

Varnes pointed out that there are a number of external or internal causes which may be operating either to reduce the shearing resistance or increase the shearing stress. There are also causes affecting simultaneously both terms of the factor of safety ratio.

The influence of different contributory factors on the factor of safety of a slope varies in time due to a variety of factors.

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back to Causes of instability

Classification of causes of instability

It must be appreciated that the causes of instability are often complex and any attempt at classification will be approximate and incomplete. The Working party on World Landslide Inventory have proposed a list of causal factors grouped under four main headings:

- Ground conditions
- Geomorphological processes
- Physical processes
- Man-made processes

Ground conditions

- Plastic weak material
- Sensitive material
- Collapsible material
- Weathered material
- Sheared material
- Jointed or fissured material
- Adversely oriented mass discontinuities (including bedding, schistosity, cleavage, faults, unconformities, flexural shears, sedimentary contacts)
- Contrast in permeability and its effects on ground water
- Contrast in stiffness (stiff, dense material over plastic materials)

Geomorphological processes

- Tectonic uplift
- Volcanic uplift
- Glacial rebound
- Fluvial erosion of the slope toe
- Wave erosion of the slope toe
- Glacial erosion of the slope toe
- Erosion of the lateral margins
- Subterranean erosion (solution, piping)
- Deposition loading the slope crest
- Vegetation removal (by erosion, forest fire, drought)

Physical factors

- Intense, short period, rainfall
- Rapid melt of deep snow
- Prolonged high precipitation
- Rapid drawdown following floods, high tides or breaching of natural dams
- Earthquake
- Volcanic eruption
- Breaching of crater lakes
- Thawing of permafrost
- Freeze and thaw weathering
- Shrink and swell weathering of expansive soils

Man-made processes

- Excavation of the slope or at its toe
- Loading of the slope or at its crest
- Drawdown (of reservoirs)
- Irrigation
- Defective maintenance of drainage system
- Water leakage from services (water supplies, sewers, stormwater drains)
- Vegetation removal (deforestation)
- Mining and quarrying (open pits or underground galleries)
- Creation of dumps of very loose waste
- Artificial vibration (including traffic, pile driving, heavy machinery)

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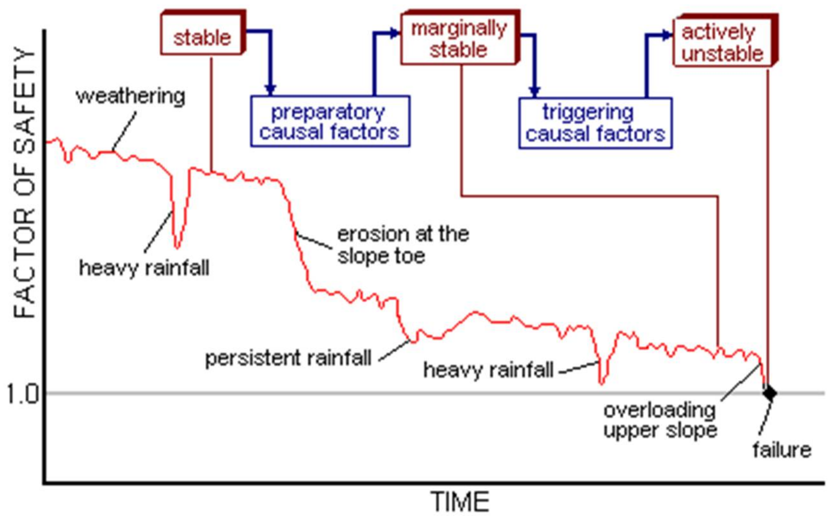
back to Causes of instability

Influence of different factors on instability



Reference

It can be seen that short-term variations in factor of safety may occur due to seasonal variations in groundwater levels while longer term trends may reflect the influences of weathering or longer term changes in groundwater conditions. This approach is useful in emphasising that landslides (and slope instability in general) may not be attributable to a single causal factor.

From the physical point of view it may be useful to visualise slopes as existing in one of the following three stages:
Stable: the margin of stability is sufficiently high to withstand all destabilising forces.
Marginally stable: likely to fail at some time in response to destabilising forces reaching a certain level of activity.
Actively unstable: slopes where destabilising forces produce continuous or intermittent movements.

These three stability stages provide a useful framework for understanding the causal factors of instability and classifying them into two groups on the basis of their function:

Preparatory causal factors - which make the slope susceptible to movement without actually initiating it and thereby tending to place the slope in a marginally stable state.

Triggering causal factors - which initiate movement. These causal factors shift the slope from a marginally stable state to an actively unstable state.

Examples of External causes Resulting in Increased Shearing Stress

- Geometrical changes
- Unloading the slope toe
- Loading the slope crest
- Shocks and vibrations
- Drawdown, and
- Changes in water regime

Examples of Internal Causes Resulting in Decreased Shearing Resistance

- Progressive failure,
- Weathering,
- Seepage erosion

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back to Causes of instability

Mechanics

- Loads
- Pore pressure
- Earthquakes
- Peak, critical state and residual strength
- Stress changes in slopes
- Choice of strength parameters
- Choice of factor of safety

In every slope there are forces which tend to promote downslope movement and opposing forces which tend to resist movement.

A general definition of the factor of safety (F_s) of a slope results from comparing the down slope shear stress (τ) with the shear strength (τ_f) of the soil along an assumed or known rupture surface:
$F_s = \tau_f / \tau$

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back to Mechanics

Loads

- Neutral Point Concept

Changes produced by loading or unloading are widely recognised as being mechanisms by which instability can occur. The effects of variations in loading can be considered using the concept of the 'neutral point' developed by Hutchinson in 1977.

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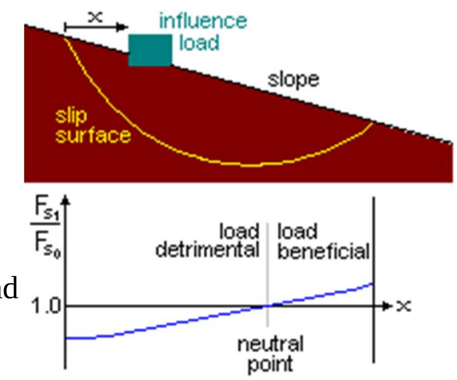
back to Loads

Neutral Point Concept

- Location of the drained and undrained neutral points
- Interpreting the neutral point concept
- Further Reading



Essentially the neutral point method considers the ratio of the new to old factors of safety (F_{s1} / F_{s0}) for a potential (or existing) failure surface, as a load is placed at different points on the ground above it.

If placed toward the toe of the failure surface, the load will produce an increase in the factor of safety (i.e. the loading is beneficial and F_{s1} / F_{s0} increases), but if placed toward the head of the failure surface, it produces a decrease in the factor of safety (i.e. loading is detrimental and F_{s1} / F_{s0} decreases).

There is clearly an intermediate position where the loading causes no change in the factor of safety for the failure surface and F_{s1} / F_{s0} = 1.0, this is termed the 'neutral point'.

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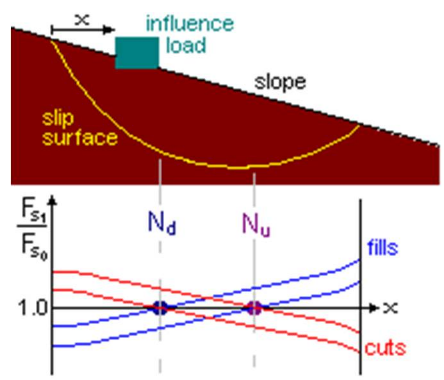
back to Neutral Point Concept

Location of the drained and undrained neutral points



It can be shown that two extreme neutral points exist, depending upon whether the loading is applied under drained or undrained conditions. For the undrained condition ($\bar{B}$=1.0), the neutral point (N_u) occurs at the point of zero inclination of the failure surface, whilst for drained conditions ($\bar{B}$=0) the neutral point (N_d) occurs where the inclination of the failure surface α is given by

$$\tan\alpha = \frac{\tan\phi'}{F_s}$$

Drainage conditions between these two extremes (1.0 > $\bar{B}$> 0) have a corresponding intermediate neutral point (N_i).

Similar effects can be observed for the removal of loads, by for example making cuts in the slope.

F_{so} = factor of safety before addition or removal of load.
F_{s1} = factor of safety after addition or removal of load.

α = local dip of slip surface (units: degrees).
ϕ' = angle of friction in terms of effective stress (units: degrees).
$\bar{B}$= pore pressure coefficient.
N_u = neutral point for undrained condition.
N_d = neutral point for drained condition.
N_i = neutral point for intermediate drained conditions.

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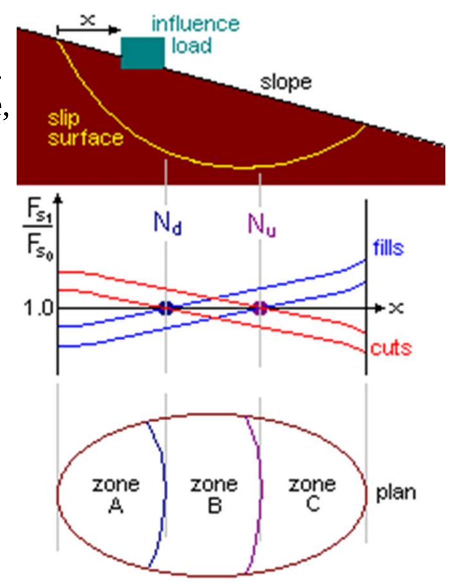
back to Neutral Point Concept

Interpreting the neutral point concept



The effects due to the location of load vary according to its position in relation to the drained and undrained neutral points. Three zones describing the effect of adding loads can therefore, be defined:

· Zone A : Always detrimental,
· Zone B : Detrimental in the short term; beneficial in the long term, and
· Zone C : Always beneficial.

Three zones can also be defined for the effects of reduced loading (i.e. cutting):

· Zone A : Always beneficial,
· Zone B : Beneficial in the short term; detrimental in the long term, and
· Zone C : Always detrimental.

The simple concept of the neutral point must, however, be applied with care. For example, the engineer must recognise that the addition or removal of load may cause a change in the position of the critical failure surface (and hence the neutral points). Particular care must be exercised when dealing with potential deep/shallow slips, since the addition of load may, in these cases, cause improvements in the stability condition of a shallow slip surface, whilst reducing the stability condition of a deeper slip surface.

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back to Neutral Point Concept

Further reading

Hutchinson, J.N., "Assessment of the effectiveness of corrective measures in relation to geological conditions and types of slope movement." General Report to Theme 3. Symposium on Landslides and other Mass Movements, Prague, September 1977. Bulletin, International Association of Engineering Geology, No. 16, 1977, pp. 131-155. Reprinted (1978) in Norwegian Geotechnical Institute Publication, No. 124, pp. 1-25.

Hutchinson, J.N. , "Engineering in a landscape." Inaugural Lecture, 9 October 1979, Imperial College of Science and Technology, University of London, London, England. 1983.

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back to Mechanics

Pore pressures

In order to estimate the factor of safety F_s for a slope in terms of effective stress (i.e. in the long-term condition), the pore water pressure must be known. This is frequently the greatest source of inaccuracy in slope stability work, since the determination of the most critical conditions of pore water pressure is complex and costly. The following three sets of conditions are usually considered for constructed slopes:
· End of construction
· Steady seepage
· Rapid drawdown

In natural slopes, the distribution of pore water pressure may be highly complicated due to changes in soil type, anisotrophy etc. The pore pressures are detemined from site measurements using observation wells or piezometers. Monitoring must be continued over long periods of time in order to define the worst or most critical conditions. Methods for the in-situ measurement of pore water pressure are described by Clayton, Matthews and Simons.

Clayton, C.R.I., Matthews, M.C., and Simons, N.E.,1995. Site Investigation. Blackwell Science. 584 pp.

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back Pore pressures

End of construction

Analyses of the short-term condition of stability are normally performed in terms of total stress, with the assumption that any pore water pressure set up by the construction activity will not dissipate at all. In many earth dams or large embankments, however, the construction period is relatively long, and some dissipation of the excess pore water pressure is likely. Under these conditions, a total stress analysis would yield a value of F_s on the low side, possibly resulting in un-economic design.

At any point, the pore water pressure is given by

$$u = u_o + \Delta \upsilon$$

The pore pressure coefficient $\bar{B}$ is defined as

$$\bar{B} = \Delta \upsilon / \Delta \sigma_1$$

Hence,

$$u = u_o + \bar{B} \Delta \sigma_1$$

The pore pressure ratio is defined as

$$r_u = u / \gamma h$$

Then,

$$r_u = \frac{u_o}{\gamma h} + \bar{B} \frac{\Delta \sigma_1}{\gamma h}$$

Generally,

$$\Delta \sigma_1 = \gamma \eta$$

Therefore,

$$r_u = \frac{u_o}{\gamma h} + \bar{B}$$

The pore pressure coefficient $\bar{B}$ may be determined in the triaxial apparatus. In embankments etc., if the soil is placed at a water content below the optimum water content, the value of u_o may be close to zero, and in this case $r_u = \bar{B}$.

In order to increase the rate of dissipation of pore water pressure (and hence reduce the value of r_u), drainage layers may be incorporated in the embankment.

u_o = initial pore water pressure
$\Delta \upsilon$ = change in pore water pressure due to a change in the major principal stress $\Delta \sigma_1$
γ = unit weight of soil
h = depth of element
$\bar{B}$ = pore pressure coefficient
r_u = pore pressure ratio
h_w = head of water
γ_w = unit weight of water

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back to Pore pressures

Steady seepage

Under conditions of steady seepage, the pore water pressure can be obtained from the flow net. The pore water pressure at a point on the actual or assumed slip surface is obtained from the value of the equipotential passing through the point.

Referring to the figure, the pore water pressure at point P is obtained by constructing the equipotential line through P and is then equal to the head given by h_w.

$u = \gamma_w \, . \, h_w$

Then,

$$r_u = \frac{u}{\gamma h} = \frac{\gamma_w h_w}{\gamma h}$$

In homogeneous conditions, r_u may attain values approaching 0.45. Internal drainage layers will produce lower values.

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back to Pore pressures

Rapid drawdown

In earth dams, rapid reductions in the water level produce significant and potentially dangerous changes in pore water pressure. This occurs because the water in the soil tends to flow back into the reservoir through the upstream face. In this scenario, even a period of some weeks may bring about a 'rapid' change in the pore water pressure distribution.

The pore water pressures under conditions of rapid drawdown are determined using the following procedure.

In the figure, consider the point P on a trial slip surface. Under conditions of steady seepage, the pore water pressure at P is obtained from the flow net, giving:
$u_o = \gamma \, (h + h_w - h')$

It is assumed that the total major stress at P is given by the overburden pressure,
$\sigma_1 = \gamma \, h$

If under drawdown, the phreatic surface falls to a level below h_w, then the change in total major principal stress is given by
$\Delta\sigma_1 = - \gamma_w \, h_w$
The corresponding change in pore water pressure is
$\Delta\upsilon = - \bar{B}\gamma_w \, h_w$

Therefore, immediately following drawdown, the pore water pressure at P is
$u = u_o + \Delta\eta$
$= \gamma_w [h + h_w - h'] - \bar{B}\gamma_w h_w$
$= \gamma_w [h + h_w (1 - \bar{B}) - h']$

And, if
$r_u = u / \gamma\eta$

Then,
$$r_u = \frac{\gamma_w}{\gamma}\left[1 - \frac{h_w}{h}(1-\bar{B}) - \frac{h'}{h}\right]$$

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back to Pore pressures

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Clayton, C.R.I., Matthews, M.C., and Simons, N.E., 1995. Site Investigation. Blackwell Science. 584 pp.

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back to Mechanics

Earthquakes

Earthquakes result from the sudden release of elastic strain energy stored in the Earth's crust. Stress accumulates locally from various causes until it exceeds the strength of the rocks, when slip occurs by brittle failure on dislocations or fractures known as faults. Earthquakes give rise to two types of surface displacement: permanent offsets on the fault itself, and transient displacement resulting from the propagation of seismic waves away from the source. A small movement on a fault may produce a considerable shock because of the energy involved. Earthquakes range from slight tremors which do little damage, to severe shocks which can cause widespread damage including the initiation of landslides, collapse of buildings and fracturing of supply mains and lines of transport.

The surface displacement associated with earthquakes may range from a few centimetres up to several meters. For example in the disastrous earthquake which affected San Francisco in 1906, the motion of the ground was mainly horizontal along the San Andreas fault with one side moving approximately 4.6m relative to the other.

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back to Earthquakes

Earthquakes and slope stability

Earthquakes can affect slope stability in three ways:

Earthquakes produce **horizontal and vertical accelerations** in soil masses. The horizontal accelerations may reach as much as 0.5g (where g is gravitational acceleration), altering the distribution of forces in hillslopes in a manner equivalent to a temporary steepening of the slope.

Rapid repeated stress fluctuations (due to cyclic loading and unloading) can induce changes in the **pore fluid pressures** which may lead to liquefaction.

Earthquake shaking may change the applicable **shear strength** properties.

The detailed discussion of the effects of earthquakes on slope stability is outside the scope of this section. It is clear however that earthquakes do influence slope stability and can lead to slope failures on a catastrophic scale (see, for example, Seed).

Most of the analytical work concerned with the stability of slopes subjected to earthquakes has been performed in connection with earth and rock-fill dams where there is great potential for damage and loss of life if failure occurs. Much of this work is also applicable to natural slopes; an excellent review of the factors involved in determining the stability of slopes under earthquake shock is given by Seed.

Seed, H.B. 1968. Landslides during earthquakes due to soil liquefaction. A.S.C.E., Journal of the Soil Mechanics and Foundations Division, 94, 5, 1055-1122.
Seed, H.B. 1979. Considerations in the earthquake - resistant design of earth dams. Geotechnique, 29, 3, 215-263.

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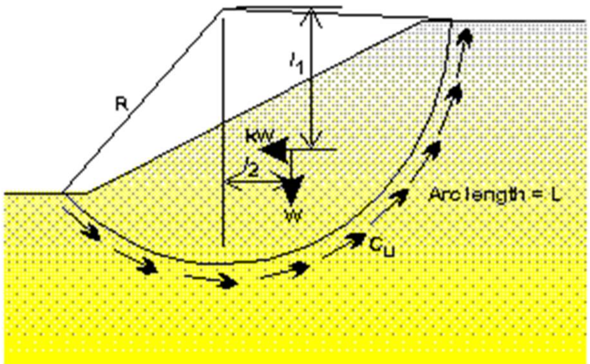
back to Earthquakes and slope stability

Stability analysis including seismic effects



The simplest way of including seismic effects is to perform a limit equilibrium analysis where the forces induced by the earthquake accelerations are treated a horizontal force. Vertical forces may also be caused by the earthquake but these are ignored in this simple form of analysis. The principle of this method is illustrated opposite, where a horizontal force F_h due to the earthquake is assumed to act through the centre of gravity of the soil involved in the predicted or actual failure. It is assumed that:

$F_h = kw = k\, mg$
where m is the mass of the soil.

Thus the seismic coefficient k is a measure of the acceleration of the earthquake in terms of g.

For a purely cohesive soil, the factor of safety F_s is given by

$$F_s = \frac{s_u L R}{W \ell_2 + k W \ell_1}$$

where s_u is the undrained strength of the soil.

This approach may also be used in conjunction with the method of slices for soils possessing cohesion and friction. For example, Sarma presented a procedure where the earthquake induced forces are considered by applying a horizontal force kW_i to the slice where W_i is the weight of slice i. The method then involves the computation of the critical horizontal acceleration (i.e. value of k) required to bring the soil above the slip surface into a state of limiting equilibrium: this critical acceleration can then be used as an index of stability.

An alternative method of analysis, using seismic coefficients has also been presented by Sarma.

Although the pseudo-static methods of analysis have many limitations, they are widely used. One of the problems occurs in estimating the value of the seismic coefficient k to be used in the analysis. This coefficient depends on the accelerations caused by the earthquake and may be difficult to define.

Recently, dynamic methods of analysis have been developed which allow for the inclusion of the full time history of the earthquake acceleration. Such methods generally allow the displacements of a potential slide mass along an assumed failure surface to be estimated. A dynamic finite element approach has been described by Seed.

Sarma, S.K., 1973. Stability analysis of embankments and slopes. Geotechnique, 23, 3, 423-433.
Sarma, S.K. 1979. Stability analysis of embankments and slopes. A.S.C.E., Journal of the Geotechynical Engineering Division, 105, 12, 1511-1524.
Seed, H.B. 1966. A method for the earthquake resistant design of earth dams. A.S.C.E., Journal of the Soil Mechanics and Foundations Division, 92, 1, 13-41.

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back to Earthquakes and slope stability

Shear strength parameters in seismic analysis

Generally, in the pseudo-static methods of analysis, the shear strength parameters used are those measured in conventional shear strength tests. This assumption appears to be justified by the relatively few examples where problems have arisen.

However, in cases where seismic loading may cause movement along an existing discontinuity (such as a fault or old slip surface), significant decreases in the value of f ' have been reported by Skempton.
Skempton, A.W. 1985. Residual strength of clays in landslides, folded strata and the laboratory. Geotechnique, 35, 1, 3-18.

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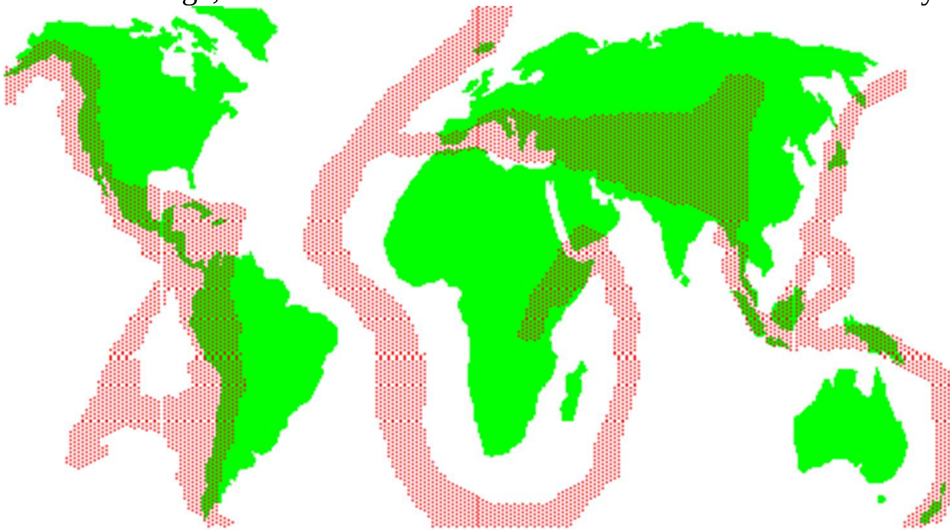
back to Earthquakes

Earthquake mechanism

Elastic strain energy may accumulate in rocks anywhere in the Earth's crust. Currently the plate tectonic model is used to explain that the most seismically active regions are at the 'active' margins of plates. Thus, many earthquake centres are located along two belts of the earth's surface: one belt extends around the coastal regions of the Pacific, from the East Indies through the Phillipines, Japan, the Aleution Islands and down the western coasts of North and South America; the other runs from Central Europe through the eastern Mediterranean to the Himalayas and the East Indies, where it joins the first belt (see figure opposite). Both of these belts are in places parallel to relatively young fold-mountain chains (e.g. the Andes), where much faulting is associated with the crumpled rocks. Many volcanoes are also situated along the active earthquake belts. Many shocks also occur in zones of submarine fault activity such as the mid-Atlantic ridge, and in fault-zones on continents such as the Rift Valley system of Africa.



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back to Earthquakes

Mercalli scale of earthquake intensity

The intensity of an earthquake is estimated from the effects produced in the affected area. The most common scale of intensity used is the Mercalli Scale, which has twelve grades as follows:

i) Detected only by instruments

ii) Felt by persons at rest

iii) Felt indoors; vibration like passing of a truck

iv) Hanging objects swing; windows, door rattle

v) Felt outdoors; some objects displaced; pendulum clocks stop

vi) Strong: felt by all, many frightened; weak masonry cracked

vii) Damage to some buildings; weak chimney fall; waves on ponds

viii) Destructive: much damage to buildings, ground cracks, flow of springs affected

ix) General damage (including reservoirs), buried pipes broken

x) Disastrous: framed buildings destroyed, rails bent, landslides

xi) Few structures left standing, fissures opened in ground

xii) Catastrophic: damage nearly total, ground twisted and warped.

A more detailed description of the Mercalli scale, and of other ways for quantifying earthquakes is given by Skipp and Emreaseys.

Skipp, B.O. and Emreaseys, N.N., 1987. Engineering seismology, in Ground Engineer's Reference Book, ed. by F.C. Bell, Butterworths, London.

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back to Earthquakes

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back to Mechanics

Peak, critical state and residual strength

When a soil is subjected to shear, an increasing resistance is built up. For any given applied effective pressure, there is a limit to the resistance that the soil can offer, which is known as the peak shear strength τ_p. Frequently the test is stopped immediately after the peak strength has been clearly defined. The value τ_p has been referred to, in the past, as simply the shear strength of the clay, under the given effective pressure and under drained conditions.

If the shearing is continued beyond the point where the maximum value of the shear strength has been mobilised, it is found that the resistance of the clay decreases, until ultimately a steady value is reached, and this constant minimum value is known as the residual strength τ_r of the soil. The soil maintains this steady value even when subjected to very large displacements.

In the absence of pre-existing failures the choice is between the peak or critical state strength. In uncemented soils the peak strength is associated with dilation and occurs at relatively small strains or displacements of the order of 1% or 1mm. The critical state strength is the shearing resistance for constant volume straining and occurs at strains or displacements of the order of 10% or 10mm. In many slopes ground movements and strains are relatively large and exceed the small movements required to mobilise the peak state. In addition, there is evidence that the peak value reflects the nature of the laboratory test procedure and gives an unconservative result in slope stability analysis.

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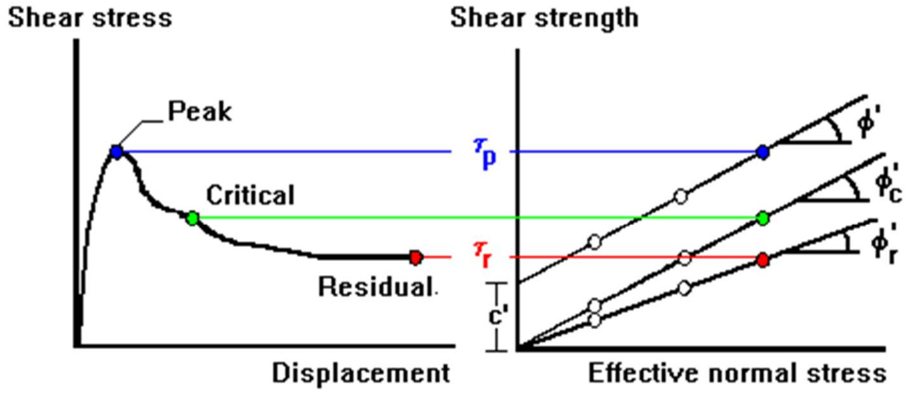
back to Peak and residual strength

Typical results for a drained test on clay



Shear strength tests

The figure shows a typical plot for a shear test which has been taken to displacements large enough to mobilise the residual strength. The decrease in shear strength from the peak to the residual condition is associated with orientation of the clay particles along shear planes.

Further tests could be made on the same clay but under differing effective pressures. The results previously described would again be obtained, and from a number of tests it would be noticed that the peak and residual shear strengths would define envelopes in accordance with the Coulomb-Terzaghi relationship, as shown. Thus, the peak strengths can be expressed as:

$\tau_p = c' + \sigma' \tan\phi'$

and the residual strengths can be expressed as:

$\tau_r = c'_r + \sigma \tan\phi'_r$

For critical state analysis the strength parameter s_u or ϕ'_c should be used.

τ_p = peak shear strength
τ_r = residual shear strength
c' = apparent cohesion
c'_r = residual apparent cohesion
ϕ' = angle of shearing resistance

ϕ'_r = residual angle of shearing resistance
σ' = applied effective pressure

---

back to Peak and residual strength

Typical values of strength parameters

	Peak shear strength		Residual shear strength		Critical state
Soil	c'	f'	c'_r	f'_r	f'c
	kN/m²	degrees	kN/m²	degrees	degrees
London clay, brown	35	20	0	13	20
London clay, blue	25	23	0	14	22
Upper Lias clay	20	24	0	15	22
Cucarasha shale	30	23	0	8	22
Upper Siwalik, Jari	40	22	0	18	20
Boulder clay, Selset	10	32	0	30	32

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back to Peak and residual strength

The significance of residual strengths

The residual shear strength condition is of considerable practical importance since, if the soil *in situ* already contains slip planes or shear surfaces, then the strength operable on these surfaces will be less than the peak strength, and if sufficient displacement has taken place, the strength may be as low as the residual strength.

There are a number of circumstances, as a result of which shearing of the soil may already have taken place, and the principal processes, summarised by Morgenstern et al., are:

landsliding,
tectonic folding,
valley rebound,
glacial shove,
periglacial phenomena and
non-uniform swelling.

The identification of the existence of shear surfaces is a problem of great importance during any site investigation, particularly where mass movements are involved.

back to Peak and residual strength

Measurement of residual shear strength

- Ring shear

It is generally accepted that the residual shear strength of a soil is independent of stress history effects, not influenced by specimen size, and rate-dependent to only a small extent unless very rapid rates of shearing are used. The major difficulty in determining the residual shear strength lies in the fact that large displacements may be necessary to achieve the required degree of orientation of the particles.

The methods of measuring residual shear strength in the laboratory are given below. The most satisfactory methods, in many ways, are to obtain undisturbed samples which contain a natural slip surface and then test them either in the shear box or triaxial apparatus so that failure occurs by sliding along the existing slip plane. Alternatively, an artificial slip plane can be produced by cutting the specimen with a thin wire-saw. Much of the early work on determining the residual shear strength of soils in the laboratory was performed using multi-reversal type tests in the shear box on previously un-sheared material.

The results of tests to measure residual shear strength in the shear box and triaxial apparatus have been reported by Skempton and Petley. There are practical difficulties with each of these tests, and they also have the major disadvantage that none of them permits the complete shear-stress-displacement relationship to be obtained.

Methods for measuring residual shear strength

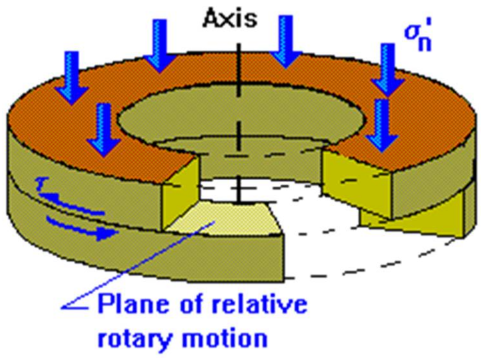
Shear box
(a) Tests on natural shear surfaces
(b) Reversal-type tests
(c) Cut-plane tests

Triaxial
(a) Tests on natural shear surfaces
(b) Cut-plane tests

Ring shear

back to Measurement of residual shear strength

Ring shear



Ring Shear Apparatus

The large displacements required to define the complete shear-stress-displacement relationship can be obtained by using the ring-shear (or torsional shear) apparatus. The apparatus, shown diagrammatically, consists of two pairs of metal rings which hold an annular sample. The sample is subjected to a normal stress and then one pair of rings (normally the lower pair) is subjected to rotation. It is therefore a form of direct shear test, and failure occurs along a predetermined plane, as with the shear box. This type of apparatus was probably first used by Hvorslev and Tiedemann. More recent designs of the ring-shear apparatus have been described by Bishop et al. and Bromhead.
Hvorslev, M.J. (1973). Über die Festigkeitseigenschaften gestörter bindiger Böden. *Ingenior Skriftor A.*, Copenhagen, 45.
Tiedemann, B. (1937). Über die Schubfestigkeit bindiger Böden. *Bautechnik*, **15**,
Bishop, A.W., Green, G.R., Garga, V.K., Andresen, A., and Brown, J.D. (1971). A new ring-shear apparatus and its application to the measurement of residual strength. *Geotechnique*, **21**, 273-328.
Bromhead, E.N. (1979). A simple ring shear apparatus. *Ground Eng.*, 12, 40-44.

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back to Peak and residual strength

Further reading

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back to Mechanics

Stress changes in slopes

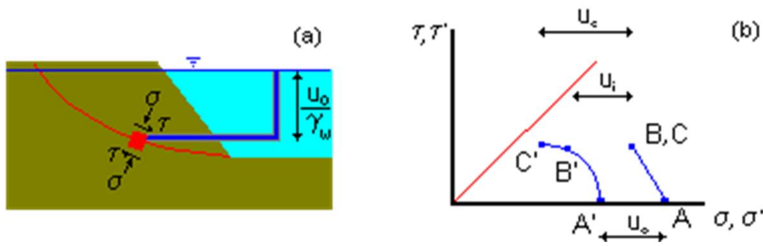
Any sudden changes in loading on a slope will lead to changes in the pore pressures. Natural slopes are usually eroded very slowly and the soil is essentially drained throughout the process. This means that the pore pressures are governed by steady seepage from the ground towards the excavation and their magnitude can be obtained from a flownet. Man-made slopes are constructed relatively quickly, and in soils with low permeability, such as clay, there will be inadequate time during the construction period for the pore pressures to adjust to the new loading conditions, and the soil will then be essentially undrained.

---

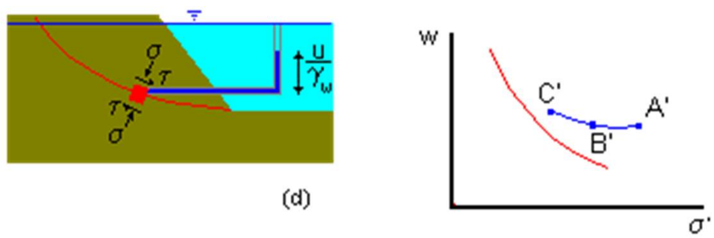
back to Stress changes in slopes

Changes in stress during undrained slope excavation

The changes in total and effective stress during undrained slope excavation are shown below. In this simple example, the excavation is assumed to be kept full of water so that the initial and final pore pressures are identical.



In (a) the total stresses on an element on a slip surface are τ and σ, and the pore pressure is indicated by the height of water in the standpipe. Obviously, before any excavation, the water in the standpipe is level with the phreatic surface, and the initial pore pressure can be represented by u_o. This initial stress state is represented on (b) by the points A (in terms of total stress) and A' (in terms of effective stress).

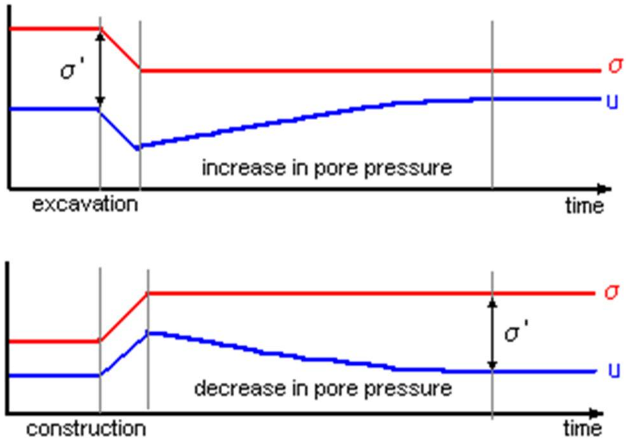


As excavation proceeds, the value of σ decreases and τ increases due to the increase in slope height and/or angle. The total stress path is AB. The effective stress path is A'B', corresponding to undrained loading at constant water content, as indicated in (c).

The precise stress path A'B' in (c) will depend upon the characteristics of the soil and whether the soil is normally or over consolidated.

Since the process of excavation leads to reductions in loading, the pore pressure immediately after construction (u) is less than the initial pore pressure (u_o), and so the initial excess pore pressure is negative (i.e. the level of the water in the standpipe is below the phreatic surface as shown in (d). As time passes, the total stress remains unchanged at B (because there is no change in the geometry of the slope). The negative excess pore pressures dissipate, leading to an increase in pore pressure. The normal effective stresses on the failure surface will decrease and swelling takes place as indicated in (b) and (c). The final state at C' corresponds to a steady state pore pressure after swelling has been completed. In this simple example, the excavation is kept full of water, and thus the initial and final pore pressures are equal (i.e. $u_o = u_c$).

Failure of the slope will take place if the states of all elements along the slip surface reach the failure line. If B' reaches the failure line, then the slope fails during the course of undrained excavation (i.e. during construction); whilst if C' reaches the failure line, it does so some time after construction has been completed. The distance of the points B' and C' from the failure line is a measure of the factor of safety of the slope.



These changes in the values of total stress and pore pressure with time are shown here.

For the case considered above, the initial (u_o) and final (u_c) values of pore pressure are identical, but in general, this may not be the case.

For cutting slopes, the effective stress reduces with time, leading to leass shear resistance and a lower factor of safety. Consequently the critical time in the life of these slopes is likely to be in the long-term when the pore pressures have come into equilibrium with the steady seepage flownet.

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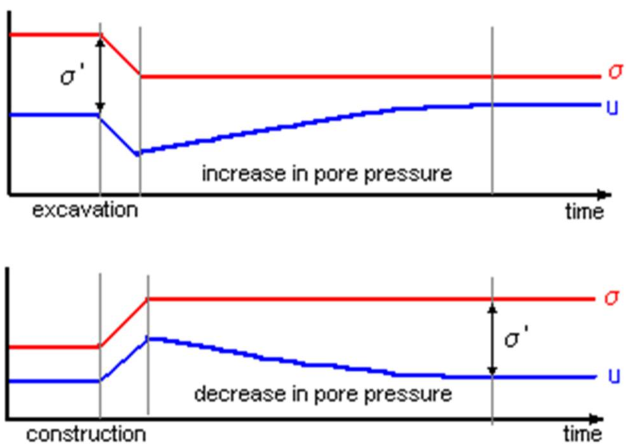
back to Stress changes in slopes

Comparison of cuttings with embankments



It is of interest to compare the behaviour of cutting slopes with that of embankments. In the case of embankments, the construction process leads to an increase in pore pressure in the foundation soil, which then dissipates with time. Thus one of the critical stability conditions is likely to be at the end of construction, and will involve failure through the foundation. The relationship between total stress, pore pressure and time for this condition is indicated in the figure. In the embankment material itself, since the fill is unsaturated, the initial pore pressures are negative, and the initial states at B and B', are similar to the cut slopes.

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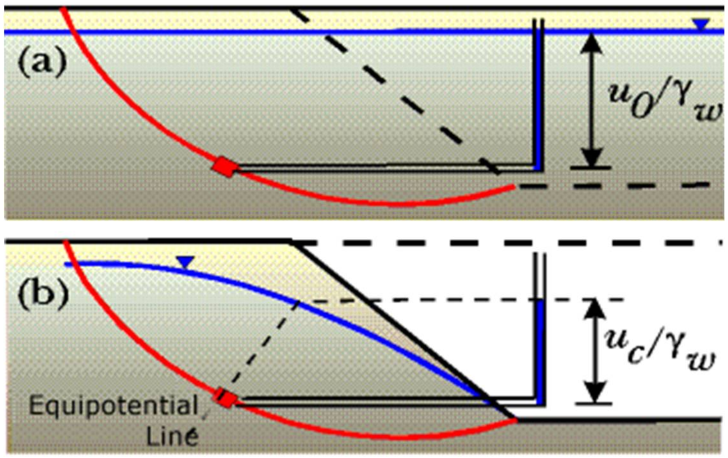
back to Stress changes in slopes

General case

The more general case is illustrated below.



Here the long-term pore pressures are controlled by the steady state flownet: the general principles, however, remain unchanged.

---

back to Mechanics

Choice of strength parameters

The choice between the undrained strength s_u and the drained strength is relatively simple and straightforward. For temporary slopes and cuts in fine-grained soils with low permeability the undrained strength s_u should be used and a total stress analysis performed. This analysis is only valid whilst the soil is undrained. The stability will deteriorate with time as the pore pressures increase and the soil swells and softens.

For any permanent slope the critical conditions are at the end of swelling when pore pressures have reached equilibrium with a steady state seepage flownet or with hydrostatic conditions. In this case an effective stress strength is appropriate and the pore pressures are calculated separately.

---

back to Mechanics

Choice of factor of safety

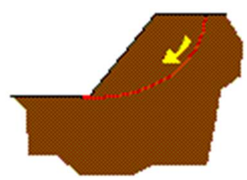
The factor of safety should take account of uncertainties in the determinations of the loads (including the unit weight), the soil strengths and particularly the pore pressure or drainage conditions and the consequences of failure. The greatest uncertainty is in the determination of steady state pore pressures in drained analyses or in the assumption of constant volume (and hence constant strength) in undrained analyses. There is no single value of F_s that can be recommended for slope stability calculations. Typical values are often in the range 1.25 to 1.35, but can fall outside this range. The table below gives a range of suggested methods of analyses and typical factors of safety.

	Typical F_s range
Temporary cuttings and embankments: using undrained strength s_u and total stresses	1.1 - 1.3
Permanent cuttings: using critical strength ϕ'_c and effective stresses	1.2 - 1.4
Embankment foundation: undrained s_u or drained ϕ'	1.2 - 1.5
Embankment fill: drained ϕ' for compacted soil and effective stresses	1.2 - 1.4
Reactivated landslip: residual strength ϕ'_r	(natural value)

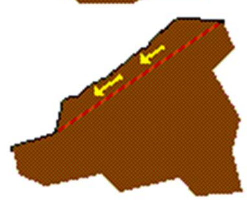
Slides

Movement involves shear displacement along one or more surfaces, or within a relatively narrow zone, which are visible or may reasonably be inferred. Two subgroups are identified as:

A. Rotational
Where movement results from forces that cause a turning moment about a point above the centre of gravity of the unit. The surface of rupture concaves upwards.



B. Translational
Where movement occurs predominantly along more or less planar or gently undulatory surfaces. Movement is frequently, structurally controlled by discontinuities and variations in shear strength between layers of bedded deposits, or by the contact between firm bedrock and overlying detritus.



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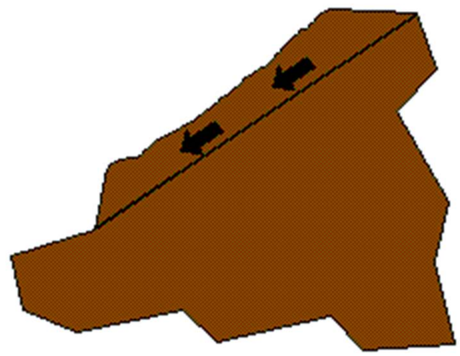
back to Slope stability

Analysis of translational slip

Translational or **infinite slope** movement predominantly occurs along more or less planar or gently undulatory surfaces. Displacement is frequently, structurally controlled by discontinuities and variations in shear strength between layers of bedded deposits, or by the contact between firm bedrock and overlying detritus.



See also the case studies:

- Shallow slab slide
- Quick clay slide

back to Analysis of translational slip

Drained soil with zero flow

In this case, the soil cohesion is zero, and the slope is dry or fully submerged, so no ground water seepage occurs to generate pore water pressures.
(In the general equation c' = 0 and m = 0.)

The Factor of Safety against slip reduces to:

$$F_s = \frac{\tan \phi'}{\tan \beta}$$

Note that the factor of safety is independent of the mass of the soil, the length of the slip and the depth of the slip surface. The limiting condition occurs when the slope angle (β) has the same magnitude as the angle of friction (ϕ').

back to Drained soil with zero flow

Derivation: Drained soil with zero flow

[Analysis of translational slip]

Drained soil with parallel flow

In this case, the soil cohesion is zero, and there is flow parallel to and coincident with the ground surface.
(In the general equation, c' = 0 and m = 1.)

The factor of safety against slip reduces to:

$$F_s = \frac{\gamma'}{\gamma_{sat}} \frac{\tan \phi'}{\tan \beta}$$

Note that the factor of safety is independent of the length of the slip and the depth of the slip surface. Because, in this case, the factor of safety is dependent upon the mass of the soil, the maximum slope angle (β_{max}) has a magnitude significantly lower than the angle of friction (ϕ').

> γ_s = effective unit weight of soil
> γ_s = unit weight of saturated soil

---

back to [Analysis of translational slip]

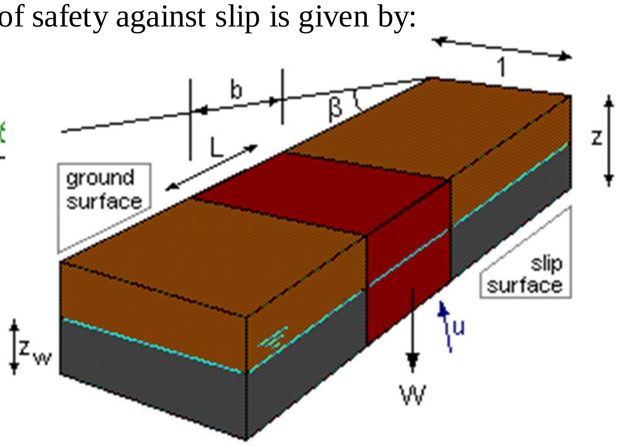
General equation

- [Derivation]

A translational slip analysis may be used for slip surfaces with small depth / length ratios.
This allows the end effects to be neglected.

For a potential slip surface in any soil, the factor of safety against slip is given by:

$$F_s = \frac{c' + [(1-m)\gamma + m\gamma_{sat} - m\gamma_w] z \cos^2 \beta}{[(1-m)\gamma + m\gamma_{sat}] z \sin \beta \cos \beta}$$



> 1 = a unit width of the slipping mass
> b = length of an element on plan (unit: m)
> L = slope length of an element (unit: m)
> m = ratio of z_w / z
> Having a range of values 0 to 1.
> Zero corresponds to a dry slope or one that is submerged with no seepage.
> u = pore water pressure due to seepage (unit: kN/m²)

W = vertical load due to the element (unit: kN)
z = vertical depth from the slope surface to the slip plane (unit: m)
z_w = vertical depth from the phreatic surface to the slip plane (unit: m)
γ_s = unit weight of saturated soil

back to General equation

Derivation of general equation

- Stresses along the slip surface
- Pore pressure

The factor of safety against slip is defined in terms of the ratio of this maximum shear strength to the disturbing shear stress:

$$F_s = \frac{\tau_{max}}{\tau}$$

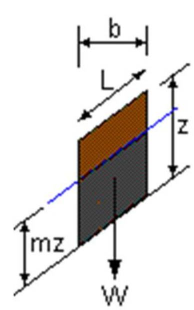
Ignoring any side forces acting on the elements, the stress conditions will be identical at every point along the slip surface. Therefore the maximum shear strength is given by the Mohr-Coulomb equation:

$$\tau_{max} = c' + (\sigma - u) \tan \phi'$$

and the stresses are determined by resolving the load due to an element.

back to Derivation of ordinary equation

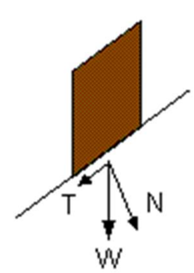
Stresses along the slip surface



Given that the soil is saturated below the phreatic surface,

$$W = [(1-m)\gamma + m\gamma_{sat}]zb$$

Resolving W into components parallel and perpendicular to the slip plane and converting to a stress, gives



$$T = W \sin\beta$$

$$\tau = \frac{T}{L} = \frac{W \sin\beta \cos\beta}{b}$$

$$N = W \cos\beta$$

$$\sigma = \frac{N}{L} = \frac{W \cos^2\beta}{b}$$

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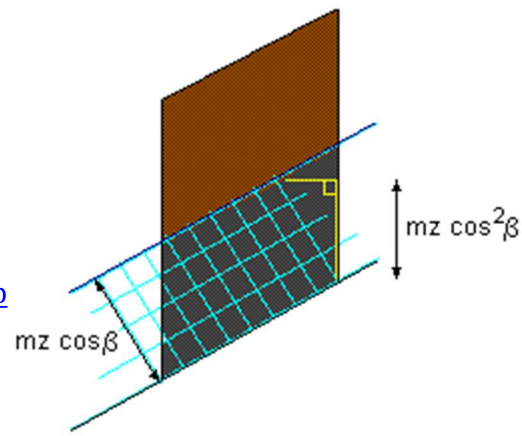
back to Derivation of General Equation

Pore pressure

From a consideration of the flownet, the pore water pressure at the slip surface is

$$u = mz\gamma_w \cos^2\beta$$



Analysis of translational slip

Stability of vertical cuts

- Derivation

It is impossible to make a vertical cut in a drained soil - this is easily demonstrated by the use of dry sand. In soils which are undrained, however, a vertical cut can be made since the negative pore pressures set up by the unloading due to the excavation will generate positive effective stresses.

If there is no tension crack present, the theoretical height of the cut is given by:
$H = (4\, s_u / \gamma)$
If a tension crack is anticipated, its theoretical value, h, is $(2s_u / \gamma)$, giving a maximum height of cut as
$H = (2s_u / \gamma)$

We should note that even if H is kept smaller than these theoretical values, local over stressing may occur near the base of the cut. As H increases towards the theoretical maximum value, the plastic zones extend, and significant deformations will take place.
γ = unit weight of soil

s_u = undrained strength of soil

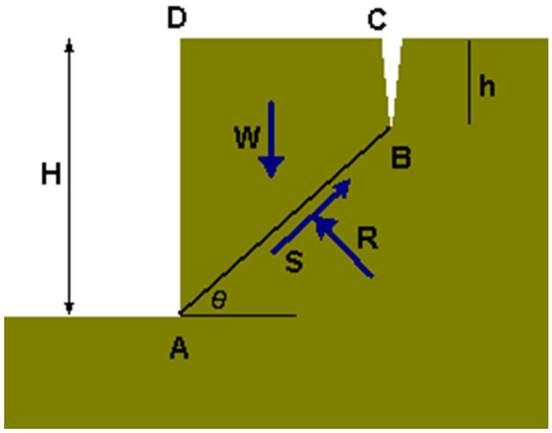
Stability of vertical cuts

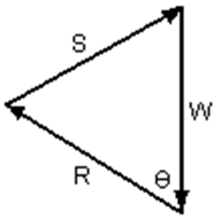
Derivation: stability of vertical cuts

Consider the vertical cut of height H shown in the figure. Assume that the soil is undrained and the strength can be represented by: $\tau = s_u$.

Consider the collapse mechanism shown, where failure occurs along the plane surface AB, inclined at θ to the horizontal. BC represents a vertical tension crack of depth h. The mass of soil represented by ABCD is in equilibrium under the action of 3 forces, namely:
W = weight of ABCD,
S = shear strength along BC,
R = normal reaction on BC.

From the triangle of forces
$\mathbf{W = R\cos\theta + S\sin\theta}$
$\mathbf{R\sin\theta = S\cos\theta}$
Now,
$\mathbf{W = 0.5\gamma(H - h)(H+h)\cot\theta}$
and,
$\mathbf{S = s_u.AB}$
where,
$\mathbf{AB = (H-h)\,cosec\theta}$
Eliminating R from these equations and substituting for W and S gives:
$\mathbf{H = (4s_u/\gamma) - h}$
If there is no tension crack, i.e. h = 0, then,
$\mathbf{H = (4s_u/\gamma)}$
The theoretical value of h is $\mathbf{(2s_u/\gamma)}$
and then H = $\mathbf{(2s_u/\gamma)}$





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back to Analysis of translational slip

Translational slip in rock slopes

- Plane failure
- Wedge failure

Translational slides in rock masses are dependent upon the spatial arrangement of the discontinuities within the mass and their relationship to the geometry of the slope. Two arrangements are considered:

Plane failure
In which slip is controlled by a single discontinuity, although others may exist as 'release surfaces'.

Wedge failure
In which slip occurs on two discontinuities and is governed by their line of intersection.

The engineer needs some means of graphically representing these discontinuities, if he or she is to be able to spot potential failure mechanisms. One graphical method uses stereonets to analyse the spatial arrangement of the planar discontinuities and slope surface.

The construction of stereonets is beyond the scope of this reference, good descriptions are available in

Goodman, R.E., "Introduction to Rock Mechanics",
2nd ed., Wiley, 1989, pp 417-434
Hoek, E. and Bray, J., "Rock Slope Engineering",
3rd ed., The Institution of Mining & Metallurgy, 1981
Priest, S.D., "Hemispherical Projection Methods in Rock Mechanics",
George Allen & Unwin, 1985

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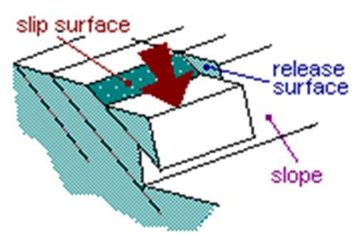
back to Translational slip in rock slopes

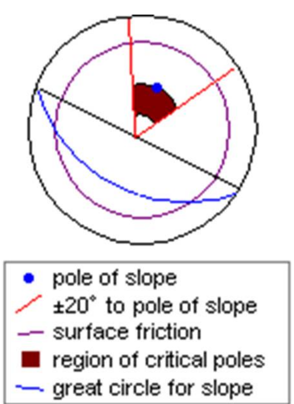
Plane Failure



Plane failure occurs due to sliding along a single discontinuity. The conditions for sliding are that:

· the strikes of both the sliding plane and the slope face lie parallel (±20°) to each other.
· the failure plane "daylights" on the slope face.
· the dip of the sliding plane is greater than φ'.
· the sliding mass is bound by release surfaces of negligible resistance.

Possible plane failure is suggested by a stereonet plot, if a pole concentration lies close to the pole of the slope surface and in the shaded area corresponding to the above rules.



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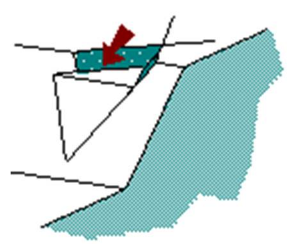
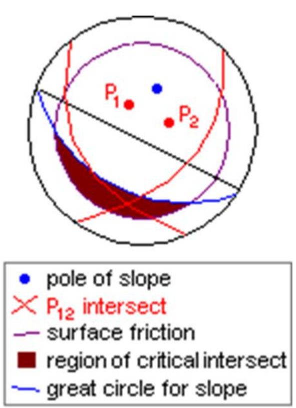
back to Translational slip in rock slopes

Wedge

Wedge failure occurs due to sliding along a combination of discontinuities. The conditions for sliding require that φ is overcome, and that the intersection of the discontinuities "daylights" on the slope surface.



On the stereonet plot these conditions are indicated by the intersection of two discontinuity great circles within the shaded crescent formed by the friction angle and the slope's



great circle. Note that this intersection can also be located by finding the pole P_{12} of the great circle which passes through the pole concentrations P_1 and P_2.

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back to Analysis of translational slip

Further reading: translational

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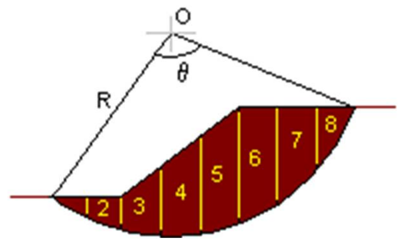
back to [Slope stability]

Analysis of rotational slip

- [Methods of analysis]
- [Ordinary Method of slices]
- [Bishop rigorous method]
- [Bishop simplified method]
- [Tension cracks]
- [Further reading: rotational]



The shear strength of the soil is a function of the normal stress s :

$\tau = c' + \sigma' \tan \phi'$

For this reason the method of analysis must take account of the changes in overburden pressure along the length of the slip circle. The procedure requires that a series of trial circles are chosen and analysed in the quest for the circle with the minimum factor of safety. Each circle is divided into vertical strips and the factor of safety is determined by considering the forces acting on each strip.

When specifying strips, care must be exercised to **avoid**:
(a) A dip angle (α_n) of zero magnitude, since this gives an infinite factor of safety for that slice.
(b) A steep dip angle (α_n) such that negative normal forces are calculated. This case corresponds to the formation of a tension crack and checks should be performed to ensure this condition does not exist.

See also the case studies:

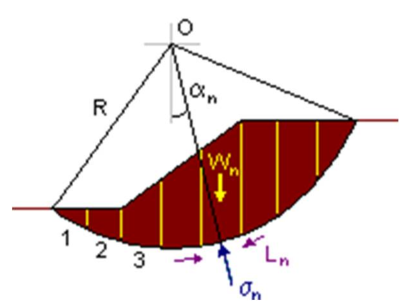
- [Circular slide]
- [Coastal landslide]
- [Movement along geological boundary]

back to [Analysis of rotational slip]

Ordinary Method of slices

- [Derivation of Ordinary Method]

This method is also refered to as "Fellenius' Method" and the "Swedish Circle Method". Consider the geometry of the trial slip circle shown in the diagram. The slipping mass is divided into



slices in the normal way and these are numbered 1, 2, 3, etc, for ease of identification. For a potential slip circle, the factor of safety against slip is given by:

$$F_s = \frac{\sum[c'L_n + (\cos\alpha_n - r_{un}\sec\alpha_n)\,W_n \tan\phi']}{\sum W_n \sin\alpha_n}$$

This solution tends to give a conservative value for the factor of safety, of between 5 and 20%. This can be expensive and therefore a more rigorous approach is favoured.

α_n = the positive or negative dip angle of the tangent line at the centre of the slice base (unit: degrees).

ϕ' = angle of friction in terms of effective stress (unit: degrees).

c' = cohesion in terms of effective stress (unit: kN/m²).

L_n = base length of a slice (unit: m).

r_{un} = pore pressure ratio for slice.

W_n = vertical load due to the slice (unit: kN)

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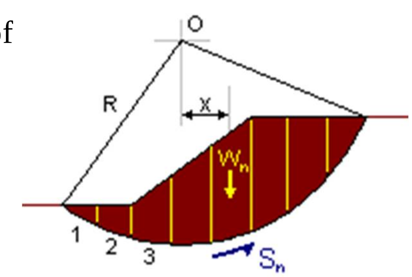
back to Ordinary method of slices

Derivation of Ordinary Method of slices

- Determining the shear foce for each slice

The factor of safety against rotational slip is defined as the ratio of the restraining moments to the disturbing moments:



$$F_s = \frac{\sum RS_n}{\sum W_n x_n}$$

Substituting for S_n gives,

$$F_s = \frac{R\sum\{c'L_n + (W_n\cos\alpha_n - u_n)\tan\phi' + [(X_{n+1} - X_n)\cos\alpha_n - (E_{n+1} - E_n)\sin\alpha_n]\tan\phi'\}}{\sum W_n x_n}$$

However, the magnitude and position of the side forces (X and E) are unknown and therefore the problem is statically indeterminate. Even so, this problem can be overcome by simply ignoring the effects of the side forces, i.e.

$$F_s = \frac{R\sum[c'L_n + (W_n\cos\alpha_n - u_nL_n)\tan\phi']}{\sum W_n x_n}$$

Then substitute for
$x_n = R\sin\alpha_n$ and
$u_n = (W_n r_{un}) / (L_n \cos\alpha_n)$
to give the ordinary equation.

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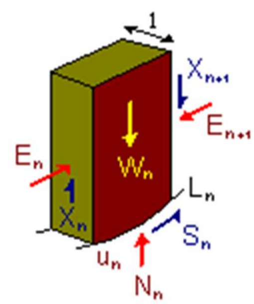
back to Derivation of ordinary method

Determining the shear force for each slice



From Coulomb's equation, the shear strength mobilised along the failure surface for slice n is:

$\tau_n = c' + (\sigma_n - u_n) \tan\phi'$

or, for a unit thickness:

$\tau_n = c' + [(N_n / L_n) - u_n] \tan\phi'$

Therefore the shear force acting along the base of the slice is:

$S_n = \tau_n .L_n$
$= L_n \{c' + [(N_n / L_n) - u_n] \tan\phi'\}$

By considering the forces acting on the slices and resolving normal to the slip surface, we can show that

$N_n = (W_n + X_{n+1} - X_n) \cos \alpha_n - (E_{n+1} - E_n) \sin \alpha_n$

giving

$S_n = c'L_n + (W_n .\cos \alpha_n - u_n).\tan\phi' + [(X_{n+1} - X_n).\cos \alpha_n - (E_{n+1} - E_n).\sin \alpha_n].\tan\phi'$

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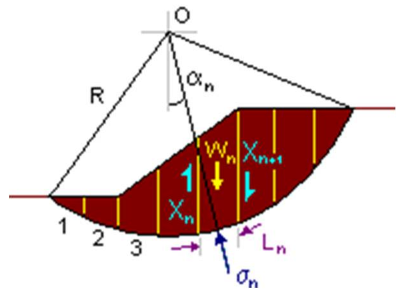
back to Analysis of rotational slip

Bishop rigorous method

- Derivation of Bishop rigorous equation

Bishop derived an expression which takes account of the interslice forces and gives a more accurate solution to the idealised geometry of circular slip. The importance of such forces can be demonstrated by considering a slice directly below the center of rotation: this slice has an independent safety factor of infinity since $\alpha = 0$ at that point. Bishop's solution requires that the factor of safety is constant along the complete slip circle. For a potential slip circle, the factor of safety against slip is given by:



$$F_s = \frac{\sum (c' L_n + V_n)}{\sum W_n \sin \alpha_n} \qquad \text{where}$$

$$V_n = \frac{\left[W_n + X_{n+1} - X_n - L_n \left(u_n \cos \alpha_n + \frac{c' \sin \alpha_n}{F_s} \right) \right] \tan \phi'}{\cos \alpha_n + \frac{\tan \phi' \sin \alpha_n}{F_s}}$$

or

$$F_s = \frac{\sum \left[\frac{Z_n \sec \alpha_n}{1 + \frac{\tan \phi' \tan \alpha_n}{F_s}} \right]}{\sum W_n \sin \alpha_n} \qquad \text{where}$$

$$Z_n = c' b_n + \left(W_n (1 - r_{un}) + (X_{n+1} - X_n) \right) \tan \phi'$$

However, the determination of the interslice forces is labourious and therefore Bishop's simplified method is prefered.

α_n = the dip angle of the tangent line at the centre of the slice base (unit: degrees).

ϕ' = angle of friction in terms of effective stress (unit: degrees).

c' = cohesion in terms of effective stress (unit: kN/m^2).

L_n = base length of a slice (unit: m).

r_{un} = pore pressure ratio for slice.

W_n = vertical load due to the slice (unit: kN).

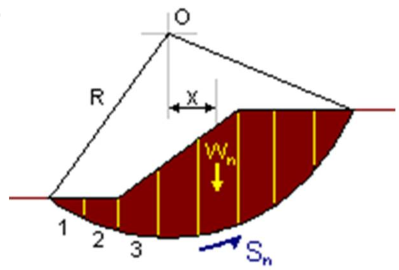
X_n = vertical interslice force (unit: kN).

u_n = pore pressure at base of slice (unit: kN/m^2).

back to Bishop rigorous method

Derivation of Bishop rigorous method

- Determining the shear force for each slice



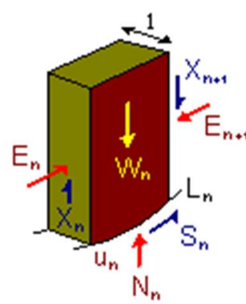
The factor of safety against rotational slip is defined as the ratio of the restraining moments to the disturbing moments:

$$F_s = \frac{\Sigma R S_n}{\Sigma W_n x_n}$$

Substituting for S_n and x_n gives the rigorous solution.

back to Derivation of bishop rigorous method

Determining the shear force for each slice



From Coulomb's equation, the shear strength mobilised along the failure surface for slice n is:

$\tau_n = c' + (\sigma_n - u_n) \tan\phi'$

therefore the shear force acting along the base of a slice of unit thickness is:

$S_n = \tau_n .L_n$
$= c'.L_n + N'_n .\tan\phi'$

Resolving the forces on the slice vertically gives:

$W_n + X_n+_1 - X_n = (N'_n + u_n L_n) \cos \alpha_n + S_n \sin \alpha_n$

But this is true for $S_n = c' L_n + N'_n \tan\phi'$ at the limiting state only. Therefore, the factor of safety is introduced into this expression for all other states (S_n / F_s) giving:

$W_n + X_n+_1 - X_n = (N'_n + u_n L_n) \cos \alpha_n + ((c'L_n /F_s) + (N'_n /F_s) \tan\phi') \sin \alpha_n$

giving,

$$N_n' = \frac{W_n + X_{n+1} - X_n - L_n\left(u_n \cos\alpha_n + \dfrac{c' \sin\alpha_n}{F_s}\right)}{\cos\alpha_n + \dfrac{\tan\phi' \sin\alpha_n}{F_s}}$$

and thus,

$$S_n = c'L_n + \tan\phi' \left[\frac{W_n + X_{n+1} - X_n - L_n\left(u_n \cos\alpha_n + \dfrac{c' \sin\alpha_n}{F_s}\right)}{\cos\alpha_n + \dfrac{\tan\phi' \sin\alpha_n}{F_s}}\right]$$

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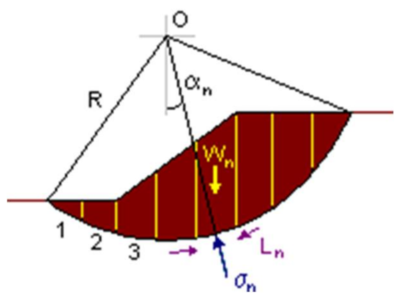
back to Analysis of rotational slip

Bishop simplified method

- Solving the Bishop simplified equation
- Short term analysis for an undrained soil

Although the rigorous method is more "accurate" it is a time consuming process and can be easily simplified by ignoring the X terms. The errors associated with such a simplification have been shown to be small.



$$F_s = \frac{\sum\left[\dfrac{(c'b_n + W_n(1-r_{un})\tan\phi')\sec\alpha_n}{1 + \dfrac{\tan\phi' \tan\alpha_n}{F_s}}\right]}{\sum W_n \sin\alpha_n}$$

The values for r_{un} vary from slice to slice but, unless zones of high pore pressure exist, an average value weighted according to area can be used throughout. Like the Ordinary Method the factor of safety calculated is conservative. The value is underestimated by about 2%, but can be as large as 7%. The nature of the equation, makes solution using a computer programme or spreadsheet desirable.

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back to Bishop simplified method

Solving the Bishop simplified equation

- Example of table calculations

$$F_s = \frac{\sum \left[\dfrac{(c' b_n + W_n (1 - r_{un}) \tan\phi') \sec\alpha_n}{1 + \dfrac{\tan\phi' \tan\alpha_n}{F_s}} \right]}{\sum W_n \sin\alpha_n}$$

Note that α_n may have both positive and negative values.

In order to solve this equation, which has F_s on both sides, the right hand value is first estimated using the Ordinary Method and then the left hand value is calculated. This new value is then used and the procedure repeated until the two values converge.

Create a table of calculations for each slice. Note that this calculates the factor of safety for this circle, not the slope. There may be another circle or failure mechanism with a lower factor of safety.

Solving the Bishop simplified equation

Example of table calculations

		1	2	3	...	
n						
	(m^2)	...	...	...	...	
area						
	(kN)	...	...	...	...	
W_n						
	(deg)	...	...	...	...	
a_n						
$\sin\alpha_n$		...	...	...	...	
$W_n \sin\alpha_n$	(kN)	...	...	...	...	S(1)
b_nBISHOPSIMPLEQT_B	(m)	...	...	...	...	
	(kN)	...	...	...	...	

exp(1)

... ...

exp(2)

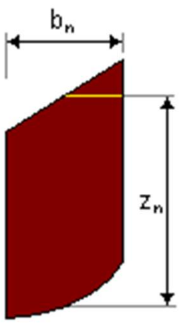
exp(1)/exp(2) (kN) ... ... S(2)

$F_s = \Sigma(2) / \Sigma(1)$

n = number of the slice under consideration

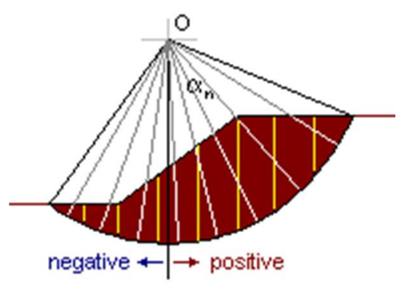
$\mathbf{b}_n$ = the plan width of the slice

area = $b_n .z_n$ = approximated area of slice



$\mathbf{W}_n$ = weight of the slice for a unit thickness, calculated by multiplying the slice area by the unit weight of the soil

$\boldsymbol{\alpha}_n$ = the dip of the slip circle at the centre of the base of the slice, most easily obtained by drawing a line from the centre of rotation to the centre of the slice at the slip circle and measuring the angle it makes with the vertical (remember its sign!)



exp(1) = $c'b_n + W_n(1-r_{un})\tan\phi'$

exp(2) = $[1+ (\tan\phi' \tan \alpha_n) / F_s] / \sec \alpha_n$

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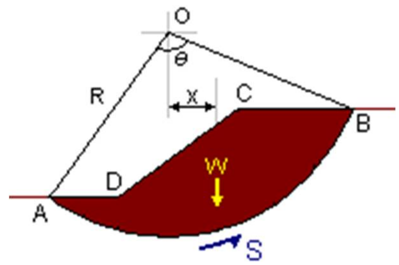
back to Bishop simplified method

Short term analysis for an undrained soil

- Locating the centre of the slip circle



This method is commonly referred to as the "Total Stress Analysis". If we consider the case of a cohesive soil and analyse its stability in the short term, we can substitute the total stress soil parameters into the Bishop simplified equation. Such that,

c' Þ s_u
ϕ' Þ $\phi_u = 0$

and the equation reduces to

$$F_s = \frac{\sum s_u \sec \alpha_n}{\sum W_n \sin \alpha_n}$$

substituting for $L_n = b_n \sec\alpha_n$ and $\sin \alpha_n = x_n / R$, gives

$$F_s = \frac{s_u R \sum L_n}{\sum W_n x_n}$$

Substitute for $\Sigma\Lambda_n = R\theta$, where θ is in radians. Then because the resistance to slip is constant along the slip circle, the slices are redundant and the slip can be treated as one mass (ABCD), giving

$$F_s = \frac{s_u R^2 \theta}{W x}$$

Where W acts through the centre of gravity of the slipping mass.

s_u = cohesion in terms of total stress (units: kN/m²).

R = radius of the slip circle (unit: m).

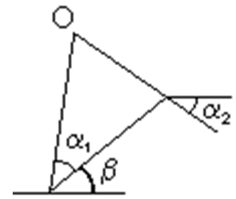
θ = angle subtended by the slip circle (unit: radians).

W = vertical load of the slipping mass (unit: kN).

x = moment arm of the slipping mass (unit: m).

Short term analysis for an undrained soil

Locating the centre of the slip circle



Choosing the centre for the circle can be daunting at first! In 1936, Fellenius proposed the following method for locating the centre of a circle passing through the toe of the slope:

slope	**b°**	**a_1 °**	**a_2 °**
1:58	60	29	40
1:1	45	28	37
1:1.5	34	26	35
1:2	27	25	35
1:3	18	25	35
1:5	11	25	37

For deeper circles, the centre of rotation is generally vertically above the mid-point of the slope.

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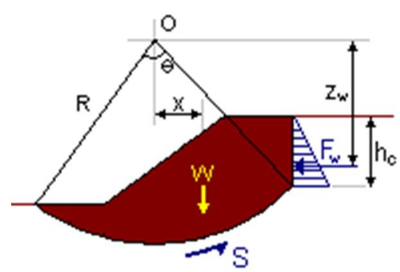
back to Analysis of rotational slip

Tension cracks



In undrained conditions, a tension crack may develop at the top of the slope and hence no shear strength can occur over that length. The angle θ must be reduced accordingly (see diagram). Furthermore, water in the crack will supply an additional hydrostatic force, acting to reduce the factor of safety. This can be incorporated into the analysis by treating it as an additional disturbing moment,
$= F_w z_w$.

The depth of the tension crack is given by:

$$h_c = \frac{2s_u}{\gamma}$$

ϕ_u = angle of friction in terms of total stress [undrained] (unit: degrees).

s_u = cohesion in terms of total stress [undrained] (unit: kN/m²).

h_c = depth of tension crack (unit: m).

γ = unit weight of soil (unit: kN/m³).

F_w = resultant force for water pressure in tension crack acting at $^2/_3$ depth of water (unit: kN).

z_w = moment arm of resultant force F_w (unit: m).

Analysis of rotational slip

Methods of analysis

- Limit equilibrium procedure

In practice, limit equilibrium methods of analysis are generally adopted, in which it is considered that failure is on the point of occurring along an assumed or a known failure surface. The shear strength required to maintain a condition of limiting equilibrium is compared with the available shear strength of the soil, giving the average factor of safety along the failure surface.

The problem is normally considered in two dimensions, with the conditions of plane strain being assumed, but a rule of thumb states that the factor of safety in 3-D is 10% greater. This is usually ignored, giving an additional safety margin.

The limit equilibrium method can be used in cases of undrained or drained loading, provided that the appropriate shear strength parameters are used. It is important to note that these strengths define the ultimate collapse states. In order to design safe structures or to limit ground movements, they may be reduced.

In using the limit equilibrium method, the geometry of the assumed slip surfaces must form a mechanism that will allow collapse to occur, but since they may be of any shape, they do not necessarily meet all the conditions of compatibility. In addition, the overall conditions of equilibrium of forces on blocks within the mechanism must be satisfied although the local states of stress within the blocks are not investigated.

For undrained loading, the ultimate strength of the soil is given by
$\tau = s_u$ where s_u is the undrained shear strength.

For drained loading, where pore pressures can be determined from hydrostatic groundwater conditions or from a steady seepage flownet, the strength is given by:

$\tau = \sigma' \tan \phi' = (\sigma - u) \tan \phi'$
where ϕ' is the appropriate angle of shearing resistance.

[Methods of analysis]

Limit equilibrium method procedure

The steps in calculating a limit equilibrium solution are as follows:

1. Draw an arbitrary collapse mechanism of slip surfaces. This mechanism may consist of a combination of straight lines or curves.

2. Determine the static equilibrium of the mechanism by resolving forces or moments and hence calculate the strength mobilised in the soil.

3. Compare the strength mobilised with the available shear strength of the soil, and hence define an average value for the factor of safety for the mechanism considered.

4. Repeat the procedure for other mechanisms and thus find the critical mechanism which defines the minimum value for the factor of safety.

Site investigation

In relation to slope stability, the main aims of site investigation are:

to obtain an understanding of the development and nature of natural slopes, and of the processes which have contributed to the formation of different natural features;
to assess the stability of various forms of slopes under given conditions;
to assess the risk of instability in natural or artificial slopes, and to quantify the influence of engineering works or other modifications to the stability of an existing slope;
to facilitate the redesign of failed slopes, and the planning and design of prevention and remedial measures;
to analyse slope failures which have occurred and to define the causes of failure;
to assess the risk of special external factors on the stability of slopes, e.g. earthquakes.

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back to Site investigation

Organisation of site investigation

Any site or ground investigation is performed under the constraints of time, money and the complexity and variability of the geological environment. It is therefore often necessary to reach a compromise between the precise details at a site and the time and money available. The investigation must be tackled in a logical and scientific manner. In addition, site investigation is a skilled operation, and must be entrusted only to suitably trained operators.

Site investigations can be considered under 3 main headings:
desk study,
field study,
laboratory work.
Here the discussion will be concentrated upon topics which are unique or have special importance in the context of slope stability.

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back to Organisation of site investigation

Desk study

The aim here is to obtain all available information with regard to the site and its geological environments. It will involve a search through records, maps, (topographical and geological), and any other information which is relevant to the geology, history and present condition of the site. A useful list of sources of information and the procedure to be followed in carrying out the desk study has been given by Dumbleton and West.

It is helpful at this stage to attempt a preliminary analysis of the geology by preparing sections etc. - this exercise may help to define where further information is required. A visit to the site must also be made to confirm observations and predictions already made.

Dumbleton, NLJ., and West, G. (1971). Preliminary Sources of Information for Site Investigations in Britain. RRL Report No. LR403, Transport and Road Research Laboratory, Crowthorne, Berks.

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back to Organisation of site investigation

Field study

- Geomorphological mapping
- Trial pits

The basic aims of the field study are to record accurately the topography of the site, to determine the precise nature of the geological deposits underlying the site and to determine their engineering properties, either by the collection of good quality samples which can be tested subsequently in the laboratory, or by performing tests in-situ.

Conventional surveying techniques may provide sufficient information to permit cross-sections etc. to be prepared. In some cases, useful input can also be obtained from air-photos.

In the context of mass movements on slopes, however, a recent development has been the introduction and application of geomorphological mapping. The employment of such techniques has been shown to usefully precede and supplement standard geotechnical and geological investigations.

In civil engineering it is quite usual in the case of light structures to limit the subsurface investigation to only a few relatively shallow trial pits. It should, however, be recognised that trial pits have an important function in investigations for other, more important structures and, particularly, on sites where landsliding and other forms of mass movement have already taken place or may be expected to occur in the future.

An essential part of any investigation concerns the groundwater conditions pertaining on the site, and the accurate measurement of water pressures in the ground. It must be stressed that great care is needed in order to obtain reliable data on this topic, which is of comparable importance in assessing the stability of a slope as the determination of the shear strength properties.

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back to Field study

Geomorphological mapping

The mapping is performed using the techniques described by Waters and Savigear. It is based on identifying breaks and changes of slope, and the resulting delimitation of slope units. The direction and value of maximum slope, when measured across each unit, is recorded. In the case of large units or in areas characterised by complex forms, the number of slope measurements per unit is increased. An example of the detail and impression of topographical form which can be obtained using this technique can be seen **here**. If necessary, a clearer visual impression of the topography can be obtained by using the slope information to prepare a slope category map, as described by Brunsden and Jones.

Further descriptions of the development of geomorphological mapping and its application in engineering projects have been given by Brunsden et al.

Waters, R.S. (1958). Morphological mapping. Geography, 10-17.

Savigear, R.A.G. (1965). A technique of morphological mapping. Mapping Assoc. Amer. Geogr., 55, 514-38

Brunsden, D., and Jones, D.K.C. (1972). The morphology of degraded landslide slopes in South West Dorset. Quart. J. Eng. Geol., 5, 205-222.

Brunsden, D., Doornkamp, J.C., Fookes, P.G., Jones, D.K.C., and Kelly, J.M.H. (1975). Large scale geomorphological mapping and highway engineering design. Quart. J. Eng., Geol. 8,227-254.

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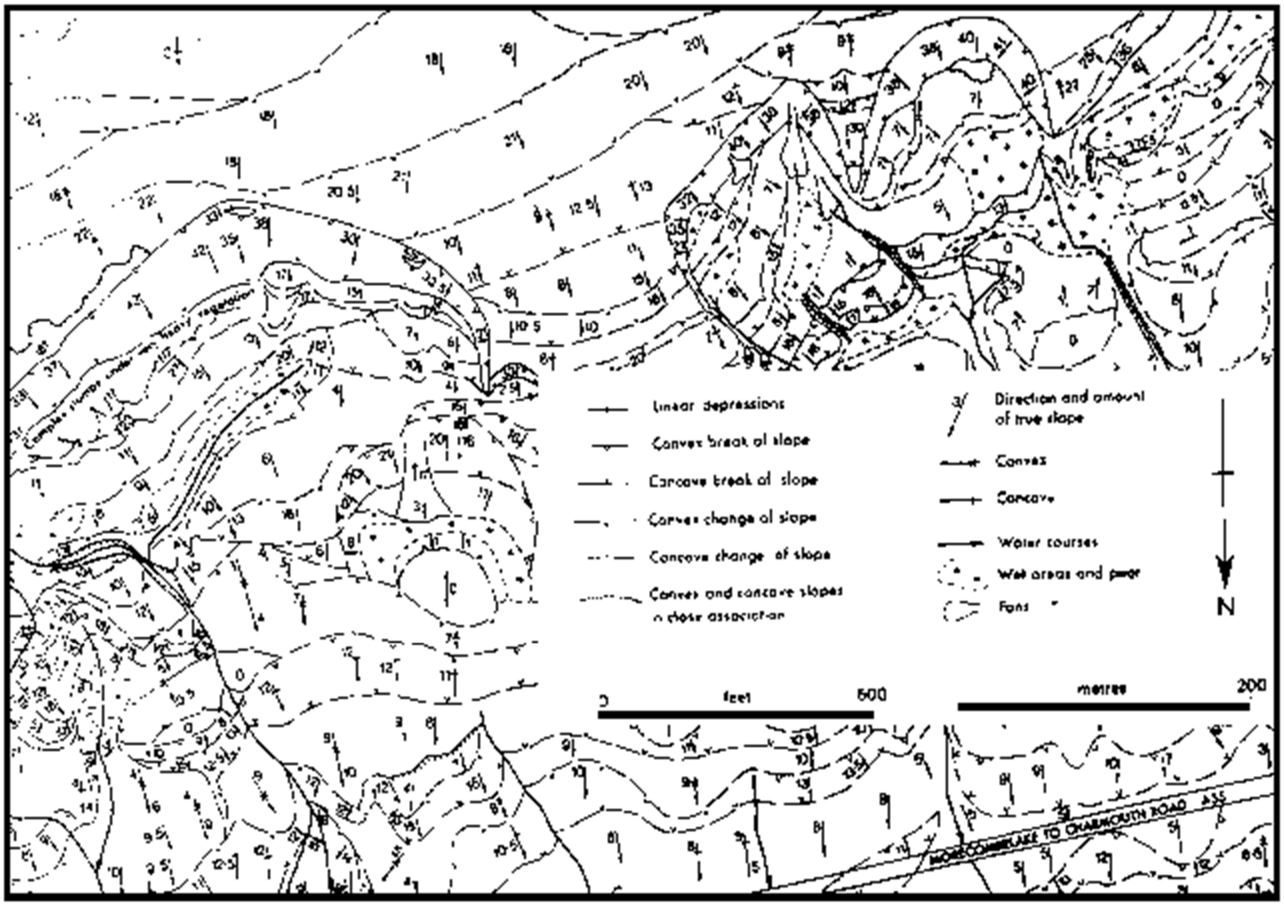


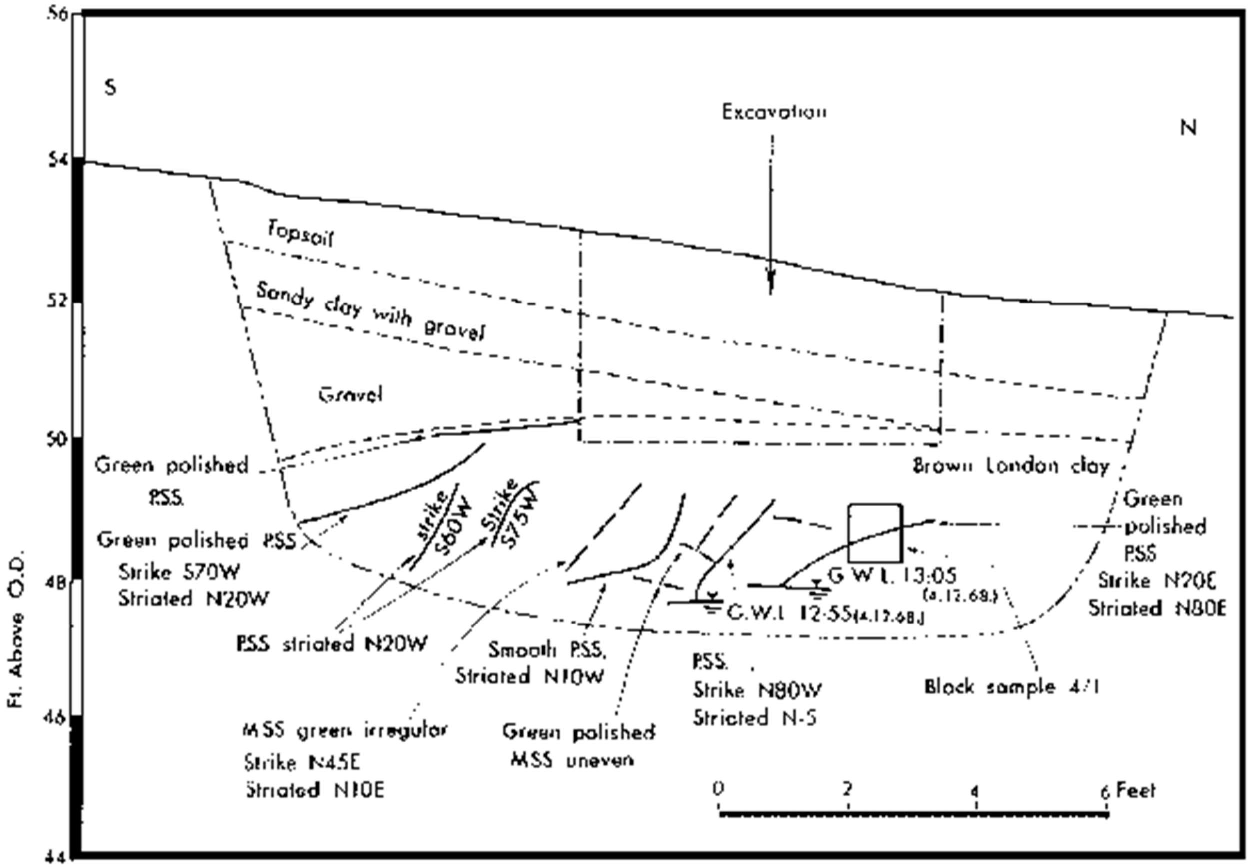
A typical geomorphological map of a landslide area (after Brunsden and Jones 1972)

back to Field study

Trial pits

Mobile rubber-tyred excavators can be used to excavate trial pits to depths of about 4-5m, and they are economical since hiring can be made on a time basis. Deeper trial pits may require the use of tracked excavators, which are more expensive since they have to be transported to and from the site by low-loader. Great care must be taken when using trial pits to avoid the risks associated with collapse of the sides of the pits. As a general rule, the spoil from the pit must be placed well clear of the top of the pit, and adequate bracing used to provide stability for the sides. deep trial pits with full timbering have been used for specialist purposes (see, for example, Hutchinson et al.), but this is an expensive operation. It has been found to be economical to use a rotary power auger in order to sink deep inspection shafts in soils or soft rocks, but these shafts generally need to be not smaller than 1 m diameter if the strata are to be examined or tests conducted in situ.

The major benefit derived from the use of trial pits, compared with boreholes, is that they permit a physical examination of the soils en masse to be carried out in their natural habitat. It is then possible to establish the degree of variation which may be found in a particular soil and it is also possible to search for discontinuities which are frequently damaged or disturbed during a sampling operation in a conventional borehole. An example of the information which can be found in a trial pit is shown **here**. Judicious positioning of a series of trial pits, or the excavation of trenches, are of great use in the investigation of mass-movement processes, particularly since it is possible to locate the surfaces along which movements have occurred. Block samples can be taken to include these shell surfaces and appropriate tests, performed in the laboratory, will detertmne the shear strength along them. An accurate stability analysis can then be performed.



A typical trial pit profile

Hutchinson, J.N., Somerville, S., and Petley, D.J. (1973). A landslide in periglacially disturbed Etruria Marl at Bury Hill, Staffordshire. Quart. J. Eng. Geol., 6, 377-404.

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back to Organisation of site investigation

Laboratory work

- Measurement of shearing resistance

The object of performing laboratory tests is to obtain information, additional to that obtained from *in situ* tests, on the composition and properties of the materials encountered on any site. Laboratory tests can be grouped under three main headings:

tests for **classification** and identification;
tests for **engineering** properties;
tests for **special** purposes in engineering construction.

The first group include tests to determine the particle-size distribution of the material, index property tests (Liquid and Plastic Limits), specific gravity tests, and tests to determine the bulk density and water content of the soils. Since these are very common tests they will not be discussed further here (see British Standard 1377).

The second group of tests includes those to determine the engineering properties of the soils, i.e. permeability, compressibility and shear strength.

In the third group are special tests devised for earthworks and roads and airfields. They have little relevance for mass-movement studies.

BS 1377 (1990). Methods of Testing Soils for Civil Engineering Purposes, British Standards Institution, London.

Measurement of shearing resistance

The accurate measurement of the shearing resistance or shear strength of a material is essential in attempting to predict future instability or to assess the present or past stability condition. As stated previously, shear strength tests must be performed on samples of the highest quality if reliable information is to be obtained. Even when this condition is satisfied, however, there may still be cases where the shear strength measured in the laboratory differs from that mobilised *in situ*.

Laboratory tests can broadly be divided into two types, depending primarily on the pore pressures set up within the sample during the test and whether dissipation of these pore pressures is prevented or permitted. Tests can therefore be categorised as either 'undrained' or 'drained'. In undrained tests, the pore pressures set up during the test are not permitted to dissipate, and the test may be performed relatively quickly. The existence of these pore pressures - which may or may not be monitored - influences the behaviour of the soil to a marked extent. It is generally considered that the results obtained from undrained tests are applicable to short-term stability conditions. In drained tests adequate time is allowed for the dissipation of pore pressures, so tests are much longer than most undrained tests. The results of these tests can be used to assess the long-term stability in slopes and cuttings.

Shear strength properties of soils are defined by two parameters, apparent cohesion *c* and the angle of shearing resistance f . In undrained tests the parameters are expressed in terms of total stresses, whereas in drained tests the parameters are denoted by c' and ϕ'. A summary of problems which can be analysed in terms of total or effective stresses has been given by Bishop and Henkel.

Bishop, A.W., and Henkel, D.J. (1962). The Measurement of Soil Properties in the Triaxial Test, Edward Arnold, London.

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back to Measurement of shearing resistance

Shear box tests

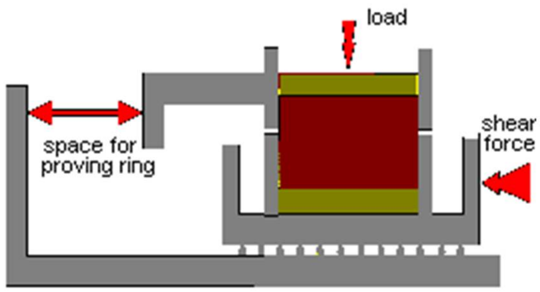
The shear box was probably the first type of apparatus used for the measurement of the shearing resistance of soils. The apparatus, which is shown in the figure, consists essentially of a square brass box split horizontally at the level of the centre of the soil specimen which is held between metal grills and porous stones. The horizontal force acting on the upper part of the box is gradually increased until the specimen fails in shear. The shear force at failure s_f is divided by the cross-sectional area *A* to give the shearing stress t_f at failure. The vertical stress σ_n is provided by a vertical load on the sample, normally by dead-weights and a lever system. The horizontal load is applied by pushing the lower part of the box by means of an electric motor and gearbox. Volume changes are monitored by a dial gauge mounted to show the vertical movement of the top loading platen.

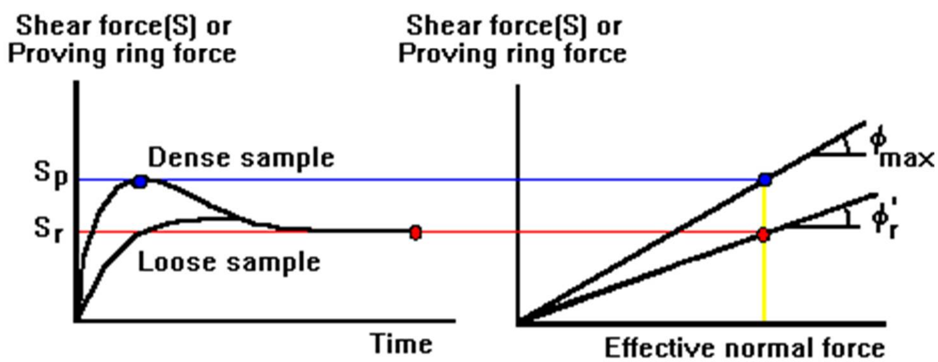
The size of the shear box normally used for tests on fine-grained soils is 60 mm square, and the sample is approximately 20 mm thick. For soils containing gravel, a shear box 300 mm square is frequently used; in dealing with some soils even larger specimens may be required since, as a rough rule, the maximum particle tested should not exceed one-eighth of the length of the shear box.

Tests in the shear box are relatively simple to perform, but the test is open to a number of criticisms. The most important of these are:

- it may be difficult to install an undisturbed sample in the apparatus;
- the stress distribution across the sample is complex;
- failure occurs along a plane dictated by the design of the apparatus;
- the area under shear reduces during the test;
- there is no direct control over drainage conditions in the sample.

Typical results from tests on well-graded sand are illustrated here.





Results of shear box tests on sand

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back to Measurement of shearing resistance

Triaxial tests

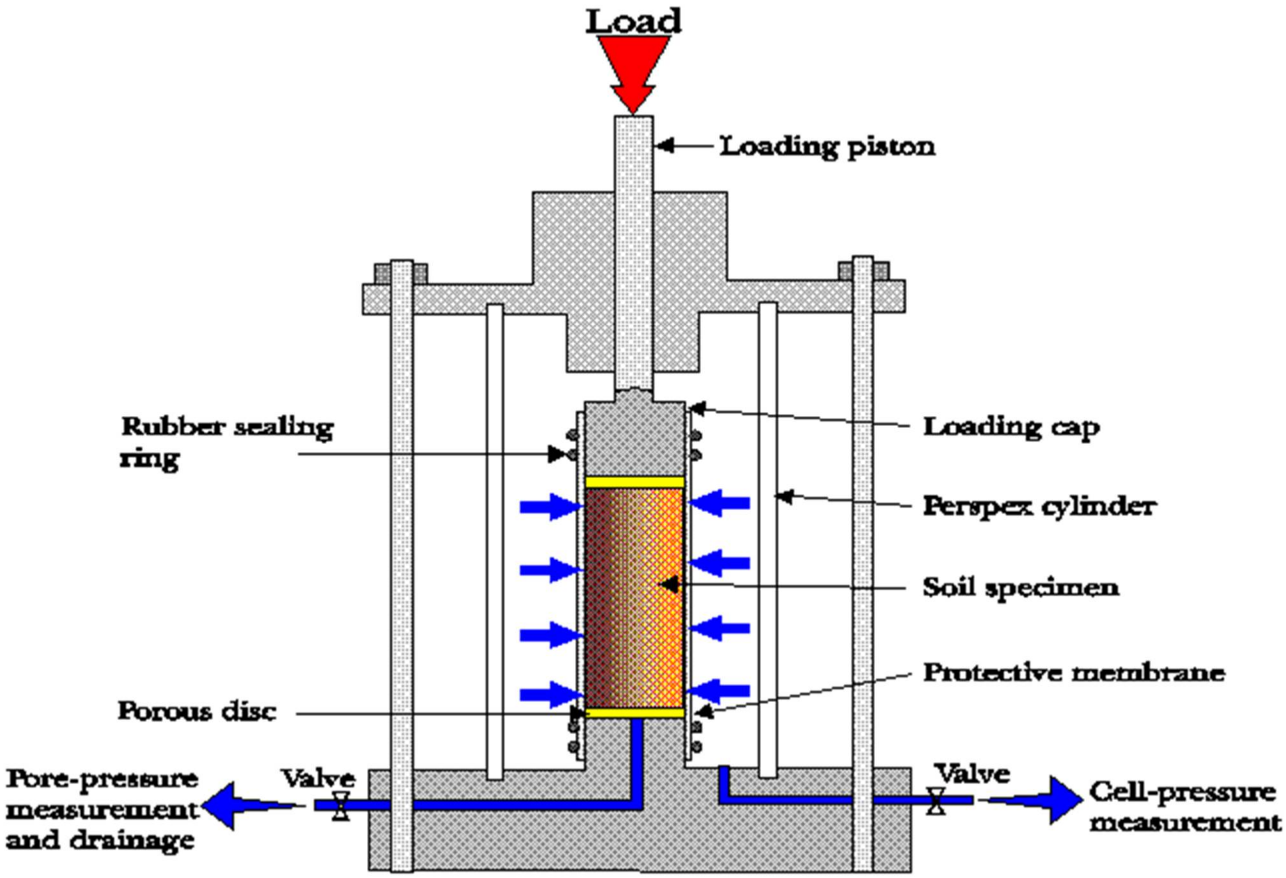
The triaxial compression test is the most widely used technique to determine the shear strength of soils. The apparatus is shown diagramatically in the figure. The sample, which is cylindrical, is tested inside a perspex cylinder filled with water under pressure. The sample under test is enclosed in a thin rubber membrane to seal it from the surrounding water. The pressure in the cell is raised to the desired value, and the sample is then brought to failure by applying an additional vertical stress.

One of the major advantages of the triaxial apparatus is the control provided over drainage from the sample. When no drainage is required (i.e. in undrained tests), solid end caps are used. When drainage is required, the end caps are provided with porous plates and drainage channels. It is also possible to monitor pore-water pressures during a test. Full details of the basic apparatus and refinements, and procedures for a wide range of tests in the triaxial apparatus, are given by Bishop and Henkel.

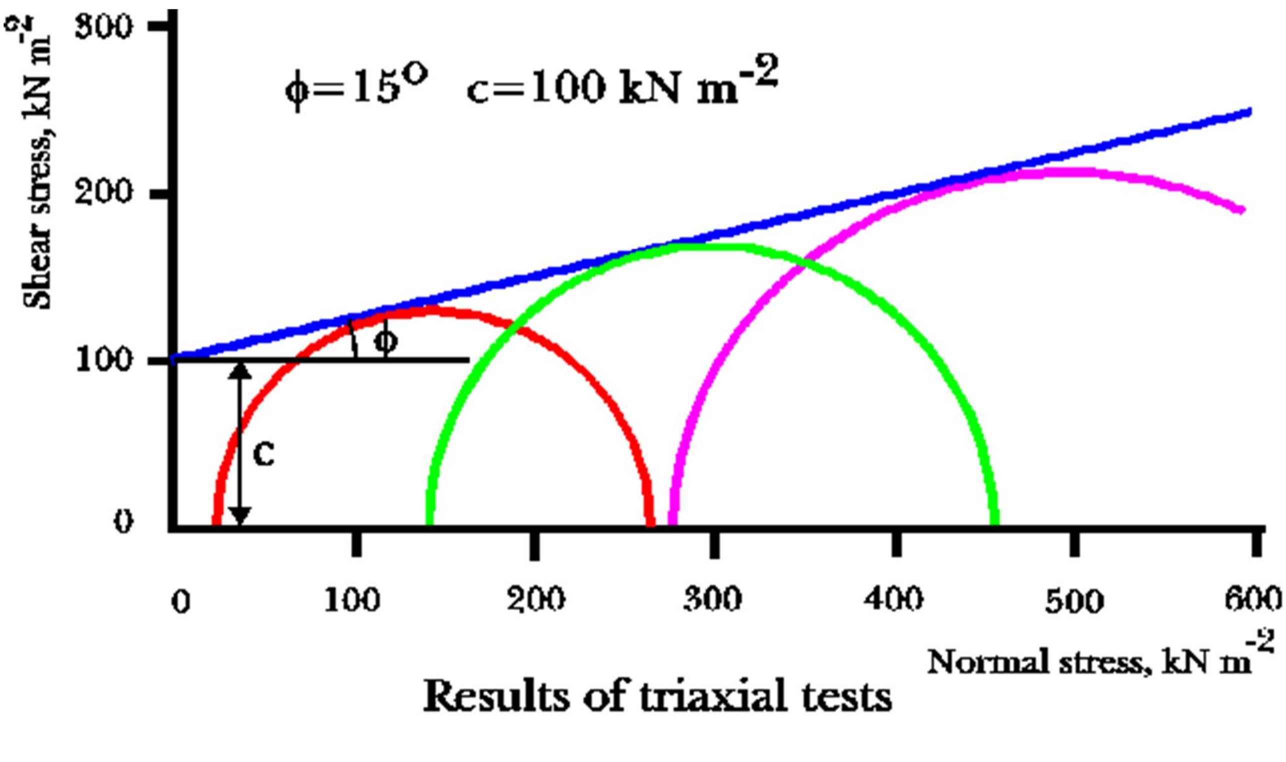
For cohesive soils, the size of sample normally used in the triaxial apparatus is 38 mm diameter and 76 mm long. When gravel is present, for example in boulder clay, larger samples may be used, the most common being 100 mm diameter and 200 mm long. For coarse gravelly soils, rockfill and artificially prepared granular material such as railway ballast, even larger samples are required if realistic values of the shearing strength are to be obtained. This is also true for fissured cohesive soils, where the sample tested must be of sufficient size to contain a truly representative collection of all the structural features which may affect the shear strength.

To obtain the shear strength parameters of the soil, a number of specimens (normally at least three) are tested at different values of cell pressure. For each test, the vertical stress s_3 at failure are determined and are used to plot a Mohr circle. The envelope to these circles then defines the shear strength parameters.

It is important that the values of the shear strength parameters c' and ϕ' are obtained from the Mohr's circles obtained by tests on similar material. In markedly heterogeneous materials, it may be difficult to obtain sufficient samples for testing, and the technique of 'multi-stage' testing may be employed. This form of test is normally perforated on 100 mm diameter samples. The sample is initially tested at a particular cell pressure and the vertical stress is increased until failure is approached. At this point the cell pressure is increased, and shearing resumes until failure is again approached under the new cell pressure. The process is repeated a number of times. There has been some criticism of this type of test, but it does appear to give reasonably acceptable results if the test is performed with care.



Triaxial apparatus



Results of triaxial tests

back to [Measurement of shearing resistance]

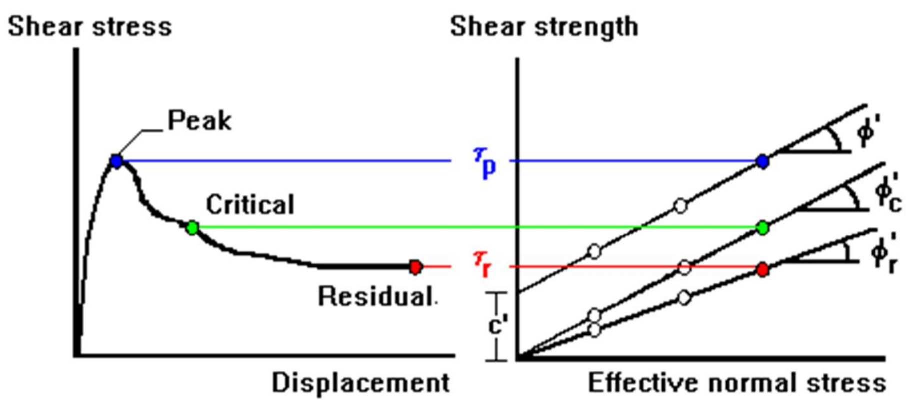
Measurement of residual shear strength

- [Residual shear strength]
- [Methods of measurement of residual strength]

When a soil is subjected to shear, an increasing resistance is built up. For any given applied effective pressure, there is a limit to the resistance that the soil can offer, which is known as the *peak shear strength* s_p. Frequently the test is stopped immediately after the peak strength has been clearly defined. The value s_p has been referred to, in the past, as simply the *shear strength* of the clay, under the given effective pressure and under drained conditions.

If the shearing is continued beyond the point where the maximum value of the shear strength has been mobilised it is found that the resistance of the clay decreases, until ultimately a steady value is reached, and this constant minimum value is known as the *residual strength* s_r of the soil. The soil maintains this steady value even when subjected to very large displacements.

Typical results for a drained test on clay, taken to displacements large enough to mobilise the residual strength, are shown below.



Shear strength tests

Further tests could be made on the same clay but under differing effective pressures. The results previously described would again be obtained, and from a number of tests it would be noticed that the peak and residual shear strengths would define envelopes in accordance with the Coulomb-Terzaghi relationship, a. Thus the peak strengths can be expressed as:
$s_p = c' + \sigma' \tan \phi'$

and the residual strengths can be expressed as:

$s_r = c_r' + \sigma' \tan \phi_r'$

The decrease in shear strength from the peak to the residual condition is associated with orientation of the clay particles along shear planes.

c' = apparent cohesion
c'_r = residual apparent cohesion
ϕ' = angle of shearing resistance
ϕ'_r = residual angle of shearing resistance
s_p = peak shear strength
s_r = residual shear strength
σ' = applied effective stress

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back to Measurement of residual shear strength

Residual shear strength

The residual shear strength condition is of considerable practical importance since, if the soil *in situ* already contains slip planes or shear surfaces, then the strength operable on these surfaces will be less than the peak strength, and if sufficient displacement has taken place, the strength may be as low as the residual strength.

There are a number of circumstances, as a result of which shearing of the soil may have taken place, and the principal processes, summarised by Morgenstern et al , are:

- landsliding,
- tectonic folding,
- valley rebound,
- glacial shove,
- periglacial phenomena, and
- non-uniform swelling.

The identification of the existence of shear surfaces is a problem of great importance during any site investigation, particularly where mass movements are involved.

It is generally accepted (Skempton and Hutchinson) that the residual shear strength of a soil is independent of stress history effects, not influenced by specimen size, and rate-dependent to only a small extent. The major difficulty in determining the residual shear strength lies in the fact that large displacements may be necessary to achieve the required degree of orientation of the particles.

Morgernstern, N.R., Blight, G.R., Janbu, N., and Resendiz, D. (1977). Slopes and excavations, 9th Int. Conf. Soil Mech. and Found. Eng., 12, 547-604.

Skempton, A.W., and Hutchinson, J.N. (1969). Stability of natural slopes and embankment foundations. State-of-the-Art Report. 7th Int. Conf. Soil Mech. Found. Eng., Mexico, 291335.

back to Measurement of residual shear strength

Methods of measurement of residual strength

- Ring shear

The methods of measuring residual shear strength in the laboratory are given in the table. The most satisfactory methods, in many ways, are to obtain undisturbed samples which contain a natural slip surface and then test them either in the shear box or triaxial apparatus so that failure occurs by sliding along the existing slip plane. Alternatively, an artificial slip plane can be produced by cutting the specimen with a thin wire-saw. Much of the early work on determining the residual shear strength of soils in the laboratory was performed using multi-reversal type tests in the shear box on previously unsheared material (Skempton). The results of tests to measure residual shear strength in the shear box and triaxial apparatus have been reported by Skempton and Petley. There are practical difficulties with each of these tests, and they also have the major disadvantage that none of them permits the complete shear-stress-displacement relationship to be obtained.

Shear box
(a) Tests on natural shear surfaces
(b) Reversal-type tests
(c) Cut-plane tests
Triaxial
(a) Tests on natural shear surfaces
(b) Cut-plane tests
Ring shear

Skempton, A.W. (1964). Long-term stability of clay slopes. Geotechnique, 14, 75-102.

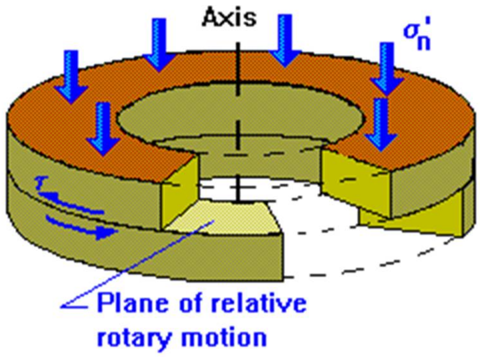
Skempton, A.W., and Petley, D.J. (1976). The strength along structural discontinuities in stiff clay. Proc. Geot. Conf. on Shear Strength of natural Soils and Rocks. Oslo, 2, 3-20.

back to Methods of measurement of residual strength

Ring shear

The large displacements required to define the complete shear-stress-displacement relationship can be obtained by using the ring-shear (or torsional shear) apparatus. The apparatus consists of two pairs of metal rings which hold an annular sample. The sample is subjected to a normal stress and then one pair of rings (normally the lower pair) is subjected to rotation. It is therefore a form of direct shear test, and failure occurs along a predetermined plane, as with the shear box.

this type of apparatus was probably first used by Hvorslev and Tiedemann. More recent designs of the ring-shear apparatus have been described by Bishop et al. and Bromhead.



Ring Shear Apparatus

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back to Measurement of shearing resistance

Difficulties

The aim of laboratory testing is to define a shear strength which is applicable to the field situation. Unfortunately, there are many reasons why laboratory tests may give values for shear strength different from those which apply in the field, and the main reasons are considered below.

The most obvious source of error is bad or indifferent sampling. it has already been emphasised that samples must not only be truly representative, but they must also be of the highest quality and this entails the use of well-designed apparatus which is in good condition and operated by skilled personnel.

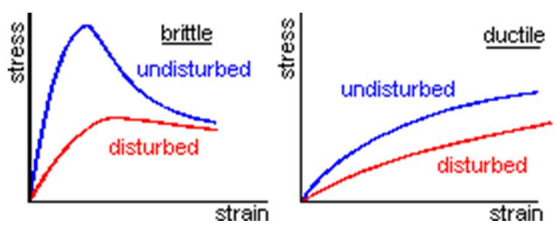
As a general rule sampling disturbance will tend to reduce the strength of the soil, and, as a further generalisation, it is likely that the shear strength parameters in terms of total stresses, (i.e. simple undrained tests) will be more greatly affected than the parameters in terms of effective stresses. The effect of sampling disturbance on the stress-strain relationship for brittle and ductile soils is given in the figure.

Most triaxial tests are performed on samples with a vertical axis, and the majority of shear box tests are performed so that failure occurs along a horizontal plane. In the field, however, a failure plane may be appreciably curved. In clays, anisotropy is likely to occur as a consequence of their mode of formation, and the presence of discontinuities such as joints and fissures which may exhibit some degree of preferred orientation. Some results indicating the anisotropy of undrained strength in London Clay are presented in the table. In terms of effective stress, it has been found that results of tests where shearing occurs in a horizontal direction are lower than of tests where failure occurs at other orientations, but there is a paucity of information on this topic.

To obtain realistic results from laboratory tests, it is essential that the tests are performed on samples which are sufficiently large to be representative of the in situ state. For intact clays, it is likely that typical laboratory specimens (i.e. 38 mm diameter for triaxial tests) are adequate for practical purposes, but in boulder clays or fissured clays, this may not be true. For London Clay, for example, it appears that the in situ undrained strength is around 65-70 per cent of the strength measured on a conventional 38 mm diameter sample, and accurate laboratory estimates of strength would only be obtained from tests on larger samples (possibly up to 300 mm diameter). Again there is a lack of information on the effect of sample size on the effective stress parameters c' and f ' of stiff fissured clay. Marsland and Butler report the following results for Barton clay:

38 mm diameter samples: c'= 11kPa, ϕ¢ =24deg
76 mm and 125 mm diameter samples: c'= 7kPa, ϕ¢ =23.5deg

A discussion of other factors which may lead to discrepancies between field and laboratory shear strengths has been given by Skempton and Hutchinson.



Ratio of undrained strength of London Clay parallel to bedding c_B and in compression specimens with their axis normal to bedding c_N (after Skempton and Hutchinson)

site	**clay**	**size of specimens**	$\frac{c_B}{c_N}$	**Reference**
Maldon	brown London Clay, shallow	38 x 76 mm	0.88	Bishop and Little (1967)
	brown London Clay, shallow	100 x 200 mm	0.86	Bishop and Little (1967)
Walton	blue London Clay, shallow	38 x 76 mm	0.78	Bishop (1948)
Wraysbury	blue London Clay, shallow	38 x 76 mm	0.75	Agarwal (1967)
	blue London Clay, shallow	300 x 600 mm	0.76	Agarwal (1967)
Ashford	blue London Clay, deep	38 x 76 mm	0.83	Ward et al. (1965)

Marsland A., and Butler, F.G. (1967). Strength measurements in stiff fissured Barton Clay from Fawley, Hampshire, Proc. Geot. Conf. on Shear Strength of Natural Soils a Rocks, Oslo., 1, 139-146.

back to Site investigation

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back to Slope stability

Remedial measures

- Drainage
- Restraining structures
- Modification of slope geometry
- Replacement
- Further reading

The factor of safety of a slope in soil possessing cohesion and friction can be written as

$$F_s = \frac{\sum\left[c'\ell + \left(W\cos\alpha - u\ell\right)\tan\phi'\right]}{\sum W\sin\alpha}$$

If, for a particular slope, the computed or actual factor of safety F_s is inadequate, clearly F_s can be increased by:
increasing the numerator (i.e. $\Sigma[c'l+(W\cos\alpha-ul)\tan\phi]$), or
decreasing the denominator (i.e. $\Sigma\Omega\sigma\iota\nu\alpha$), or
a combination of the above.

The main methods for achieving this increase in Fs, are: replacement; modification of slope geometry; drainage; use of restraining structures.
Detailed reviews of the wide range of remedial methods used in improving the stability of slopes are given by Hutchinson and Zaruba and Mencl.

Hutchinson, J.N. 1977. Assessment of the effectiveness of corrective measures in relation to geological conditions and types of slope movement. Bulletin of the Int. Assoc. Eng. Geol., 16, 131-155.

Zaruba, Q. and Mencl, V., 1982. Landslides and their control. Elsevier, Amsterdam; Academia, Prague.

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back to Remedial measures

Drainage

Drainage is one of the most widely used methods for improving stability. Clearly surface water must be removed and build-up of water pressures in tension cracks prevented. Subsurface drainage must be designed to reduce the water pressures acting on actual or potential slip surfaces; in this way, the value of the pore pressure (u) is reduced, thereby producing an increase in the factor of safety.

Several methods exist for subsurface drainage, including:
trench drains
horizontal drains
vertical drains (or wells)
galleries

Drainage may also be achieved by the use of electro-osmosis and by planting suitable vegetation.

Of these various methods, trench drains are frequently the cheapest and most widely used method. They are applicable to slips of moderate depth, but for deeper failures other methods may be more appropriate.

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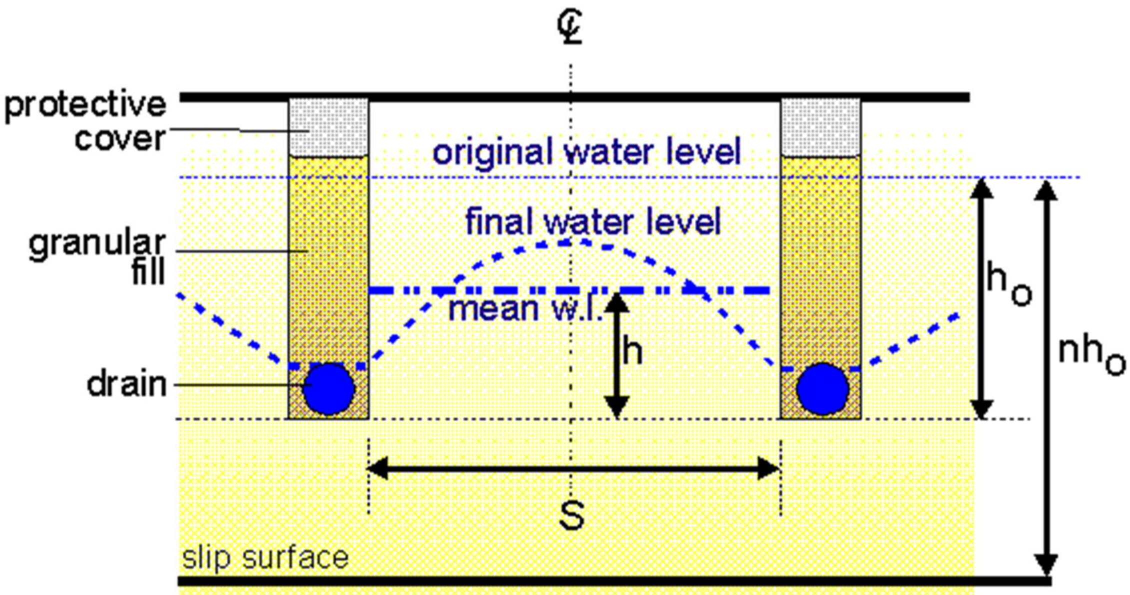
back to Drainage

Trench drains

Trench drains are normally constructed by machine; the drains are typically 0.5 to 1.0m wide and up to 7 or 8m deep. They are back-filled with suitable free-draining material with a porous pipe at the base to collect and remove the water. Provision to prevent clogging must be incorporated in the design. Ideally, the drain should penetrate through the slip surface (such drains are referred to as "counterfort" drains) and then in addition to the improvement in stability as a result of reduced pore water pressured on the slip surface, some additional restraint is achieved by the replacement of the weak slipped material by the stronger material in the drain.



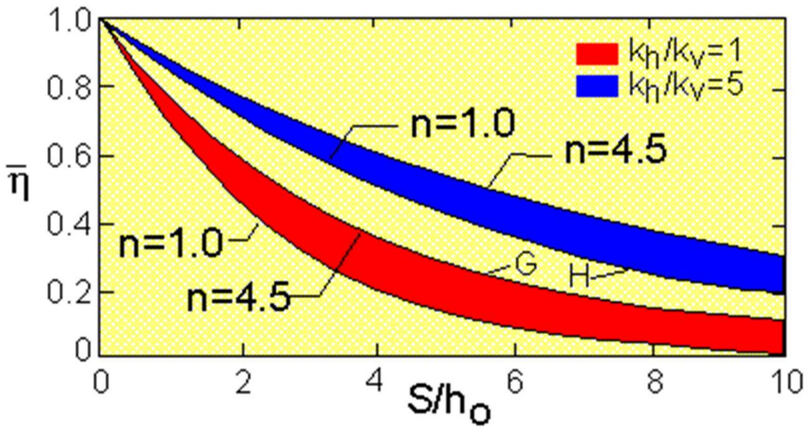
Trench and Counterfort Drains

back to Trench drains

Design of trench drains

- Example drainage design

An approximate method for designing trench and counterfort drains has been developed by Hutchinson using finite element analyses and assuming two-dimensional steady-state flow. Hutchinson used the results of the analysis to define the efficiency (η) of the drains and to relate the efficiency to the ratio s/h_o where s is the spacing of the drains and h_o is the depth of the drains beneath the groundwater level (see diagram). He suggested that Lines G and H can be taken as reasonable upper and lower bounds for the design of drains. For the purposes of preliminary design, curve G can be regarded as a conservative lower bound. Hutchinson also presented data from the long-term performance of drains at 6 sites, with encouraging results.



Efficiency of Trench and Counterfort Drains

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back to [Design of trench drains]()

Example drainage design

- [Solution without drains]()
- [Solution with drains]()

Consider a slope inclined at 9º to the horizontal in which a translational failure has occurred. The slip surface exists at a depth of 4m below ground level, and the phreatic surface has been located at 0.5m below ground level. If the shear strength parameters on the shear surface are given by $c'_r = 0$ and $f'_r = 16º$, what is the factor of safety of the slope in its present condition and what will be the factor of safety if trench drains 4m deep are installed at 10m spacings?

Take $\gamma = 20 kN/m^3$ $\gamma_w = 10 kN/m^3$

The introduction of these drains in the slope increases the factor of safety from 1.02 to 1.34.

[Example drainage design]()

Solution without drains

For a plane translational slip,

$$F_s = \frac{c' + (\gamma z - \gamma_w h)\cos^2\beta \tan\phi'}{\gamma z \sin\beta \cos\beta}$$

If c' = 0, then

$$F_s = \left(\frac{\gamma z - \gamma_w h}{\gamma z}\right)\frac{\tan\phi'}{\tan\beta}$$

In this example, γ = 20kN/m³ , γ_w = 10 kN/m³, z = 4m, h = 3.5m, ϕ'_r = 16º and β = 9º

Therefore, without drains,

$$F_s = \frac{(20x4) - (10x3.5)}{(20x4)} \cdot \frac{\tan 16°}{\tan 9°} = 1.02$$

Example drainage design

Solution with drains

The depth of the drains below the water table, h_o = 3.5m and their spacing, s = 10m.

Therefore s / h_o = 10 / 3.5 = 2.9

On the figure opposite, Hutchinson suggested that Line G can be taken as a reasonable upper bound for the design of drains. So, using Line G in this case,

for s / ho = 2.9, we obtain $\bar{\eta}$= 0.48

$\bar{\eta}$ is the average efficiency of the drains, and is given by:

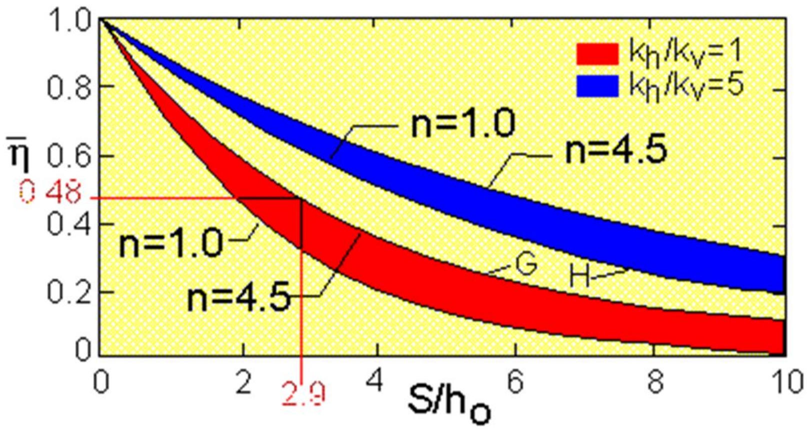
$$\bar{\eta} = \frac{h_o - \bar{h}}{h_o}$$

Thus for $\bar{\eta}$= 0.48 and h_o = 3.5m,

$\bar{h}$= 1.82m

The new value of F_s can be computed by substituting h = $\bar{h}$ in the appropriate equation:

$$F_s = \frac{20x4 - 10x1.82}{20x4} \cdot \frac{\tan 16°}{\tan 9°} = 1.34$$



Efficiency of Trench and Counterfort Drains

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back to Remedial measures

Restraining structures

Retaining structures such as piles, walls and anchors may be used to improve stability. It must be appreciated that the forces and moments to which these structures are subjected may be very large and hence careful design is essential. The detailed design of these structures is outside the scope of this section. Useful discussions are given by Zaruba and Mencl, Bromhead and Leventhal and Mostyn.

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back to Remedial measures

Modification of slope geometry

Changing the geometry of a slope to improve stability can involve the following:
excavation to unload the slope,
filling to load the slope,
reducing the overall height of the slope.

Where excavation and/or filling are used as remedial measures, it is essential that they are correctly positioned, and use should be made of the Neutral Point Concept.

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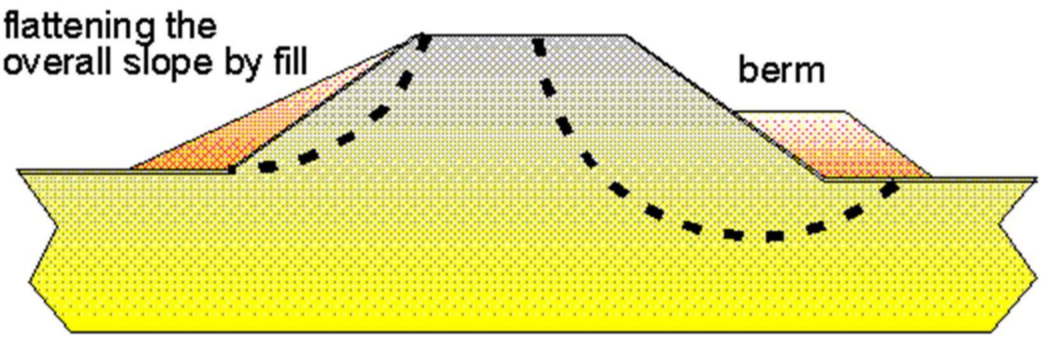

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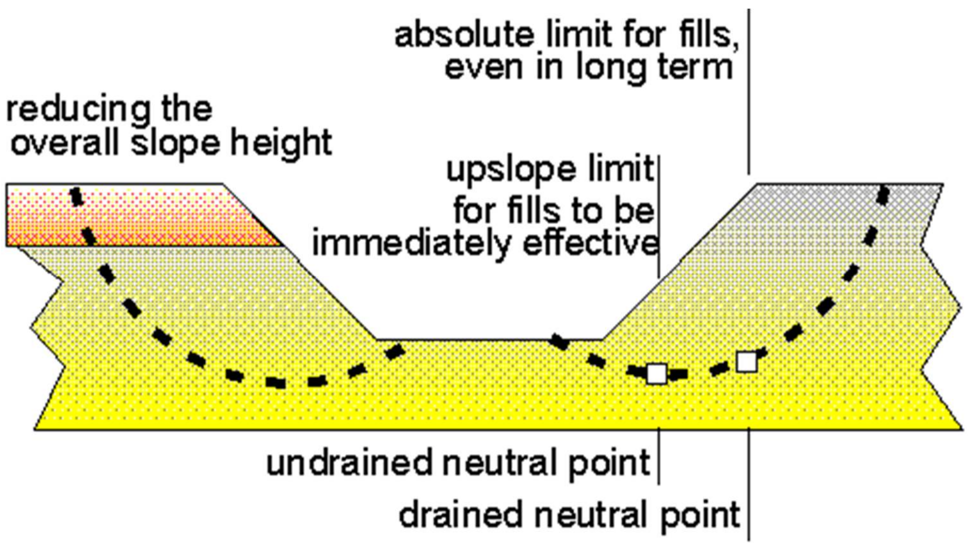

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back to Remedial measures

Replacement

Where the slip surface is not unduly deep, removal of all (or part) of the slipped material and replacement provides a relatively simple and straightforward remedial measure. The removed soil may be replaced by free-draining material (in which case some additional benefit may be achieved by drainage) or by the recompacted slip debris. If shear surfaces exist at shallow depth, they can be destroyed by digging out, remoulding and recompacting. A recent development has seen the incorporation of geotextile reinforcement within the replaced material.

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