IX. Mass Wasting Processes

1. Debris Flows

Flow types:

Debris flow, lahar (volcanic), mud flow (few gravel, no boulders)

Flowing mixture of water, clay, silt, sand, gravel, boulder, etc.

Flowing is liquefied with about 15% of water by weight.

Rheology: function of grain size distribution.

Mud flow $\rightarrow$ non-newtonian fluid

Wet grain flow $\rightarrow$ friction and collisions with pore pressure

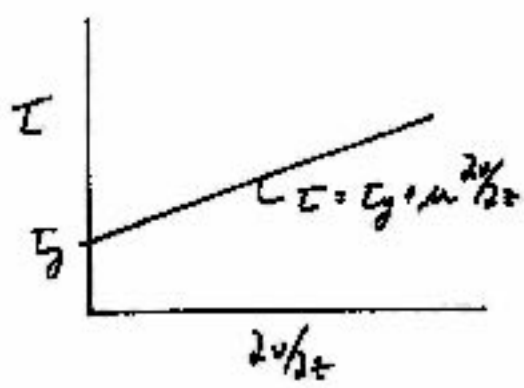
Most Debris flows: debated if more like fluid mud or more like wet grain flow.

Mud flows:

Visco-plastic (simplification)

$$\tau = \tau_y + \mu \frac{\partial u}{\partial z}$$

$$\frac{\partial u}{\partial z} = \frac{1}{\mu}(\tau - \tau_y)$$



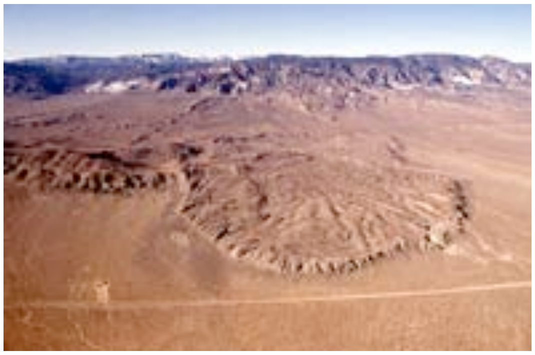
Simplification:

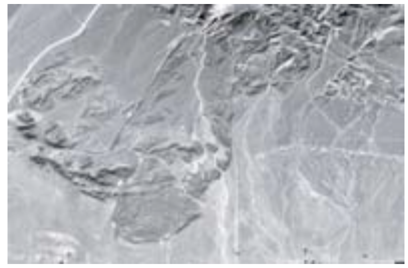
τ_y = constant (f(grain size, H_2O%))

μ = constant (f(grain size, H_2O%))

MOVIE SHOW (made by USGS in 1984) on Debris Flow Processes

1. Landslides





Types of Landslide:

Rock avalanches (Blackhawk slide is an example)

Rock fall (toppling of blocks)

Shallow soil landslides (tabular)

Deep bedrock landslides (tabular)

Earth flows (slow oozing $\Rightarrow$ reactivations over long time)

Rotational slumps

Infinite Slope Stability Analysis (initiation of failure)

- Assumptions
    1. 2-D planar failure at impermeable interface (no side-wall or end effects)
    2. Mohr-Coulomb failure criterion
    3. Slope-parallel groundwater seepage

F $\Rightarrow$ factor of safety

F = 1, at failure (or critical)

F > 1, stable

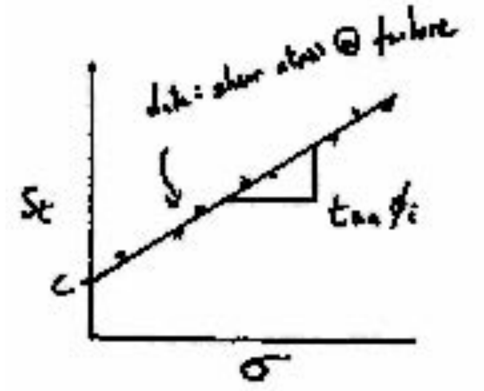
F < 1, unstable

$$F = \frac{\text{strength (resisting force)}}{\text{driving force}} = \frac{s_t}{\tau_{b_{wet}}} = \frac{c + (\sigma_{wet} - p)\tan\phi}{\rho_{b_{wet}} ghS}$$

where ϕ is the internal friction angle.

Infinite slope approximation $\Rightarrow$ no end effects, assume parallel seepage (uniform level of saturation)

$s_t = c + \sigma \tan\phi_i$ where ϕ_i is internal friction angle.



More generally, many factors (including root networks, capillary tension, weathering) influence the effective cohesion; also pore pressures reduce normal stress:

$s_t = c' + (\sigma - p)\tan\phi$; where c' is total effective cohesion.

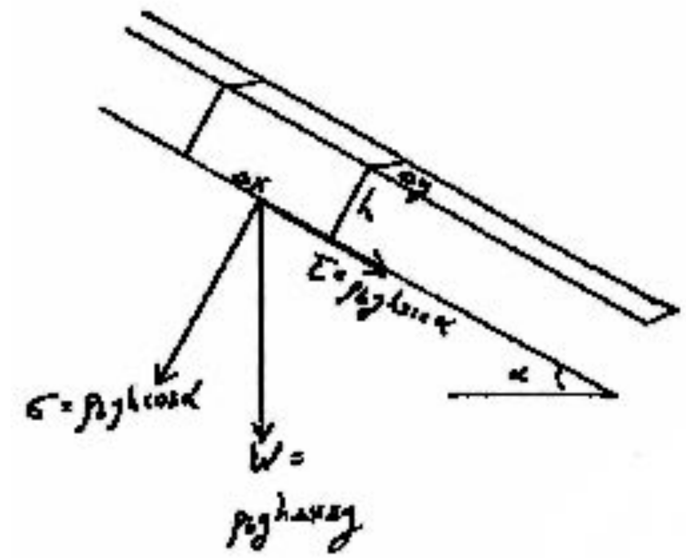
$F_s = \dfrac{s_t}{\text{driving stress}} \equiv 1$ at failure (by definition)

$\tau_b = \rho_b gh \sin\alpha$ where ρ_b is wet soil bulk density. {As derived earlier for unaccelerated fluids and a rigid block on an inclined plane}.

Wet bulk density: $\rho_b = v_s\rho_s + m(1 - v_s)\rho_w$, where v_s is volume fraction solids and *m* is fraction of soil depth saturated.

$\sigma = \rho_b gh \cos\alpha$ {normal stress component due to wet weight of soil}

SKETCH

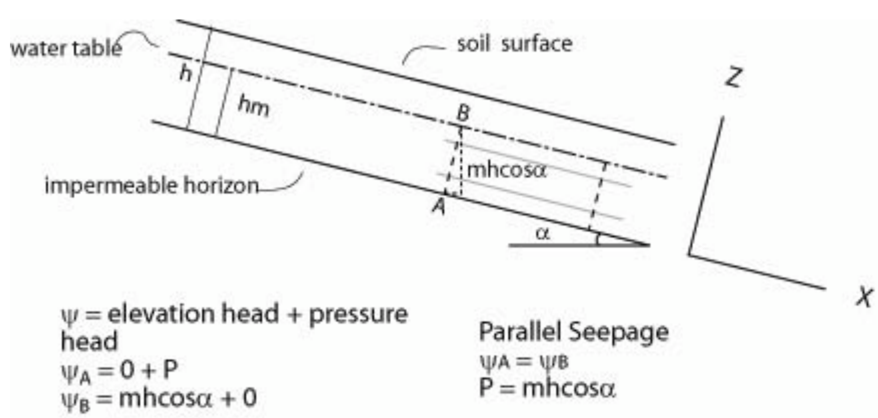


Thus write factor of safety equation:

$$F_s = \frac{c' + (\sigma - p)\tan\phi}{\tau_b} = \frac{c' + (\rho_b g h \cos\alpha - p)\tan\phi}{\rho_b g h \sin\alpha}$$

Pore pressure for parallel seepage (part of the "infinite slope approximation")

$$p = \rho_w g m h \cos\alpha$$



Substitute into factor of safety equation:

$$F_s = \frac{c' + (\rho_b - m\rho_w) g h \cos\alpha \tan\phi}{\rho_b g h \sin\alpha}$$

$F_s \leq 1$ failure

Implications for Cohesionless soil

$c' = 0$ if cohesionless

$$F_s = \frac{(\rho_b - m\rho_w)\tan\phi}{\rho_b \tan\alpha}$$

F_s =1 at maximum stable slope, set this and solve for maximum stable slope:

$$\tan\alpha_{\max} = \frac{(\rho_b - m\rho_w)\tan\phi}{\rho_b}$$

If dry, cohesionless, $m = 0$ and thus:

$$\tan\alpha_{\max} = \tan\phi, \quad \alpha_{\max} = \phi$$

Angle of Repose = angle of internal friction!

Seepage Forces

The above derivation was done in terms of pore pressures. An alternative formulation instead considers stresses due to the action of seepage forces (fluid drag on sediment particles). These are both equivalent – just different ways to cast the problem mathematically. The use of pore pressures was introduced to simplify the mathematics. However, for some problems, recasting in terms of seepage forces yields improved intuition.

We Follow the work of Iverson and Major (1986), WRR

Seepage Forces can act in both x- and z-directions and thus contribute to both normal and shear stresses. Generally:

Normal stress:

$$(\rho_b - m\rho_w)gh\cos\alpha + seepageforce(z)$$

Driving stress:

$$(\rho_b - m\rho_w)gh\sin\alpha + seepageforce(x)$$

where $\rho_b - m\rho_w$ is buoyant weight of wet soil. (*Note* we treat only buoyancy, not pore pressures)

In the case of parallel seepage, seepage force in (z) = 0; so normal stress is simply the normal component of the buoyant weight (intuitively satisfying)

In the x-direction:

$$f_{seepage} = \frac{q}{K}\rho_w g$$

where q is water flux per unit volume, so this is seepage force per unit volume.

Darcy's law: $q = K\sin\alpha$, where K is hydraulic conductivity.

$$f_{seepage} = \rho_w g\sin\alpha \qquad \text{\{per unit volume\}}$$

Thus, stress due to seepage force in (x) is = $\rho_w gmh\sin\alpha$ (ie. this is a force per unit area of soil, where mh is the height of soil column over which the seepage force acts). If you look at the expression for the shear (driving) stress:

$$(\rho_b - m\rho_w)gh\sin\alpha + seepageforce(x)$$

you see that the effect of the seepage force is to cancel out the effect of buoyancy – this is why the driving stress is the shear stress due to the full wet weight of the soil.

Substituting the seepage force term into the factor of safety equation yields:

$$F_s = \frac{c' + (\rho_b - m\rho_w)gh\cos\alpha\tan\phi}{\rho_b gh\sin\alpha}$$

The same relation – only a more intuitive, and more general, derivation.

Sub-aqueous Slope Stability

Consider a talus cone on the sea floor. Cohesionless material, fully saturated (m = 1). Is the angle of repose less than, greater than or equal to the angle of internal friction and why?

Recall for dry, cohesionless soil: $\tan\alpha_{\max} = \tan\phi$, $\alpha_{\max} = \phi$

If you try to address this problem in terms of pore pressures it can be confusing, and most students will guess the angle of repose is reduced due to the lubricating effects of water. However, if you consider the problem in terms of seepage forces, you will realize that there are no seepage forces involved because the water is *not moving*. Thus from above you can see that both normal forces and shear forces are due simply to the buoyant weight of the material, and for cohesionless soil $\tan\alpha_{\max} = \tan\phi$, $\alpha_{\max} = \phi$ -- exactly the same underwater, on dry land, on Mars, on the Moon, etc.

Non-parallel Seepage

Iverson and Major (1986), WRR, exploited the generalized treatment in terms of seepage forces to address the effects of non-parallel seepage on slope stability of cohesionless material under fully saturated conditions (m = 1). This problem is rather nasty in terms of pore pressures, but, as they demonstrated can be rather elegantly treated in terms of seepage forces.

They write:

$$\tan\phi = \frac{[(\gamma_t/\gamma_w)-1]\sin\theta + i\sin\lambda}{[(\gamma_t/\gamma_w)-1]\cos\theta - i\cos\lambda}$$

Where $\gamma_t = \rho_b g$, $\gamma_w = \rho_w g$, ρ_b is wet bulk density, i is the magnitude of the seepage force vector and λ is its orientation.

For parallel seepage $\lambda = 90°$ and $i = \sin\theta$. SKETCH

This readily confirms that the solution for parallel seepage is correct (same as we had above for case m = 1 and c' = 0).

From analysis of their equation above, Iverson and Major (1986) can discover generally what seepage directions are most destabilizing to the slope.

What is *your* intuitive ranking: vertical down, horizontal out, normal down, parallel seepage directions and why?

Normal down: increases stability. No effect on shear stress, counteracts normal buoyancy.

Vertical down: no net effect. Slightly increases normal stress, and equally increases shear stress.

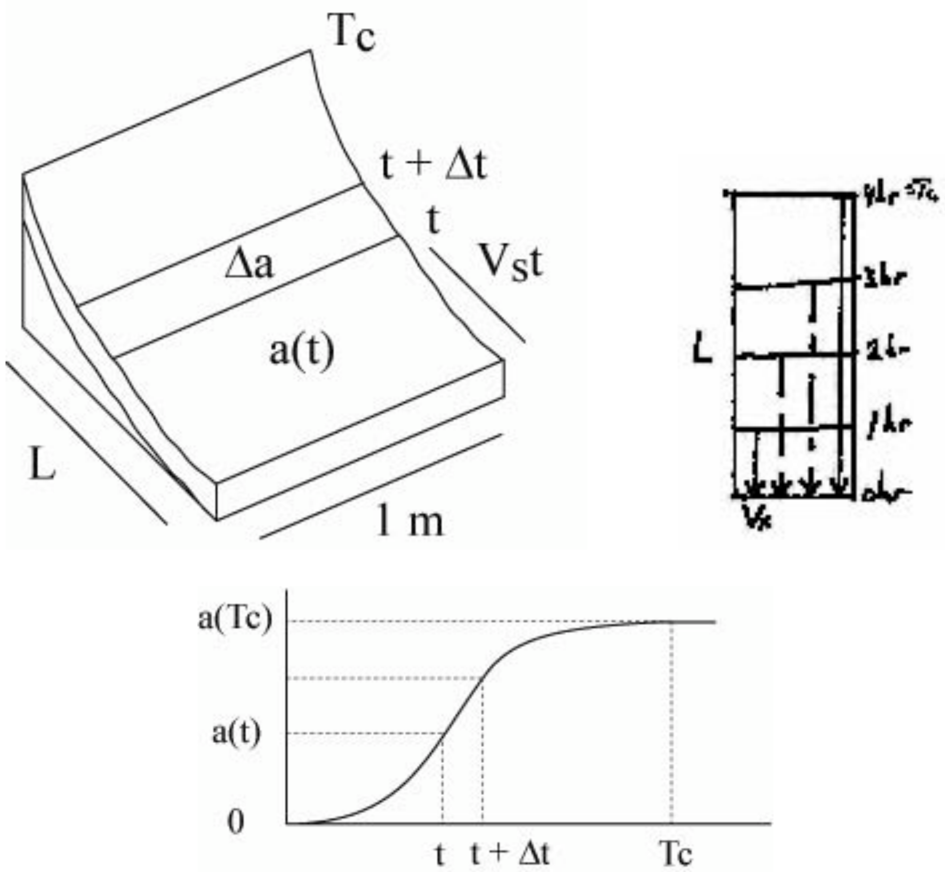
Parallel: decreases stability. No effect on normal stresses, but counteracts buoyancy in shear stress.

Horizontal: most destabilizing. Decreases normal stress and increases shear stress. Condition expected at base of slopes – this is one of the main deviations in nature from conditions assumed in the "Infinite Slope" Stability Analysis considered above.

Flow Convergence and Soil Saturation Levels

Iida (1984), Japanese Geomorphological Union considered the other major deviation in nature from conditions assumed in the "Infinite Slope" Stability Analysis: Flow convergence dictated by surface topography.

FIGURE: Iida, 1984 definition sketch: problem formulation



Assume unsaturated flow/storage is negligible; write relation conservation of mass (water)

$$q(t) = Ia(t)$$
$$q(t) = V_{darcy} h_{sat}(t) = V_{darcy} \Delta z(t) \cos\alpha$$
$$\Delta z(t) \cos\alpha = q(t) / V_{darcy}$$

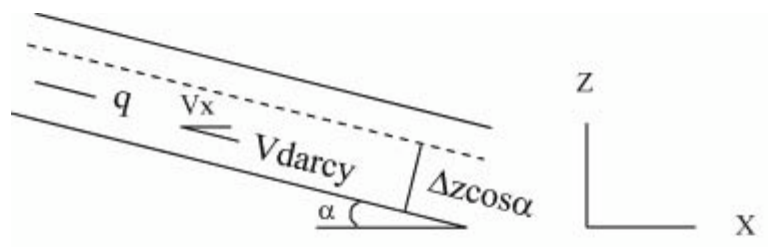
$q(t)$ is discharge/unit width, I rainfall intensity, $a(t)$ contributing area, $\Delta z(t)$ saturation level, and V_{darcy} is the Darcian velocity

For a straight slope (no convergence), constant slope (α)

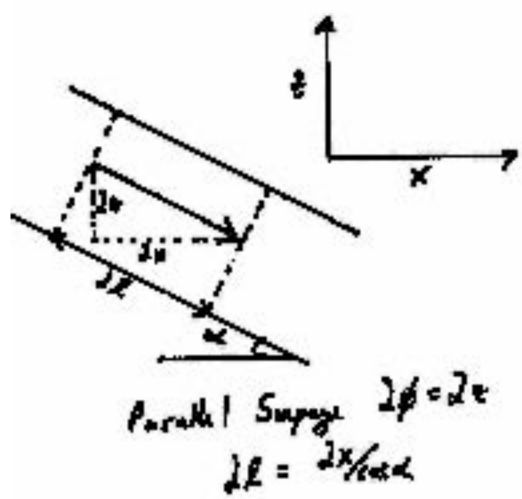
$$a(t) = V_x t \quad ; \qquad V_x = \frac{V_{darcy}}{\lambda_p} \cos\alpha$$

$$V_{darcy} = K \sin\alpha$$

V_x is horizontal component of interstitial velocity (porosity correction relates Darcy velocity to true interstitial fluid velocity, cosine term gives the horizontal component of fluid velocity), λ_p porosity, and K hydraulic conductivity



Why is $V_{darcy} = K \sin\alpha$ *?* – Darcy's Law



$$V_{darcy} \equiv K \frac{\partial \psi}{\partial l} \qquad \text{\{Darcy's Law\}}$$

Parallel Seepage:

$$\partial \psi = \partial z \quad ; \quad \partial l = \frac{\partial x}{\cos\alpha}$$

$$\frac{\partial \psi}{\partial l} = \frac{\partial z}{\partial x} \cos\alpha = \cos\alpha \tan\alpha = \sin\alpha$$

$$V_{darcy} = K \sin\alpha$$

$$V_x = \frac{K \sin\alpha \cos\alpha}{\lambda_p}$$

Combine above relations into conservation of mass:

$$q(t) = Ia(t)\ ;\ \Delta z(t)\cos\alpha = {q(t)}\big/{V_{darcy}}$$

to write:

$$q(t) = \frac{IK \sin\alpha \cos\alpha}{\lambda_p} t$$

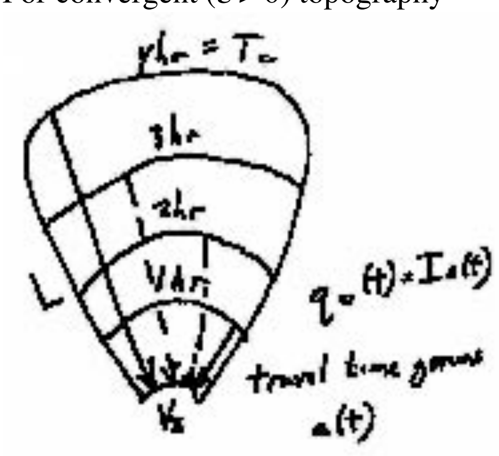
$$\Delta z(t) = \frac{It}{\lambda_p}$$

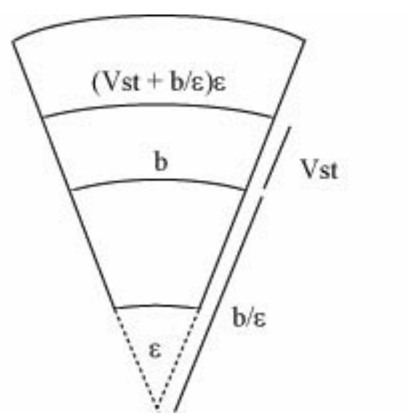
Steady-state solution (Δz_{max}; $t = T_c$)

$$T_c = \frac{L}{V_x} = \frac{L\lambda_p}{K \sin\alpha \cos\alpha}$$

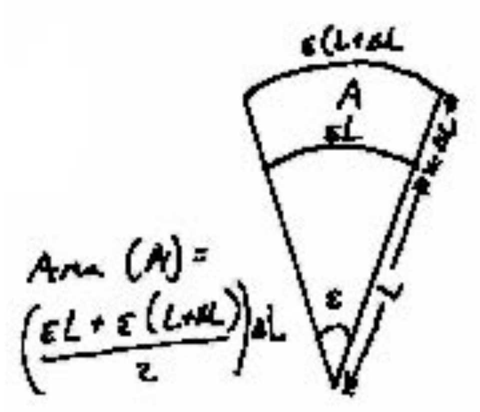
$$\Delta z_{\max} = \Delta z(T_c) = \frac{IL}{K \sin\alpha \cos\alpha}$$

For convergent ($\varepsilon > 0$) topography





Recall that arc length = $L\varepsilon$; Δa = average arc length x ΔL



$$\Delta a = \frac{1}{2}\left[\varepsilon L + \varepsilon(L + \Delta L)\right]\Delta L$$

$$L = \frac{b}{\varepsilon} \quad ; \quad \Delta L = V_x t$$

$$a(t) = \left[\frac{\left(V_x t + {}^{b}\!/_{\varepsilon}\right)\varepsilon + b}{2}\right] V_x t$$

$$q(t) = \frac{IK \sin\alpha \cos\alpha}{\lambda_p}\left[\frac{\varepsilon V_x t}{2} + b\right] t$$

$$\Delta z(t) = \frac{It}{\lambda_p b}\left[\frac{\varepsilon V_x t}{2} + b\right]$$

Comparison to the solutions for non-convergent topography (planar hillside), we see that the depth of saturation is enhanced by a factor of $\frac{1}{b}\left(\frac{\varepsilon V_x t}{2} + b\right)$, due to the effects of convergence.

Steady-state solution (Δz_{max}; $t = T_c$)

Recall

$$T_c = \frac{L}{V_x} = \frac{L\lambda_p}{K \sin\alpha \cos\alpha}$$

$$\Delta z_{\max} = \Delta z(T_c) = \frac{IT_c}{\lambda_p b}\left[\frac{\varepsilon L}{2} + b\right] = \frac{IL}{bK \sin\alpha \cos\alpha}\left[\frac{\varepsilon L}{2} + b\right]$$

At steady state, the saturation enhancement factor can be written entirely in terms of morphologic variables:

$$\frac{1}{b}\left(\frac{\varepsilon L}{2}+b\right)$$

Note, if storm is brief ($T_s < T_c$), peak Δz occurs after the storm ends, but is of lesser magnitude than Δz_{max}

FIGURE: Iida, 1984: $\Delta z(t)$ vs. ε

Limitations of infinite slope stability analysis

- Neglects 3-D effects
    - Neglects stress-field rotations (Anderson and Sitar, 1995)
    - Neglects flow through shallow bedrock fractures
    - Seismic loading affects both driving stress and pore pressures